CONCRETE CONSTRUCTION

A Practical Treatise on all Branches of Plain and Reinforced Concrete Construction

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PREFACE

THE primary aim of this volume is to describe the temporary and permanent works involved in reinforced concrete construction and to interpret the requirements of reinforced concrete specifications in terms of constructional procedure. It is hoped that much relevant data has been presented that will be of value to those concerned with the practical side of reinforced concrete construction.

The fundamental principles of site work, shuttering, reinforcement bending and placing, and concreting are dealt with in Chapters I to VI, while in the remaining chapters the application of these principles to the circumstances arising in the case of different types of structures is considered, together with any special features appertaining to the construction of foundations and superstructures.

In preparing the book a considerable amount of assistance, directly or indirectly, has been obtained from many engineers, contractors, and others. Acknowledgment for such assistance is specially due to Mr. Ernest A. Scott for observations on contractors' plant, to Mr. L. Henderson for information on steel scaffolding, to Mr. A. P. Mason for data on permanent joints and testing completed structures, to Mr. J. G. Sansom for data on bridge work, and to Mr. R. L. Hicks for his co-operation in preparing the diagrams.

Illustrations have been selected from numerous sources and the useful photographs of work under construction which are a feature of the monthly journal "Concrete and Constructional Engineering" have been freely used by permission. Finally, the author wishes to express his appreciation of the many suggestions and the assistance he has received from the Editorial staff of the Publishers throughout the preparation of this work and in reading the proofs.

C. E. R.

May 1938.

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CHAPTER I

PLANNING AND GENERAL EQUIPMENT

SECTION I.—REINFORCED CONCRETE CONSTRUCTION.

Design and Construction.

The success of a reinforced concrete structure depends as much on the care and skill displayed in the erection as on the exactness of the design. In the immediate future, major advances in this branch of structural engineering will probably be along the lines of improvement in materials and site processes rather than in the direction of refinements of theoretical calculation. In considering the relation between design and construction there should be no need to enlarge upon the necessity of co-operation between theory and practice. Such faults as poor and incorrectly located construction joints, violating the essential principles of monolithic design, should be self-evident, while the accurate disposition of the reinforcement in accordance with the assumptions made in the calculations can only be maintained if the designer appreciates the difficulties confronting the constructor. In turn, the latter, in his interpretation of the drawings, should be in a position to realise the designer's intentions.* Although this mutual understanding between the two principal branches is readily preserved in a contracting and designing organisation, it can also be obtained in other cases if the specification is reasonably drawn up so as to exclude requirements which are unnecessarily difficult to fulfil on the site and can be maintained by personal contact between the designer and those on the site.

Success in reinforced concrete construction depends not only on the co-ordination of sound design with good materials and expert workmanship, but upon a

* In the author's volume, "Reinforced Concrete Designers' Handbook," an endeavour has been made to keep before the designer the practical aspect of concrete construction, while in "Elementary Guide to Reinforced Concrete," by Albert Lakeman, design principles, the understanding of which is essential to conscientious construction, are set out in a simple manner suitable for those engaged on the erection of structures.

factor that is equally important from the client's view-point, namely, economy. Constructional economy results from careful planning. Economy in design depends upon judicious selection and use of the available materials and upon the disposition of the structural members in such a way that the loads are transmitted in the most efficient manner to the foundations. The economic relations between the three principal items—concrete, reinforcement, and shuttering—are appreciated by designing engineers, who should also realise that it is not always the design in which the quantities of these three are reduced to a minimum that results in the cheapest structure. For instance, in an attempt to decrease the concrete quantities the shuttering may become so irregular and non-repetitive that the increase in its cost outweighs any advantage gained by reducing the quantities. Similarly a reduction in the weight of the reinforcement may result in complicated bending.

Preliminaries to Construction.

With specified materials and given designs, the best opportunity remaining to the contractor to effect economy is the programme of construction. This will entail a study of all aspects of the job, the method of carrying out successive operations, and the installation and general arrangement of the plant. Many of the preliminary steps in a contract will have been taken with more or less precision when the contractor makes up his tender. After the production of the general and certain detail designs by the engineer, the first step is thoroughly to study the drawings and specification to grasp the essential particulars and requirements. If the contractor prepares the designs he will gain familiarity with these requirements when the preliminary schemes are being produced.

A study of the bill of quantities will indicate the various trades involved and the amounts of materials required. Where bills of quantities are not supplied by the engineer or architect, the contractor has to prepare his own, and the bill should be taken out in sufficient detail to enable him closely to estimate the amount of permanent and temporary material to be provided. It is unusual for a bill of this kind to be prepared in such detail as occurs in a bill prepared by an engineer or quantity surveyor for inviting tenders or for assessing payment due to the contractor for work executed at schedule rates. If it is proposed to carry out any of the work by sub-contract, a detailed bill covering that part of the work may be necessary and, if such a bill is not supplied to the principal contractor, the latter may have to prepare a bill himself for submission to the sub-contractor. If facilities for preparing a bill are not available within the contractor's organisation, this work is best entrusted to a quantity surveyor. If the contractor decides to prepare his own bill, it should preferably be compiled in one of the standard forms depending on whether the contract is classed as a building or a civil engineering work.

The preliminary study of the drawings, quantities, and other contract documents will assist in determining the plant required, and the size of the job will fix the sum that can reasonably be spent on labour-reducing items such as concrete mixing and placing plant, equipment for handling and transporting material, cranes and hoists, excavators, bar-benders, woodworking machinery, and pneumatic tools. Practicable and alternative arrangements of plant must be considered in relation to the type of work, the site, and the means of access to it. The problems associated with plant installation and with labour are interconnected. Therefore, in considering mechanical equipment, such items as local rates of wages, availability of skilled and unskilled men, housing for workmen, and kindred matters must be determined. Since the "Conditions of Contract" sometimes requires a certain proportion of the workmen to be obtained from a specified source, the competence of these men must be considered. The "Conditions" will also influence arrangements for financing the work during progress, since clauses are usually inserted to cover the method of certification and payment, legal charges payable by the contractor, the amount of retention money (if any), the probable allowance of bank interest on retention money, the penalties and bonus for variation from the contract time, and the conditions of maintenance upon completion of the work. "Prime Cost" items included in the bill of quantities and the discounts on these should be examined during the preparation of the tender. Some contract documents define "prime cost" as the sum paid to the merchant after deducting all trade discounts but not deducting discount for cash. Others specify that the "prime cost" shall be the sum paid by the contractor to the merchant after deducting all trade discounts and any discount for cash in excess of 2½ per cent. In building contracts governed by standard "Conditions of Contract" "prime cost" items are subject to 5 per cent. discount. To any "prime cost" item it is necessary to add profit and, if not allowed elsewhere, additions must be made for the cost of unloading, handling, storing until required, and hoisting and fixing the goods or materials.

Before a tender is submitted the site must be inspected and any working restrictions peculiar to the proposed contract must be examined, since it is on an initial understanding of these difficulties that the economic success of the job usually depends. During the course of the job the unforeseen difficulties are sufficient without being increased by problems that should have been anticipated and solved before the work begins.

Site Inspection.

Foremost among the points that should be settled during inspection of a site is the availability of suitable materials necessary to the contract. The means of access to the site, whether by road, rail, or waterway, should be studied. For a large job the economic possibility of installing a special railway siding could be considered. The means of access should also be thought of in terms of the material merchant's available means of transport. The distance of the site from the contractor's office and yard, upon which freight charges and travelling expenses will depend, should also be determined. When the site is remote from a railway or public road the possible lines of temporary roads or railways should be mapped out and, where necessary, permissions obtained to cross private property. Provision must be made for respecting any rights-of-way across the site or land to be occupied by the contractor or arrangements must be made in agreement with the local highway or other authority, who should also be approached when the work involves the diversion of traffic or the enforcement of a speed limit on road or rail traffic. The space available to the contractor for erecting plant or storing material around the work should be clearly defined and agreed, and the probable cost of making-good damage to adjacent property should be determined. When the contract involves piling, the effect of vibration on nearby structures should be considered. Allowance must also be made for shoring-up existing buildings adjacent to deep excavations, and the effect of excessive removal of ground water by pumping in excavations must be considered. If crushing plants are installed, these should be so disposed that prevailing winds will not blow dust on to adjacent growing crops or elsewhere to do damage for which the contractor might be liable to pay indemnities.

Available power and water supplies and the rates of payment must also be investigated. If water can only be obtained from natural sources, the suitability of the water for concrete purposes should be considered. Observations on the quality and quantity of water required for a reinforced concrete contract are given in a subsequent chapter. If water is taken from a public supply arrangements should be made with the water company for connecting up the contractor's pipes, and an appropriate sum should be included for this in the tender. The character of the available power should be considered in relation to that required to operate the contractor's stock plant. If the latter is steam driven and electric power can be obtained free or at low cost, the comparative costs of running on steam or converting to electric drive should be investigated. If steam or oil-driven plant is used, the source, cost, and means of delivery of the coal, oil, or other fuel should be determined.

In any riverside work involving temporary damming, diversion, or other restriction of the flow of the stream, the probable effects of raising the water level upstream or decreasing the flow downstream should be investigated. The former may cause flooding and the latter may cause inconvenience and loss to persons downstream who depend on a certain flow being maintained. Care should be taken not to pollute waters by discharging undesirable matter into them when pollution would be detrimental to downstream users. Fishing rights should also be respected, and the need for navigation lights or other warnings should be foreseen. The possibility of floods or unusual flow should also be considered in relation to the contract work, and for marine structures tidal movements should be obtained for the appropriate period of the year. These can usually be obtained from the local harbour authorities, or " tidal tables " will give the necessary information for the port nearest the site ; adjustments have to be made to obtain local times and levels and the differences can be obtained from local authorities or, failing that, from direct observation. It should be remembered that nominal high and low-water levels can be considerably altered by such conditions as the outline of the coast or inlets, prevailing winds, currents, and varying depths. Wave crests under normal conditions may be a few feet above nominal high water and strong inshore winds may result in higher tides than would otherwise be expected. When considering the effect of waves on stagings, coffer-dams, or other temporary structures, the fact should be realised that green water will subject such structures to a greater buffeting than does " white " water, since the latter is largely cushioned with air bubbles.

The necessity of providing watchmen during the course of the contract and the number required can also be determined in the preliminary site inspections, together with any local regulations regarding the licensing and provision of hoardings and the provision of fire-fighting apparatus. At the same time, particulars of any special conditions that may limit the work in any way should be noted. For example, for work within existing buildings, there may be a restriction on the length of timber or reinforcing bar that can be conveniently handled and it may be necessary to handle the concrete in small quantities or place the mixers away from the place where the concrete is to be deposited. These and similar matters will affect the cost of the work.

Before the cost of foundation work can be estimated it is essential to have particulars of the nature of the ground to be excavated, for which purpose it may be necessary to dig trial holes, drive trial piles, or resort to boring. The information obtained should give a description of the ground at varying depths and should be in sufficient detail to determine whether timbering or pumping will be required. Particulars of these operations, together with descriptions of loading tests on ground of uncertain carrying capacity, are given in a subsequent chapter (XI).

Contract Time and Construction Programme.

Among the matters to be considered when planning a contract is the important factor of time. The period that elapses between taking possession of the site and handing over the completed structure to the client depends primarily upon the volume of work to be executed, that is upon the quantities, but the length of this period can be materially affected by such factors as choice of plant, arrangement of plant, continuity of supplies of materials, the type of cement used, prevailing weather, labour troubles, and kindred matters. Dependent upon the time taken will be such items as the cost of the staff on the site, depreciation or hire charges of plant, availability of plant for other work, finance, insurance and, in the estimating stage, the chances of securing the contract. In preparing the estimate it is usual to draw up a rough construction programme to determine an approximate contract time; sometimes a number of programmes adopting different equipment or methods may be prepared so that the method of tackling the work in the most efficient and economical method can be established. When construction is about to commence it is necessary to prepare a more detailed programme in order to correlate supplies of materials, the various trades, the re-use of shuttering, and to ensure that the job shall be undertaken in such a way from the start that completion within the contract time is practicable.

Some examples of preliminary estimates of the time required are given on *Fig.* 1 for a number of different classes of work. After the initial stages it is usually necessary to consider only the time required to carry out the main operation (usually concreting for reinforced concrete jobs) and the period of lag of subsidiary operations such as striking shuttering, filling, surfacing, dismantling and removing plant, and clearing up. Among the preliminary operations that occupy time at the beginning of the job are the clerical work following the receipt of the order, the interval between receiving the order and taking possession of the site, placing orders for and awaiting delivery of materials, preparation of and obtaining approval of sufficient designs to commence work, obtaining labour, delivery of plant and equipment for the staff, erection of plant and barriers, serving

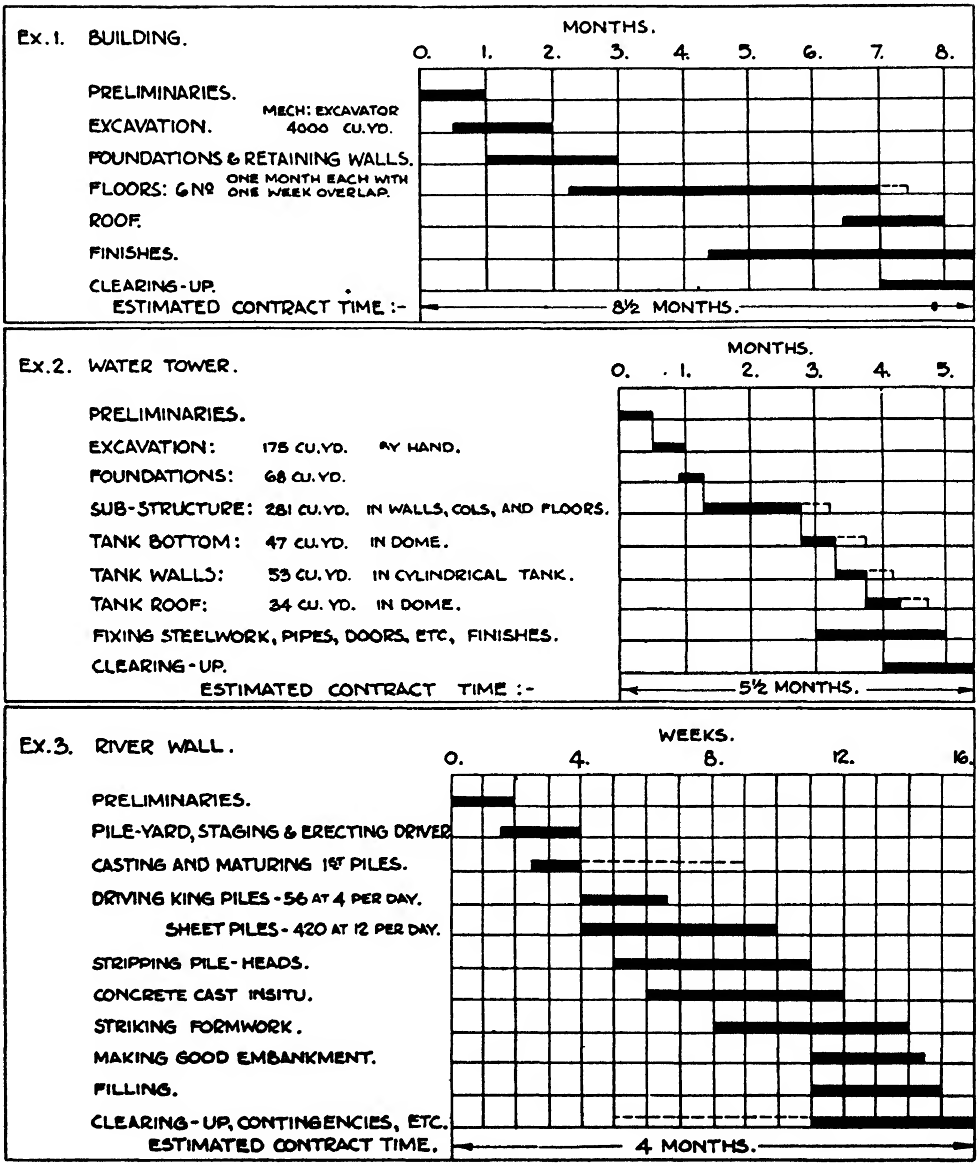


FIG. 1.—ESTIMATING CONTRACT TIME.

notices to authorities, and setting out the work. Most of these operations run concurrently.

The time taken for the principal construction can be estimated from the quantities of material and labour in each stage and the available equipment. In repetition work, alternative estimates can be prepared to determine the most economical number of units that can be constructed simultaneously by balancing the saving in time against the increase in the number of plant installations or sets of shuttering. On tidal work the contract time is determined by the available hours of sufficiently low water, paying due regard to seasonal changes of tide levels and daylight hours at low tide. On large jobs or important and urgent work the saving in time and increase in cost due to introducing a continuous working day, with night and week-end shifts, should be investigated. On small jobs it may be necessary to take into account the incidence of public and local holidays. The amount of work that can be done in a working day will depend primarily on the plant installed. Thus the rate of concreting will be determined by the output of the mixer or the speed with which the concrete can be transported from the mixer to the shuttering and the possibility of placing concrete continuously. The output of concrete mixers is considered later. The quantity of shuttering provided for repetition work, such as identical floors or precast piles, will also decide the rate at which the work can proceed when considered in relation to the rate of hardening of the concrete.

The time occupied in excavation depends primarily on the depth and nature of the soil and the use of manual labour or mechanical equipment. It is also affected by the available means of disposing of the spoil. In assessing the time required for mechanical excavation, allowances must be made for changing position, bad weather, and breakdowns.

Certain features should be noticed in connection with the examples on *Fig.* 1. It will be realised that, for the building, operations on successive floors can overlap considerably, whereas for the water tower the commencement of each successive operation is dependent on the completion of the preceding operation. On the building job, in the absence of frost, concreting is always in progress on one part or another, but for the water tower concreting is intermittent as intervals have to be allowed for the erection of shuttering and fixing the steel. As far as possible the shuttering for any particular stage would be prepared and the reinforcement bent beforehand, so that both are ready for erection as soon as concreting the previous stage is completed. Where the operations include concreting, the ends of the black lines on *Fig.* 1 indicate the completion of concrete placing, the broken-line continuation indicating the period of striking the shuttering.

Sequence of Operations.

Before compiling a detailed programme of construction it is essential to realise the proper sequence of operations and the interdependability of one operation upon another. In a reinforced concrete structure the three major operations are making and erecting the shuttering, fixing the reinforcement, and placing the concrete. Consideration of each of these involves (i) supply of material, (ii) preparation and erection of material, and (iii) any appropriate subsidiary operations. An idea of the relative values of the main stages of the work can be obtained from the following typical analysis of costs in which the cost of each operation is expressed as a percentage of the gross cost.

Type of Contract	Bridge. Excluding foundations, filling, masonry facing, drainage, etc.	Silo. Excluding foundations, fittings, windows, etc.	Water Tower. Excluding foundations, stairs, doors, windows, pipes, etc.	Bunker. Excluding foundations, fittings, etc.
Approximate total cost including profit	£50,000	£8,000	£3,000	£600
	Per cent.	Per cent.	Per cent.	Per cent.
Concrete materials	20	16	24	14
Shuttering materials	14	6	12	7
Reinforcement materials	14	12	10	11
Concrete labour	6	7	6	7
Shuttering labour	14	21	13	20
Reinforcement labour	12	9	4	5
Plant (including depreciation, carriage, erection, power, etc.)	3	8	9	6
Small tools, consumable stores, workmen's insurance, etc.	2	2	2	3
Site offices, cement and other stores, men's shelter, water, lighting, hoardings, etc.	1	2	4	8
General site staff, head office costs, supervision, watching, etc.	4	8	3	7
Miscellaneous (including testing structure, making good and rubbing down, insurance of works, etc.	1	—	4	3
Profit	9	9	9	9

A detailed construction programme can be made up in a form similar to the example given on *Fig.* 2, which is an elaboration of the first of the approximate contract time schedules shown on *Fig.* 1. Although the contract times for the two programmes are necessarily identical, it will be noted that the periods for each operation are not precisely the same, as minor variations naturally seem reasonable when preparing the more detailed consideration. Each operation must be fitted in its correct sequence in the work. Consideration of the methods to be adopted for the excavation determines the amount of soil that can be removed in a working week, while similar consideration of the number and output of the mixers gives an idea of the time required to place a given volume of concrete. The essential calculations for these two operations are given on *Fig.* 2 and, based upon the known quantities in each section of the work, a tentative programme can be made up. Work can be found for two mixers practically throughout the whole course of the work. In the early stages one mixer prepares the concrete for the blinding layer and afterwards the concrete in the foundation raft, while the other mixer supplies the concrete for the retaining wall and afterwards for the internal columns up to the ground floor. During the floor construction one mixer is operated solely for the concrete in the slabs and beams, while the other provides the material for the columns, walls, and stairs. Since the latter work is only intermittent the second mixer can be used in the intervals to assist the first on the construction of the floors. There must necessarily be a lag between successive operations; for example, during the floor construction there would be an interval of a few days between the commencement of the erection of the

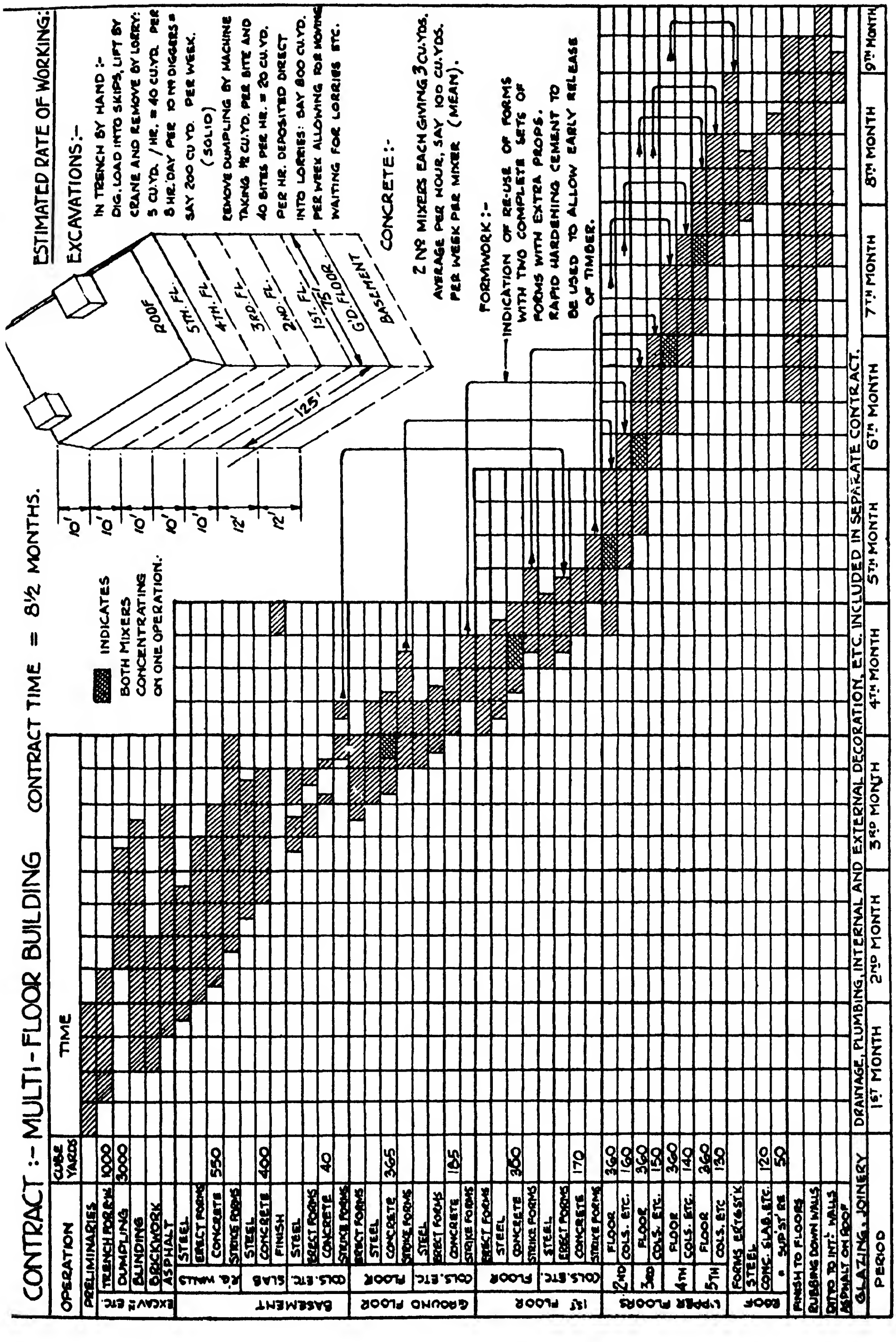


FIG. 2.—EXAMPLE OF JOB CONSTRUCTION PROGRAMME.

shuttering and placing the reinforcement so as to allow the carpenters to prepare sufficient shuttering to enable the steel for complete panels of floor to be fixed. Similarly there would be a lag between placing the steel and concreting to allow the steel-fixers to get clear ahead of the concreting position and to allow the assembled steel to be inspected.

One of the most important factors in determining the time taken to construct a particular job is the treatment accorded to the shuttering problem. If in the job illustrated in *Fig.* 2 the use of rapid-hardening Portland cement is decided upon, so as to release the shuttering as early as possible, it is seen from an inspection of the diagram that two sets of floor shuttering only need be supplied, each set being used on alternate floors or columns and walls. Enough time is allowed between successive uses of each set of shuttering to allow for cleaning, adjusting, repairing, and hoisting to the new level. Since the props under beams would be left in longer than the sheeting, more than two sets of props would be required.

SECTION II.—CONSTRUCTIONAL EQUIPMENT.

Plant Requirements.

The diversity of the operations connected with reinforced concrete construction demands a variety of appliances and equipment, the more common items, any or all of which may be required for one contract, being as follows:

GENERAL EQUIPMENT USED FOR MORE THAN ONE OPERATION:

- Transport equipment including standard-gauge or jubilee tracks, wagons, bogies, trucks, locomotives, haulage ropes, winding drums and operating engines, motor lorries, steam wagons, horse-drawn carts, barrows, etc.
- Derrick, gantry, locomotive and other types of cranes, manual or power-operated block and tackle hoists, winches, gin wheels, wire or hemp ropes, chains, etc.
- Boilers and steam piping.
- Water piping, valves, taps, hoses, water cans, buckets, etc.
- Scaffold poles, putlogs, planks, wire or rope lashings, etc.; tubular steel scaffoldings; ladders, ramps, steps, etc.
- Concrete breakers and compressed-air plant.
- Offices, workmen's shelters, stores, desks, stationery, thermometers, ambulance case, stoves, pay-rack, and other office equipment.
- Small tools including hammers, chisels, spanners, crowbars, nailbars, jim-crows, stocks and dies, spades, shovels, picks, brooms, tarpaulins, and other protective coverings.
- Consumable stores including grease, oils, coal, nails, screws, bolts, washers, wire, dog-spikes, etc.
- Warning lamps and notices, electric or other lighting equipment, fire-extinguishers, etc.
- Workshops, etc., and equipment for repair of mechanical plant.
- Tapes, chains, setting-out pegs, ranging-rods, levels, theodolites, etc.

PLANT USED IN FOUNDATION WORK ONLY:

Excavating machinery, mechanical shovels, draglines, trench-diggers, scrapers, grabs, etc.
Timbers, pumps and equipment.
Pile-driving equipment including power-hammers, drop hammers, hammer-operating gear, winches, frames, dollies, packings, etc.
Boring tools or other apparatus for testing the ground.

PLANT FOR CONCRETING OPERATIONS:

Aggregate crushing, screening, and washing apparatus.
Aggregate bins and cement stores.
Aggregate and cement batch-boxes, automatic batching plants.
Concrete mixers and engines, hand-mixing platforms, timber bankers, etc.
Concrete skip hoists, elevators, masts and guys, chuting plant, conveyors, concrete pumps, grouting pumps, piping and hose.
Barrows, concrete carts, crane skips, wagons, bottom-opening skips, tremies, runways, etc.
Tamping tools, vibrators, etc.
Trowels, floats, screed boards, brushes for applying grout wash, paints, etc., wire brushes, carborundum blocks, rubbing-down machines, tooling machines, indentation rollers, and other apparatus involved in surfacing processes.
Apparatus for testing materials, slump cones, and moulds for test cubes.

REINFORCEMENT PLANT:

Hand or power-operated bar cutting machines, bolt croppers, wire cutters, etc.
Bar-straightening machines, scaling brushes, etc.
Hand or power-operated bending machines and benches.
Concrete, steel, asbestos or similar bar spacers, binding wire, wire ties, ferrules, turnbuckles, etc.

SHUTTERING PLANT:

Carpenters' benches, vices, etc.
Circular saws, planes, and other woodworking machinery.
Brushes for applying retarding liquids or mould oil.
Retarding liquid, mould oil, scrapers for cleaning shuttering, caulking material, packing for use in cold weather, etc.

Consideration is given in later chapters to the plant required for such operations as handling aggregates, mixing, hoisting and placing concrete, and bending reinforcement. The present section deals with items in the foregoing list that cannot be allocated to a specific operation but may be used in a number of operations.

Transport.

Methods adopted for transporting materials and plant to and about the site vary considerably and depend upon the locality, the nature of the ground, the type and quantity of the load, and the distance of haul. The power can be supplied by men, by animals, mechanically by steam-, electric-, petrol-, or Diesel-driven power units acting directly or by rope haulage. The containers may be wheelbarrows, hand-carts, lorries, railway trucks, contractor's wagons, or wagons of the jubilee or Decauville type. The tractors, wagons, or carts may be fitted with road-wheels or rail-wheels, and the tractors may travel on " caterpillar " tracks. Haulage may be over earth surfaces, along existing highways, on sleeper roads or other temporary timber tracks, or on rail-tracks of broad, standard, or narrow gauge. Rail-tracks may be laid directly on the ground, on embankments, or on temporary stagings of timber or steel. On the other hand, transport may be conducted along waterways or, abroad, by methods specially adapted to local conditions.

With such a wide choice, it is clear that no hard-and-fast rules can be laid down ; general comments serve only to indicate present-day practice in this country.

Materials are brought to the site in railway-trucks, road-lorries, or by barge, depending upon the facilities at the site and at the source of supply. Motor-lorries or steam-wagons would be used on city sites or where good roads connect the source of the aggregates and the site. For large contracts alongside rivers or canals, materials can usually be delivered in bulk by boat or barge and discharged by cranes operating either grabs or hand-filled skips. Economical transport by water or rail necessitates the source of material supply as well as the site being alongside a waterway or railway. Obvious exceptions to this occur in the case of work abroad and other cases in which the distance to be travelled is great. Owing to the widespread distribution of sources of aggregate and cement works in this country it usually pays to transport materials by road throughout the whole distance if any part of the journey involves road travel. The foregoing remarks also apply to the transport of concrete mixers, scaffolding, and other equipment. The carriage of reinforcement is affected by circumstances that are considered in a later chapter. Motor lorries are used extensively to convey workmen to and from isolated sites or to contracts remote from the contractor's own district.

Excavated materials from open sites are conveyed by jubilee wagons, contractor's carts, or standard-gauge railway-trucks to the dump, but on city sites the material is usually loaded directly into motor-lorries.

On jobs of small or moderate size the materials are usually conveyed from the material heaps to the mixers in wheelbarrows, in which concrete is often transferred from the mixer or head of the hoist to the concreting position. Hand-carts are also used for the latter purpose, but where the distance from the mixer to the boundaries of the work is considerable jubilee or similar wagons are employed for transporting concrete and materials to and from the mixers. Various methods of handling materials and distributing concrete are discussed in subsequent chapters.

The tractive power required depends on the load and the type of surface. On rough firm earth, not too soft, the tractive effort required is about 5 per cent. of the load and may be as high as 10 per cent. If the ground is very soft it is better to lay corduroy tracks of timber sleepers or to roll in rubble or broken brick ; such prepared tracks may reduce the effort required on the level from above

10 per cent. to less than 3 per cent. of the load hauled. Macadam roads require a tractive effort of about 2 per cent., wood paving $1\frac{1}{2}$ per cent., and asphalted roads less than 1 per cent. Well-laid rail-tracks entail an effort of about 1 per cent. at the speeds associated with construction work, but usually the tracks about the site are imperfectly laid and this value is considerably increased. Acceleration and gradients greatly increase the tractive effort required.

Steam locomotives are generally used on large civil engineering contracts such as dock or railway construction or where large quantities of bulky materials have to be conveyed considerable distances. Internal-combustion engines, from 10 up to 100 h.p., are more common on extensive reinforced concrete structures. Low-power engines usually work on petrol, and high-power engines on heavy fuel oil. When the tracks are tolerably straight but involve heavy gradients, rope haulage, the power for which is provided by a stationary engine or electric motor, is advantageous.

Horse-drawn vehicles, although tending to be superseded by mechanical traction, are still used on occasions. On prepared hard earth or macadam roads a single horse can draw on the level a useful load of over half a ton throughout a working day, while on well-laid stone blocks up to $1\frac{1}{2}$ tons constitutes a useful load, or 3 tons on an asphalt road. On ordinary soft earth a load of $\frac{1}{4}$ ton in addition to the weight of the cart represents the uniform exerted capacity of a single horse, while on dry sand only half this figure is attained. For short periods higher loads can be transported.

For ordinary wheelbarrows with iron or with pneumatic tyres, or hand-carts pushed by one or two men, timber tracks are required to reduce the effort necessary. These tracks may be simply planks for single-wheeled barrows or three-plank runways for the two-wheeled carts. Such tracks are adjustable to the exigencies of a site. Gradients for hand-propelled barrows or carts should not generally exceed 1 in 30, or more than 1 in 12 in extreme cases over a few feet. If the hauling distance approaches 100 yd. it is usually worth while considering the employment of horse-drawn or mechanically-propelled vehicles. Even for short distances, if the volume of material to be moved is considerable, it usually pays to lay rail tracks for hand operated trucks of the jubilee type.

Cranes and Other Hoisting Gear.

Cranes can be divided broadly into two classes, stationary and travelling. The former is generally more suitable for buildings and similar structures that have not excessively long dimensions, but if the work extends a considerable distance in one direction, as in wharf construction, a travelling crane is more useful. Much depends upon the circumstances controlling any particular job as well as the plant available and the height of the work. A number of the diagrams and illustrations in the following pages demonstrate applications of cranes.

Stationary cranes are usually of the derrick type, as shown in *Fig.* 3, and are steam or electrically driven. They range in capacity up to 20 tons with jibs up to 120 ft.; a useful size is a 5-ton crane with an 80-ft. jib, giving a practical maximum working radius of about 75 ft. with reduced load. Official regulations now require the provision of an indicator to show whether the safe load at any radius of jib is being exceeded. When the structure is of great height it is necessary to erect the crane on towers so that it shall command the whole of the work at any elevation. Three towers are necessary for a single crane, but if two cranes are erected adjacent to each other economy in tower construction is obtained by providing five towers without suffering much reduction in covering area. The towers are usually built up from 5-in. by 2½-in. or similar rough-sawn timbers

FIG. 3.—DERRICK CRANE ON STEEL TOWERS.

bolted together and well braced by diagonal timbers, but they can also be constructed in steel as illustrated in *Fig.* 3, in which case wire tie-ropes may be used for bracing. In *Fig.* 4 a derrick crane on a bridge contract is carried on a timber piled trestle driven into the river bed.

The counterweights for derrick cranes are provided by bricks, blocks of stone, iron weights, pig iron, old pile heads (as in *Fig.* 4), or similar heavy material. With towers the counterweights are provided at the foot of the tower, and the weight of the latter also assists in providing counterweight. At the same time

Fig. 4.—Derrick Crane on Piled Trestles.

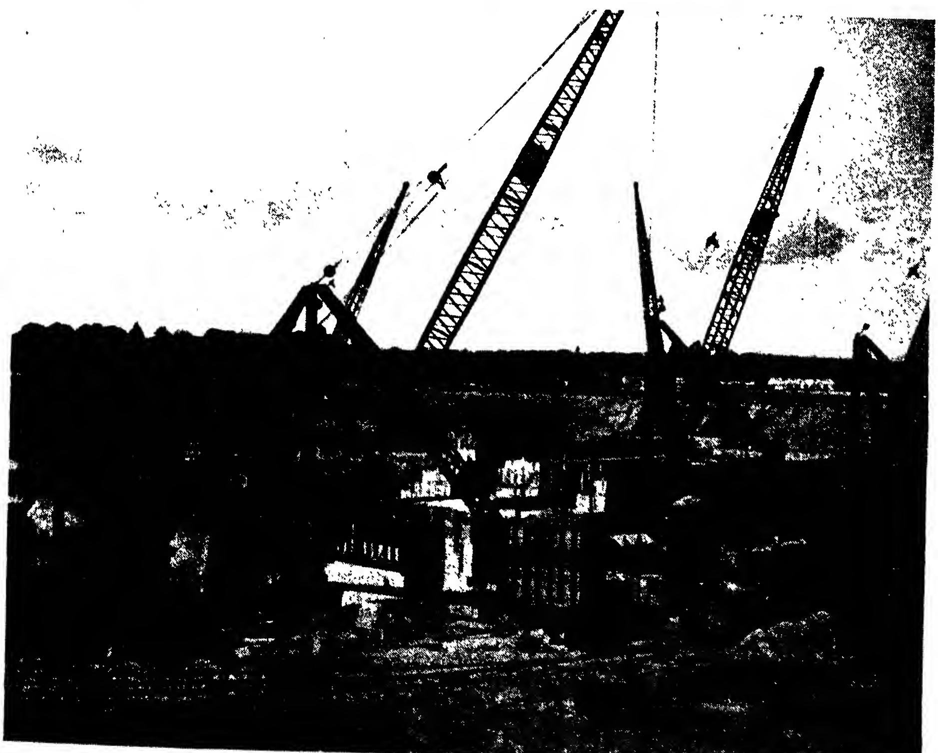


Fig. 5.—Cranes Operating at Ground Level.

additional effective counterweight is required to offset the overturning effect due to wind on an elevated crane and its substructure. The amount of counterweight provided should be the full amount specified by the makers. When these weights are placed in the tower, bricks, gravel or sand are suitable materials, since they are easily packed in between the bracing. If loose material like sand is employed, a timber cribbing as in *Fig.* 3, must be formed to contain it. The

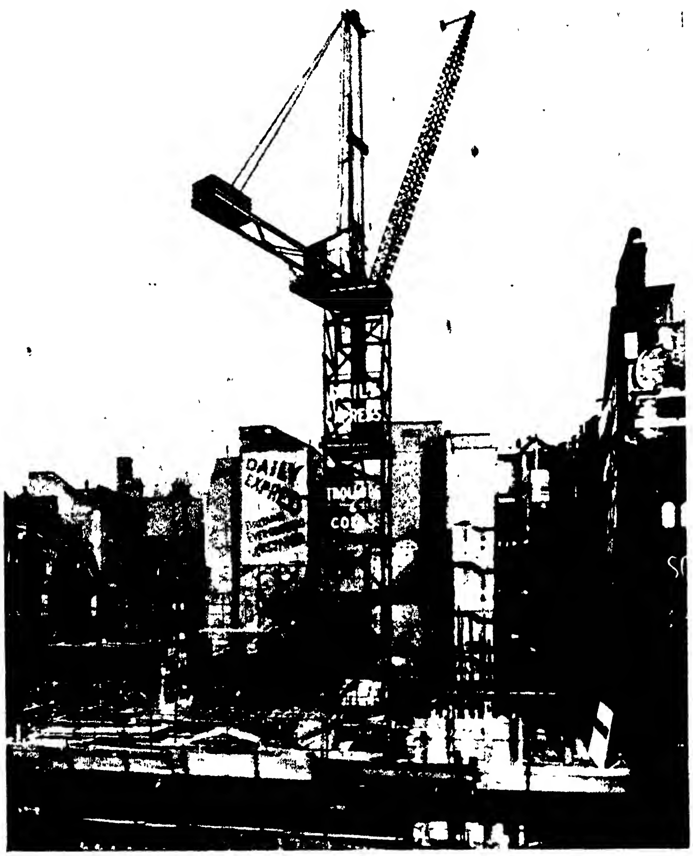


FIG. 6.—TOWER CRANE.

counterweight for the group of cranes illustrated in *Fig.* 5 operating at ground level, is also provided by ballast contained in timber cribs.

If blocks of stone, old pile heads, or similar single pieces of great weight are used, it is necessary to provide lifting tackle to hoist them to the platform. Crane erection, which is a specialist operation that whenever possible should be entrusted to experienced men, is controlled by official regulations given in the Building Regulations issued in accordance with the Factory and Workshop Acts, 1901 to 1929. Derrick crane installations are also dealt with in British Standard Specification No. 327.

The erection of a crane usually requires a ganger and four or five men. To erect a 3-ton hand-winch operated crane complete with counterweight loading requires about 130 man-hours if a mast is used; to erect a 5-ton power crane would take 190 to 250 man-hours, depending on whether a hand-crane or a mast is used.

A class of stationary crane that is becoming increasingly popular for the erection of buildings on city sites is the single-tower type (*Figs.* 6 and 7) in which one crane in the centre of the site covers the whole work. The tower is

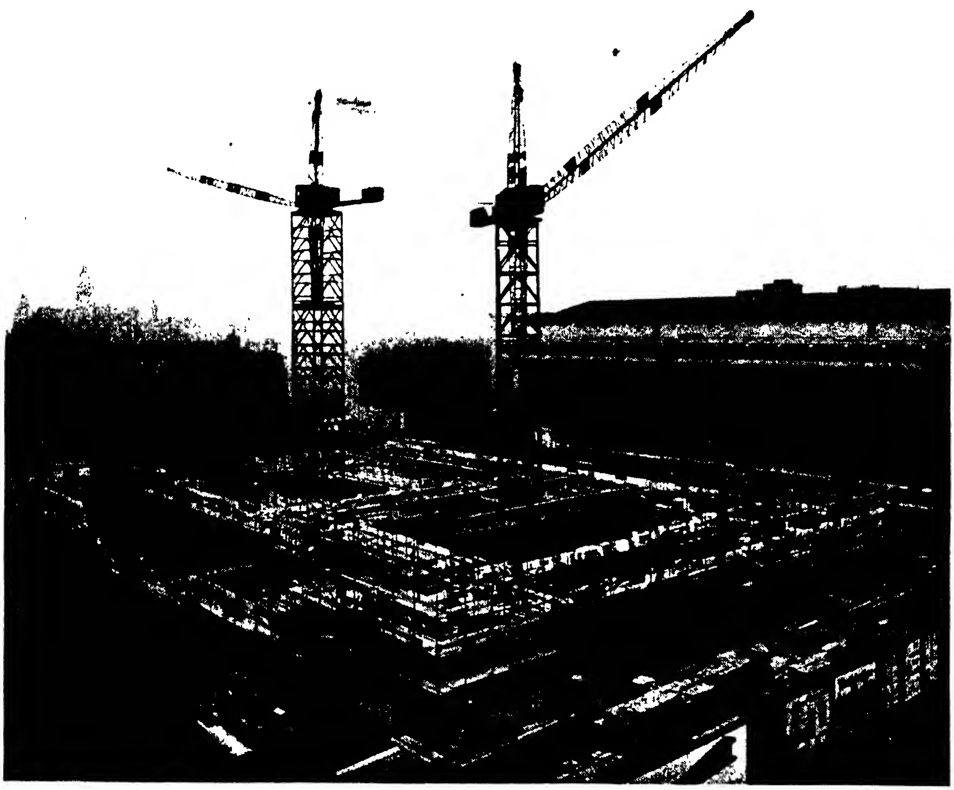


FIG. 7.—TOWER CRANES.

of steel and the counterweight is on the balance arm at the head of the tower. These cranes are expensive to install, as heavy foundation work is required to provide anchorage against wind and racking loads.

Travelling cranes, steam or electrically driven, may run on rails or on caterpillar tracks, and may work at ground level or on temporary timber gantries. The construction of the latter is discussed in the present chapter. Travelling steam cranes may have capacities up to 5 tons; a 2-ton crane is serviceable for most reinforced concrete jobs on which a crane can be used for handling materials and pre-cast pieces.

Fig. 8 shows a crane of this type unloading bags of cement from barges alongside a wharf. The bags are packed into jubilee wagon skips, the latter being lifted by the crane, which then travels up to the end of the timber gantry and lowers the loaded skip on to the bogie waiting to receive it. Two tracks are

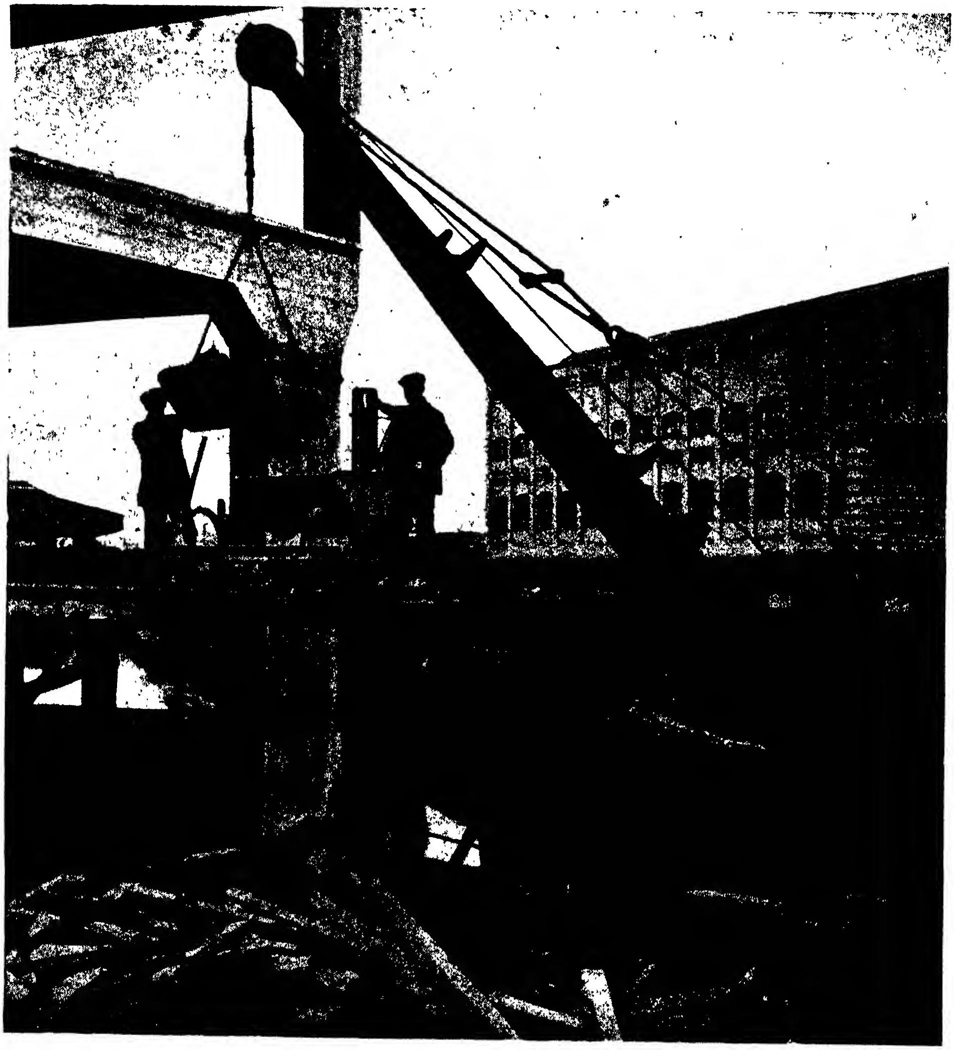


FIG. 8.—TRAVELLING CRANE.

provided on the staging and two or more wagons, so that while one wagon is being wheeled to the cement store the crane can transfer the body of the other to the barge for refilling.

Derrick cranes can also be used as travelling cranes by mounting the counterweight platform on a bogie running on a rail-track ; the platform carrying the cab, jib and mast will be similarly mounted on a bogie running on a parallel track as illustrated in *Fig.* 9, where the crane is shown working in a pile-yard.

In compliance with the Building Regulations crane hooks must be fitted with a safety catch (*Fig.* 10) which is held in position by a spring that prevents accidental release of the load while being hoisted or lowered, and is designed to admit the ring of the chain sling or the handle of a skip. Spring hooks designed for a working load of 1 ton usually have an opening that will admit an $1\frac{1}{4}$-in. diameter rod; a $\frac{1}{2}$-ton hook will admit a 1-in. diameter rod; and a 3-ton hook a 2-in. diameter rod.

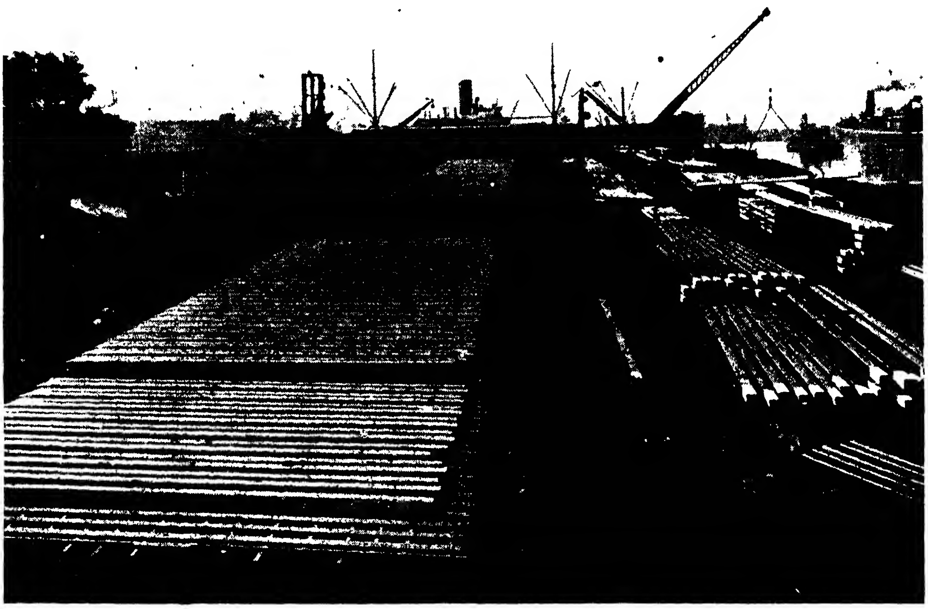


FIG. 9.—TRAVELLING CRANE IN PILE-YARD.

Although cranes or special hoists may be installed for handling materials and concrete, there are few contracts on which useful work cannot be found for hand-operated or low-power lifting tackle. If a platform elevator running in a partly enclosed well is used for hoisting concrete, the restricted space does not usually allow bars and timber to be lifted. For this purpose a simple block and tackle is usually sufficient as the loads do not generally exceed a few hundredweights. This is also a useful device even when a crane is installed, and saves delay caused by waiting for the crane to be free. The pulley-block can be hung from a tripod or cat-head as circumstances permit and for moderate lifts can be operated by hand. A suitable and simple type of pulley for this purpose is a gin block (*Fig.* 11) consisting of a light wrought-iron frame in which runs a grooved cast-iron pulley.

FIG. 10.—SPRING CRANE HOOK.

FIG. 11.—GIN BLOCK.

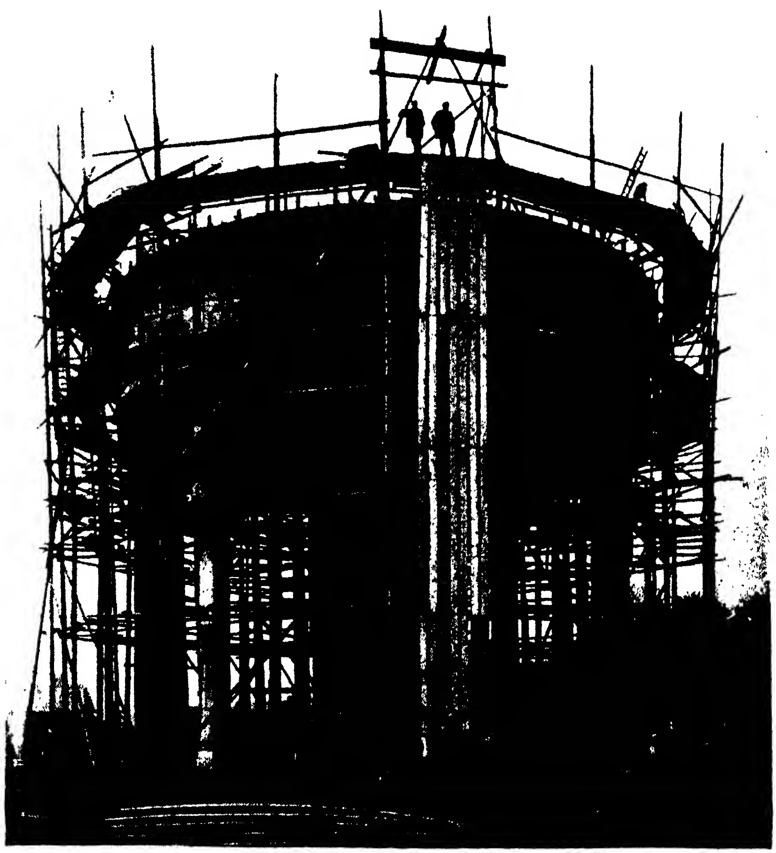


FIG. 12.—BOARDED FACE FOR LIGHT HOIST.

For structures approaching or exceeding 30 ft. high a small power-driven winch is more serviceable. It is usually necessary to provide a boarded face (*Fig.* 12) against which the bars or other material slide while being lifted without fouling the scaffolding or other work. The chain sling by which the load is suspended may consist of single, double, or triple chains depending on the nature of the load. For lifting baulks of timber or similar material, it is necessary to provide a single chain fitted with a hook at the lower end and a ring at the upper end or with a ring at either end, in which case one ring is designed to pass through the other. A barrow would require a treble-chain sling with hooks at the lower end. The maximum working load imposed on a single chain should not exceed 15 cwt. if the chain links are forged from $\frac{3}{8}$-inch diameter metal or 30 cwt. if from $\frac{1}{2}$-in. diameter metal. Heavy chains, say from 1-in. metal, can take up to 6 tons.

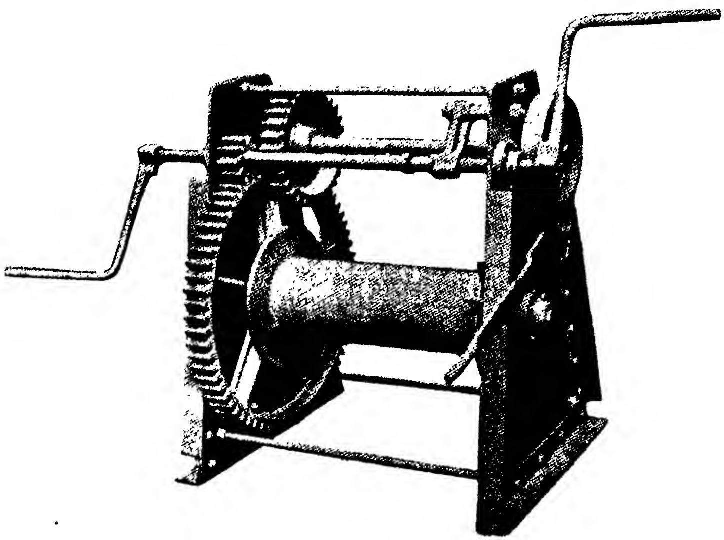


FIG. 13.—CRAB WINCH.

For heavier loads which cannot be lifted by direct pull on a rope passing over a snatch-block or gin-wheel, a geared winch is required. An illustration of a geared hand-winch for use with wire rope is shown on *Fig.* 13. A crab winch of this kind is commonly used in excavating and can be operated by two men ; it is provided with a hand-brake capable of sustaining the full working load. The diameter of the rope barrel varies with the size of the rope and the load to be lifted. A winch to lift $\frac{1}{2}$ ton, using a 1-in. circumference wire rope, would have a 6-in. diameter barrel ; a 1-ton winch with $1\frac{1}{2}$-in. rope, an 8-in. barrel ; and a winch to lift 5 tons on a 3-in. rope would have a 12-in. barrel.

Power hoists may be either belt-driven or may be constructed for direct drive from a petrol engine or electric motor. The belt-driven friction hoist illustrated in *Fig.* 14, which is made in various patterns to lift loads from 3 to 15 cwt., has a pulley-shaft that revolves continuously. When the hoisting barrel is not in operation, the drum is held against the brake by the action of the weight on the operating lever. When the lever is fully raised, the cast-iron grooved friction pulleys engage and rotate the drum and lift the load. With the lever

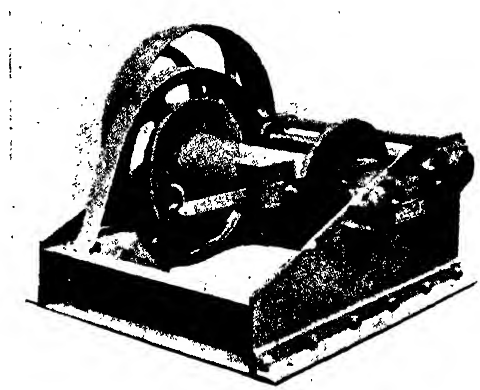


FIG. 14.—BELT-DRIVEN FRICTION HOIST.

in a midway position, the drum is free of both the brake and the pulleys and the load is free to descend. A cord can be attached to the lever so that the hoist can be operated from a distance.

A friction-hoist operated by a hopper-cooled petrol engine is shown in

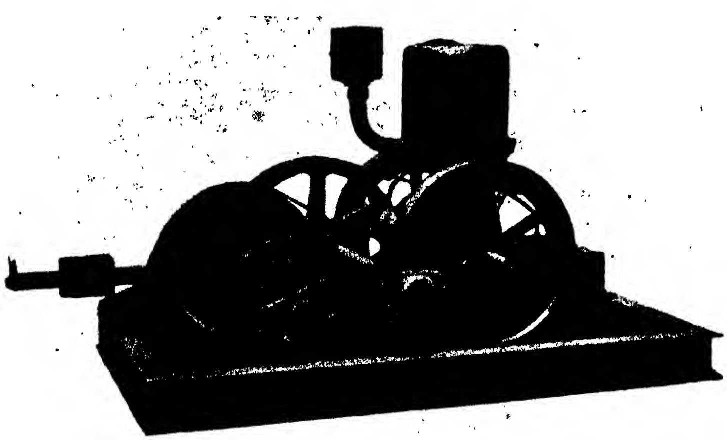


FIG. 15.—POWER DRIVEN FRICTION HOIST.

Fig. 15. The single-cylinder engine, which is mounted on the same base as the hoist barrel, is operated as described for a belt driven hoist, the working load for such a winch with a 4-h.p. engine being ¼ ton, and the hoisting speed 120 ft. per minute. For lifting ½ ton, a 9-h.p. engine is necessary. If in place of a friction drive direct gearing through a friction clutch is provided between the hoist barrel and the engine, the working load of engines of similar power is increased by 50 per cent., the cost of the winch being increased by 33⅓ to 50 per cent. For greater loads twin-cylinder engines are necessary and can be obtained in sizes from 10 h.p. for a working load of 1 ton up to 20 h.p. for 2½ tons. To allow the engine to run continuously for long periods a fan-cooled radiator is fitted on these machines. When electrical power is available, power winches fitted with electric motors can be used in place of the petrol engines.

Precautions should be taken to ensure that wire ropes for hoists are not overloaded, and these should be inspected at intervals.

The working load on new flexible steel wire ropes made up of six strands of 24 wires each should not exceed the following:

Circumference.	Working Load.
1 in.	⅓ to ½ ton
1½ in.	¾ to 1 ,,
2 in.	1½ to 2 tons
2½ in.	2 to 3¾ ,,
3 in.	2¾ to 4¾ ,,
3½ in.	3¾ to 6½ ,,

The lower value should be used if the working conditions involve fluctuating or suddenly applied loads, while the higher figure, giving a factor of safety of six on the ultimate breaking load, can be used if the load on the rope is steady. Alternatively, the breaking strength in tons of new wire ropes of the lowest grade of wire used for winding ropes can be determined from the expression 3·3 (circumference in inches).[2] The working load would be taken as one-sixth to one-tenth of this value.

The working load on a wire rope of unknown strength but apparently sound can be calculated from the expression $1{\cdot}5D^2$ tons, where D is the diameter in inches. Thus, if a wire rope is available having a girth or circumference of 2½ in., the diameter being therefore $\frac{2{\cdot}5}{3{\cdot}14} = 0{\cdot}8$ in., the safe working load would be $1{\cdot}5 \times 0{\cdot}8^2 = 0{\cdot}96$ ton, compared with 2 tons for a new rope.

The diameter of the pulley around which a wire rope is turned should not be less than six times the circumference of the rope.

Pulley-blocks suitable for wire ropes vary from a single snatch-block of 6 in. diameter capable of carrying a working load of ½ ton up to a two or three-sheave block of 23 in. diameter for a working load of 30 tons. Makers' catalogues usually give full details of intermediate sizes and their lifting capacity. These blocks are usually made in steel or wrought iron, except the pulleys which are of cast iron, turned and bored. The smaller blocks are fitted with a swivelling hook, but blocks designed for loads of 10 tons and upwards are usually fitted with a swivel ring. Most blocks are tested to a 50 per cent. overload.

The safe load in hundredweights on a new hemp rope is numerically equal to the square of the circumference. About half this value should be used for a sound old rope. It is usual to add one-third to the load to be lifted to allow for friction on the pulleys.

Pulley-blocks for use with hemp ropes are similar in construction to those for wire ropes and may vary from 2½-in. diameter pulleys suitable for ⅜-in. diameter ropes tested to take a load of 2¾ cwt. for single or double sheaves or 4½ cwt. for three sheaves, up to 10-in. diameter pulleys for 2-in. diameter ropes tested to 150 cwt. for single or double sheaves and 220 cwt. for three sheaves. A useful intermediate size is a 6-in. pulley for a 1-in. diameter rope with a test load of 47 cwt. for single and double sheaves and 68 cwt. for three sheaves. In selecting a block, the test load should be about 50 per cent. higher than the working load.

Chain blocks are also used for hoisting purposes but are not generally suitable on reinforced concrete building contracts. For loads less than 1 ton ungeared blocks are often used, but for heavier loads worm-geared blocks are more common.

Timber Staging.

Timber staging for temporary crane or runway tracks or for supporting pile drivers, arch centering, or similar use are generally made up from 12-in. by 12-in. or 10-in. by 10-in. horizontal and vertical timbers, bolted or spiked together, and well braced diagonally. The verticals may be driven into the ground as piles when

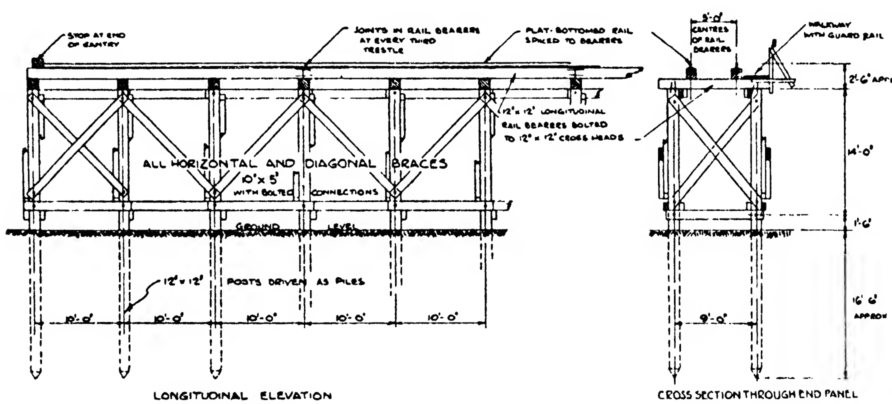


FIG. 16.—TEMPORARY TIMBER STAGING FOR TRAVELLING CRANE.

the bearing power of the ground is low or when the staging is over water; when erected over good ground they may rest on a ground frame. Calculations are not usually made to investigate the loads and stresses on various parts of the staging; experience establishes that for normal purposes the verticals should be from 10 to 15 ft. apart and should be braced in both directions at the top and bottom and at intermediate points not more than 10 ft. apart. Stages only one bay wide should be double-braced laterally with diagonal timbers, but elsewhere single half-timber braces in alternate directions are sufficient. Splices in horizontal timbers should be made over the heads of vertical timbers, while splices in the latter should be cleated on all four sides. Horizontal timbers should be provided directly under the rails as shown in the example on *Fig.* 16, which illustrates a temporary timber gantry for a travelling crane on standard-gauge track.

A similar light gantry to carry a narrow gauge track is illustrated in *Fig.* 17. In this case the rail-track is carried on transverse sleepers spanning between longitudinal timbers supported on a double cross-braced piled frame. Scaffold planks laid on the sleepers provide a walkway, a guard-rail being provided by a rope attached to the projecting ends of the main vertical members.

An alternative construction for a light gantry carrying a double line of jubilee track has already been illustrated in *Fig.* 8.

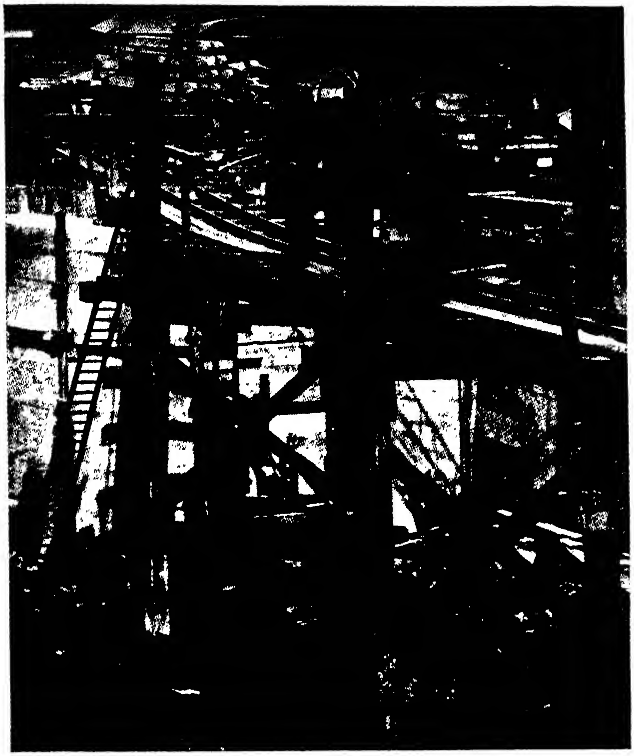


FIG. 17.—TIMBER GANTRY FOR NARROW-GAUGE TRACK.

When a gantry is to be designed for a specific purpose, the following data may be used as a guide in making the calculations. The wheel loads of cranes under various working conditions are usually specified by the makers, while the axle weights of loaded lorries and railway wagons would not in general exceed those given in *Table* I.

TABLE I.

MAXIMUM AXLE WEIGHTS OF LOADED VEHICLES.

Description of vehicle	Front axle	Centre axle	Back axle
Two-axle motor lorry	4 tons	—	5½ tons
Three-axle motor lorry	4 tons	7½ tons	7½ tons
5-ton steam wagon	4 tons	—	8 tons
20-ton railway wagon	15 tons	—	15 tons

To these live loads must be added an allowance of, say, 25 per cent. for the impact of the moving vehicles. Normally these loads will act as point loads, but it is admissible to consider that the rail and bearer will distribute the load over a length of four to six times the combined depth of the rail and bearer. The railbearers and cross-beams would be designed as simply supported beams unless they are constructed in such a manner that continuity over two or more spans is assured. The weight of the track, timbers, and other permanent or incidental loads should be combined with the worst disposition of the rolling loads in the calculation for all beams and posts. If the gantry consists of a single bay in cross-section, as in *Fig.* 16, it may be advisable to investigate the effect on the load on the posts of the wind on the loaded structure. The weights of seasoned timber can be taken from *Table* II which also gives the relative resistances of various classes of timber taking pitch-pine as unity.

TABLE II.

PROPERTIES OF TIMBERS.

Description	Weight (lb. per cubic foot)	Relative resistance values in temporary gantry or similar constructions	
		In Bending Transversely to Grain	In Direct Compression along Grain
Spruce	30 to 35	$\frac{2}{3}$	1
Yellow or red pine	30 to 35	$\frac{1}{3}$	$\frac{2}{3}$
Larch	35 to 40	1	1
Elm	40 to 45	1	1
Pitch-pine	45 to 50	1	1
Ash	45 to 50	$1\frac{1}{3}$	$1\frac{1}{3}$
Beech	45 to 50	$1\frac{1}{3}$	1
Teak	45 to 50	$1\frac{1}{3}$	$1\frac{1}{3}$
Oak	55	$1\frac{1}{2}$	$1\frac{1}{2}$
Greenheart	70	2	$2\frac{2}{3}$

The moment of resistance of a timber beam can be expressed as $M = \frac{bd^2}{100}$ ft.-tons where b and d are the breadth and depth of the section in inches. This expression assumes a safe stress of 1,680 lb. per square inch in compression or tension due to bending, which in temporary work applies to seasoned pitch-pine, larch, or elm. For other timbers the resistance should be multiplied by the appropriate factor taken from *Table* II. If the load is uniformly distributed throughout the length of the beam, the total safe load that can be carried by a pitch-pine (or equivalent timber) beam of span L feet, simply supported, is $W = \frac{bd^2}{12{\cdot}5L}$ tons. *Table* III, which is based upon this formula, gives the total safe distributed load on certain common sections over usual spans and can be used directly if all the loadings are reduced to equivalent uniformly distributed loads. The values given in *Table* III or calculated from the expression for W can be modified by using *Table* II if other timbers are used.

TABLE III.

LOADS ON TIMBER BEAMS.

(Total Safe Uniformly Distributed Loads in Tons on Pitch-pine Beams.)

Span (ft.)	Size of beam [depth × breadth (in.)]					
	8 × 4	8 × 8	10 × 5	10 × 10	12 × 6	12 × 12
10	2·0	4·1	4·0	8·0	6·9	13·8
12·5	1·6	3·3	3·2	6·4	5·5	11·0
15	1·3	2·7	2·7	5·3	4·6	9·2
18	1·1	2·3	2·2	4·4	3·8	7·7
20	1·0	2·0	2·0	4·0	3·5	6·9

The safe load carried on posts or struts in temporary timber stagings can be computed from $V = \frac{3}{8}bdF$ tons, where b and d are the minimum and maximum cross-sectional dimensions in inches, and F is a reduction factor (not exceeding unity) depending on the unbraced height of the post ($= H$ in.), such that $F = 1{\cdot}5 - \dfrac{H}{30b}$. Values of F in accordance with this expression, which assumes that the ends of the post are restrained in position but not necessarily in direction, are given in *Table* IV.

TABLE IV.

REDUCTION FACTORS FOR LONG POSTS OR STRUTS.

$\dfrac{\text{Unbraced height}}{\text{Least dimension}}$	15 or less	20	25	30	45 or greater
Value of F . . .	1·0	0·833	0·667	0·50	zero

Values of V for certain common posts or struts (F = unity) are given in *Table* V in accordance with the previous formula, which is based on a safe working stress of 840 lb. per square inch in direct compression in the direction of the grain. This gives a factor of safety of approximately 8 for timbers such as seasoned pitch-pine, elm, spruce, larch, and beech, and allows a margin for accidental eccentricity. For other timbers, the calculated or tabulated loads should be multiplied by the appropriate factors given in *Table* II.

TABLE V.

LOADS ON TIMBER POSTS.

(Safe Concentric Loads on Pitch-pine Posts in Tons.)

Unbraced length of post	Size of post (in.)				
	6 × 6	8 × 8	10 × 10	12 × 12	14 × 14
5 ft.	13·5	24·0	37·5	54·0	73·5
10 ,, ...	11·3	24·0	37·5	54·0	73·5
15 ,, ...	6·8	18·0	33·7	54·0	73·5
20 ,, ...	—	12·0	26·2	45·0	68·3
25 ,, ...	—	6·0	18·8	36·0	58·0

If the posts are driven as piles the resistance to settlement must be sufficient to allow a margin of safety on the maximum load to which the post can be subjected in service. For temporary work the safe load can be approximately calculated from the Dutch formula in the form

$$W = \frac{whn}{k(1 + R)}$$

in which W = safe load on post (tons);
w = weight of drop hammer (tons);
h = drop of hammer (in.);

$$R = \frac{\text{weight of pile}}{\text{weight of hammer}};$$

n = number of blows per final inch of penetration; and
k = a factor having the following values:

	In firm soil.	In resilient soil.
With dolly	6	8
Without dolly	8	10

The value of k can be decreased by 10 per cent. if a single-acting steam-hammer is used and should be increased by 50 per cent. if the bulk of the load on the post is live-load.

The coefficients do not apply to reinforced concrete piles. The value of H in the expression for F can be taken as two-thirds the length of the pile embedded in the ground plus the length from ground level to the first effective brace.

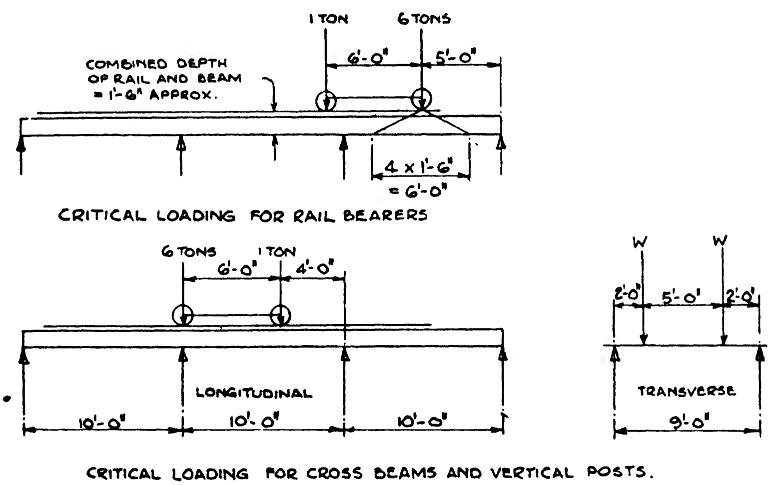


FIG. 18.—LOADINGS ON CRANE GANTRY.

The following calculations show the essential steps in designing the gantry illustrated in *Fig.* 16, and demonstrate the use of the foregoing formulæ and tables. The gantry will be designed to carry a light crane on a standard-gauge track, the maximum wheel load being 6 tons when the loaded crane is travelling along the gantry. The critical loading on the rail bearers occurs when this wheel load is at midspan. Assume that the wheel-base of the crane is such that the next wheel bears on the adjacent span as shown in *Fig.* 18. Allowing 25 per cent. increase for impact the design load is $7\frac{1}{2}$ tons and assuming that dispersion through the rail and beam distributes this load over, say, 4×1 ft. 6 in. $= 6$ ft. length of beam the live-load bending moment on a freely-supported beam is

$$\frac{7\frac{1}{2}}{2}\left(\frac{10}{2} - 1{\cdot}5\right) = 13{\cdot}2 \text{ ft.-tons.}$$

	lb. per foot run.
Weight of rail, say 60 lb. per yard	= 20
Weight of 12-in. by 12-in. timber	= 50
Incidental loading, say,	= 90
Total uniform load	= 160

$$\text{Bending moment on free beam} = \frac{160 \times 10^2}{8 \times 2240} = 0{\cdot}9 \text{ ft.-ton.}$$

$$\text{Total free bending moment} = 13{\cdot}2 + 0{\cdot}9 = 14{\cdot}1 \text{ ft.-tons.}$$

Since the railbearers are continuous over three spans, the bending moment might be reduced to $0{\cdot}8 \times 14{\cdot}1 = 11{\cdot}3$ ft.-tons. The resistance moment of a 12-in. by 12-in. pitch-pine beam is $M = \frac{bd^2}{100} = \frac{12 \times 12^2}{100} = 17{\cdot}3$ ft.-tons, which is satisfactory.

An alternative method, adopting *Table* III, would be to convert the actual load into an equivalent uniformly-distributed load, thus: The actual load produces a bending moment of $11{\cdot}3$ ft.-tons and the uniformly distributed load on a 10 ft. "free" span to give this moment would be $\frac{11{\cdot}3 \times 8}{10} = 9{\cdot}04$ tons. From *Table* III, a 12-in. by 12-in. pitch-pine section spanning 10 ft. can carry a uniformly-distributed load of $13{\cdot}8$ tons.

The loads on the crosshead or transverse beam will be arranged as in *Fig.* 18 to give the maximum moment on the beam and the maximum reaction on the posts. The value of each of the loads W is:

	Tons.
Direct reaction from maximum loaded wheel	= 6·0
Reaction from minimum loaded wheel $= \frac{4}{10} \times 1{\cdot}0$	= 0·4
Total	= 6·4
Add 25 per cent. for impact	= 1·6
Weight of logitudinal beam, etc. $= \frac{160 \times 10}{2240}$	= 0·7
W	= 8·7

$$\text{Bending moment} = 8{\cdot}7 \times 2{\cdot}0 = 17{\cdot}2 \text{ ft.-tons,}$$

or allowing for the weight of the beam, a 12-in. by 12-in. pitch-pine beam having a resistance of $17{\cdot}3$ ft.-tons is suitable.

The maximum load carried by each of the vertical posts driven as a pile is the reaction of $8{\cdot}7$ tons from the crosshead plus the dead load of the crosshead, pile, crossbracing, etc., say, 10 tons total. The safe column load on the 12-in. by 12-in. section provided is found as follows:

The unbraced length of column between the crosshead and the lower tier of braces is 14 ft. (see *Fig.* 16), while from the latter to the pile toe the distance is 18 ft. If we allow for the supporting effect of the earth on the 16-ft. 6-in. embedded length, the effective unbraced length is $(\frac{2}{3} \times 16 \cdot 5) + 1 \cdot 5 = 12 \cdot 5$ ft. The maximum ratio of $\frac{H}{b}$ is therefore $\frac{14 \text{ ft.}}{12 \text{ in.}} = 14$; therefore $F = 1$ (from *Table* IV). Hence the safe working load $(= \frac{3}{8}bdF) = \frac{3}{8} \times 12 \times 12 \times 1 \cdot 0 = 54$ tons (or this value could be read directly from *Table* V). This safe load is amply in excess of the actual working load.

If we assume that the piles will be driven with a 1-ton winch-operated drop-hammer falling 4 ft., the required set is calculated as follows:

$$\text{Weight of 32-ft. pile 12 in. square} = P = \frac{32 \times 50}{2{,}240} = \frac{3}{4} \text{ ton.}$$

$$\text{Ratio of } \frac{\text{pile weight}}{\text{hammer weight}} = R = \frac{P}{w} = \frac{\frac{3}{4}}{1} = 0 \cdot 75.$$

Assuming a value of $k = 8$ for a drop-hammer, fairly firm soil, and a pile fitted with a helmet, the value of n (= number of blows per final inch of penetration) for a safe load of 10 tons (which includes an impact allowance) will be given by substitution in the Dutch formula.

$$\text{Here } 10 = \frac{1 \times 48n}{8(1 + \frac{3}{4})}$$

from which $n = 3$ blows per inch. This is very easy driving and by decreasing the set to 5 or 6 blows per final inch of penetration additional security against side racking effects due to wind or other causes is obtained.

Scaffolding.

Scaffolding to provide temporary working stages above ground level may be either timber or tubular steel. Due to the necessity of having a firm and secure scaffold, all scaffold erection, whether in timber or steel should be entrusted to experienced erectors. Official requirements applicable to scaffold construction are given in Building Regulations, 1926 to 1931, issued under the Factory and Workshop Acts, 1901 to 1929.

A timber scaffold of conventional design is illustrated in *Fig.* 19. Such scaffolds usually consist of a single or double row of round poles, say 30 ft. long, spaced at intervals of 8 to 10 ft., the lower ends being secured by embedding in the ground or in tubs of earth. Horizontal poles are lashed to the verticals at intervals of about 6 ft., and on the horizontals are supported 4-in. by 4-in. putlogs 6 ft. long. These in turn carry the working platform of 9-in. boarding, $1\frac{1}{4}$ or $1\frac{1}{2}$ in. in thickness and in 12-ft. lengths. To prevent splitting the ends of the boards are chamfered and bound with metal. Economy in the provision of boards and putlogs is secured by removing them from the lower tiers when they can be spared and using them on the upper tiers. The corners of a scaffold require bracing by diagonal poles. These features are clearly indicated in *Fig.* 19, which illustrates a typical example of timber scaffolding construction at the corner of a building.

Intersections of scaffold poles are secured with either rope, chain or wire lashings, the latter being preferable since once they are tightened they remain taut, whereas ropes may work loose under alternate wetting and drying. Scaffold poles are usually spruce and are sometimes delivered with the bark of the tree still on; if so, the bark must be removed. Stripping the bark from a 30-ft.

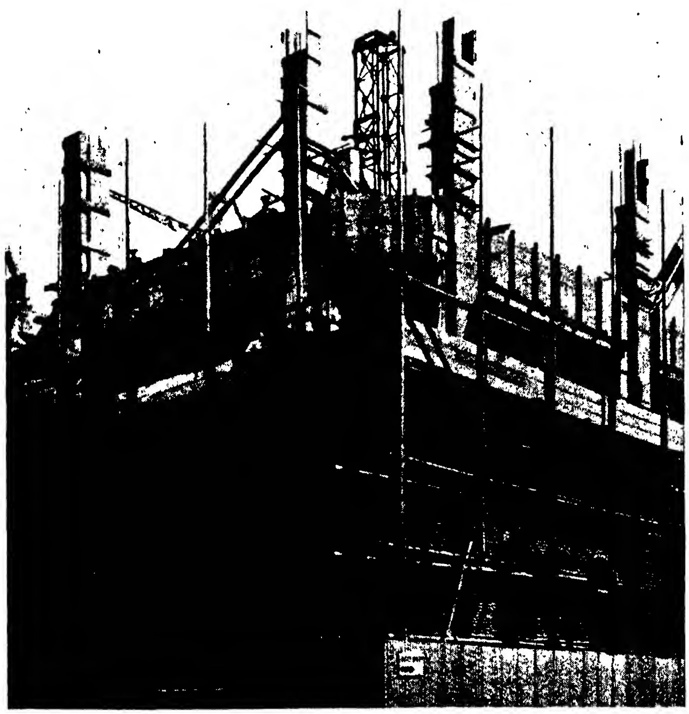


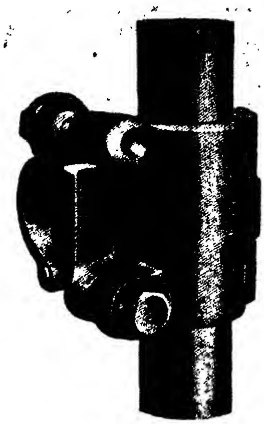
FIG. 19.—TIMBER SCAFFOLDING.

pole may occupy two men about 20 minutes. If ordinary scaffold poles are not available, suitable scaffolds can be made from unwrought sawn timbers.

Tubular steel scaffolding, whether hired or purchased, is economical and adaptable in the construction of reinforced concrete structures. On a number of contracts the hiring charges together with the cost of erection, maintenance and dismantling have been found to be less than the cost of buying only the timber for an ordinary scaffold.

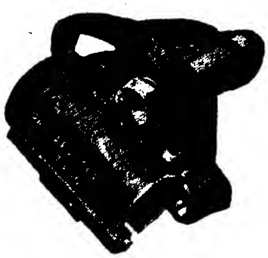
Factors contributing to the greater economy of steel scaffolding include speedier erection and, excepting a small depreciation, the permanence of the equipment. Maintenance entails only occasional oiling and painting. The use of shorter members than in timber scaffolding results in lower transport costs and also less obstruction to pedestrians and workmen.

Steel scaffolding follows the general lines of timber scaffolding as regards the arrangement of the vertical and horizontal members and the use of scaffold

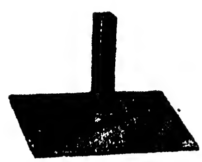


(*a*) Double Coupler.

(*b*) Putlog Coupler.

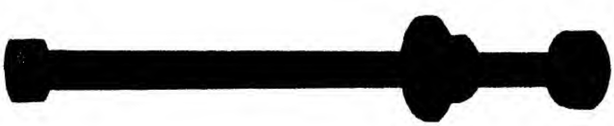


(*c*) Swivel Coupler.



(*d*) Base Plate.

(*e*) Split Joint Pin.



(*f*) Reveal Pin.

FIG. 20.—TUBULAR STEEL SCAFFOLDING.

planks. The greatest departure is the method of connecting by couplers two steel members whether at right-angles to each other or in the same vertical line. The couplings are of various designs depending upon their purpose and upon the manufacturer. A double-coupler as illustrated in *Fig.* 20(*a*) is used to connect the main horizontal members (or ledgers) to the vertical members (standards). The main danger in steel scaffolding being the tendency for the ledger to slip down the standard, particular attention has to be paid to the design of a double coupler to prevent this. The design illustrated, while not symmetrical nor interchangeable with regard to the vertical and horizontal members, attains this security against sliding by the double grip on the standard afforded by the two bolts and has the added advantage of being in a single unit, there being no loose pieces except the bolts. An alternative design of coupling is shown in the illustration of assembled steel scaffolding in *Fig.* 21.

A transom or putlog coupler [*Fig.* 20(*b*)] is used to connect subsidiary cross members (putlogs) at right-angles to the ledgers.

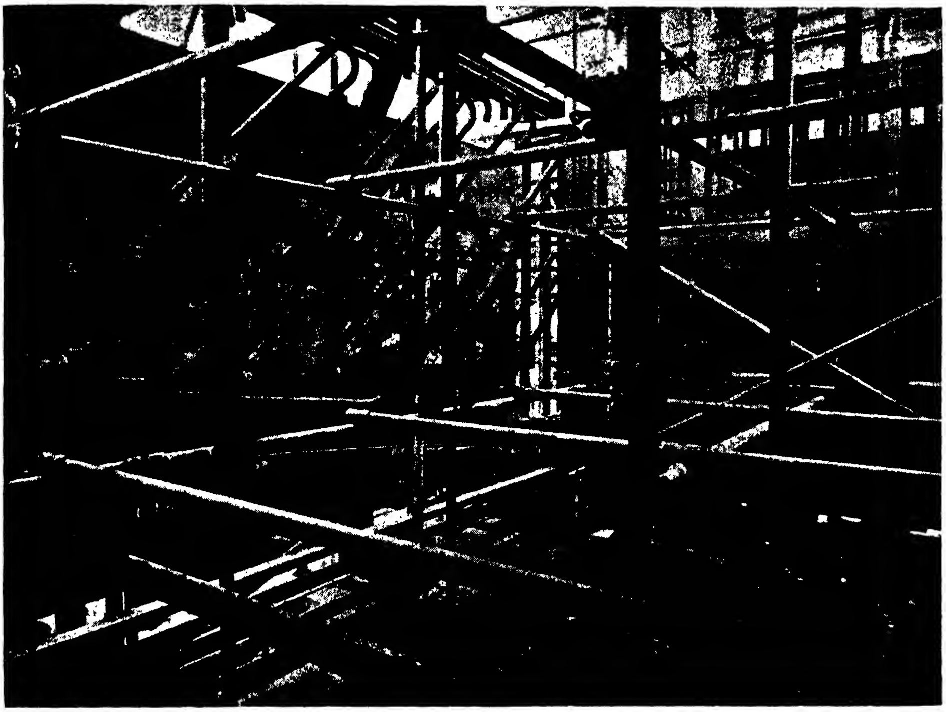


FIG. 21.—ASSEMBLED STEEL SCAFFOLDING.

Swivel-couplers [*Fig.* 20(*c*)] are used with members not at right-angles, but are more expensive than ordinary couplers.

The tubes are commonly No. 6 gauge drawn or welded steam tubing (0·192 in. thick) having an internal diameter of 1½ in. Tube lengths vary from 6 ft. to 22 ft.

At the foot of all standards a base plate is fitted which may either be circular as in *Fig.* 21 or 6 in. to 9 in. square as in *Fig.* 20(*d*) with a projecting pin 6 in. long.

Two tubes may be joined in line by using an expanding joint pin consisting of a bar of steel divided laterally on each side of a central collar as in *Fig.* 20(*e*). The two limbs are placed in the ends of the tube, and are expanded by the external nut to give a tight fit. The position of vertical joints should be staggered to avoid a number of joints occurring at any one level and producing a plane of weakness in the scaffold.

The stability of external scaffolding can be increased by using reveal pins; these are small jacks [*Fig.* 20(*f*)] which, when inserted in the end of a short length of tube, allow the latter to be fixed securely in a window opening or other recess in the wall. The fixed tube is then coupled up to the main scaffolding. If the wall face is long and flush, inclined strutting must be provided to aid stability.

A volume or birdcage scaffolding, that is an internal scaffolding, is stabilised by making the ends of the horizontals bear against the containing sides, the ends of the tubes being padded with sacks or cement bags to prevent damage to the wall face.

The working platform on a tubular scaffolding is formed by laying 9-in. scaffold boards across the putlogs or other horizontal members. Steel boards have been used for this purpose but are noisy and slippery.

For transport, the weights of steel scaffolding units are as follows:

1½-in. internal diameter tubing	3½ lb. per foot.
Double-couplers	5 lb. each.
Putlog couplers	3 lb. ,,
Joint pins	5 lb. ,,
Base plates	5 lb. to 8 lb. ,,
9 in. × 1¼ in. scaffold plank, 13 ft. long . .	35 lb. to 40 lb. ,,

All small fixings should be carried in 1-cwt. lots in sacks or in small boxes. For convenience of transport and handling, tubes should not be more than 22 ft. long. For ordinary work lengths of 18, 14, 12, 10 and 6 ft. with a number of short random lengths are most useful. For jobs more than 18 ft. high a rope and wheel for hoisting the units is necessary. When loading a lorry for transporting scaffolding the longest tubes should be placed at the bottom and the planks on top of the tubes.

In a standard builder's wall scaffold, the standards are placed 6 ft. apart longitudinally and 4 ft. transversely, the inner row of standards being placed 1 ft. 6 in. from the wall face as shown in *Fig.* 22. The vertical lifts, or distance between ledgers, is 6 ft., the joints in the standards being staggered and the scaffold strutted or otherwise tied to the building as previously described. At the beginning of the erection the plumbing of the standards is a primary consideration. The base-plates must have a level foundation, packing being inserted where the ground slopes.

A squad of three men adopt the following procedure when erecting a builder's scaffold. The positions of the base-plates are set out on the ground and the base-plates placed in position. On two of the inner lines of base-plates, two men fix standards of different lengths, say 12 ft. and 14 ft., to allow the vertical joints to be staggered. The third man, having attached an inclined strut to a ledger, couples the latter to the two standards. The latter, with the inclined strut bearing on the ground (*Fig.* 22) can now be left standing while the same process is repeated with the corresponding pair of outer standards. Putlogs are then coupled across between the inner and outer rows and the framework, consisting of four standards, two ledgers, two putlogs, one inclined strut, four double-couplers and four putlog couplers, is plumbed. The remainder of the erection follows the sequence of standards, ledgers, and putlogs until the first lift is completed. Ledgers and putlogs are fixed to form the second lift. Joint pins are placed in the top of the standards and tubing added to whatever height is necessary. At the third or fourth lift it is quicker to provide lifting tackle to hoist the material instead of passing it up by hand.

Internal scaffold construction follows similar lines except that the standards should be placed at about 10-ft. centres giving one standard for each 100 sq. ft. of plan area. The use of steel internal scaffolding to support floor shuttering is described in Chapter VIII.

When erecting steel scaffolding, three men should erect an average of 50 sq. yd. of wall scaffolding not exceeding four lifts, that is 24 ft. high, in $2\frac{3}{4}$ hours. Dismantling may take one-quarter to one-third of this time. Similar times would be taken in erecting and dismantling 200 cu. yd. of volume scaffolding. If the latter type of scaffolding is used for strutting floor or arch soffits, these

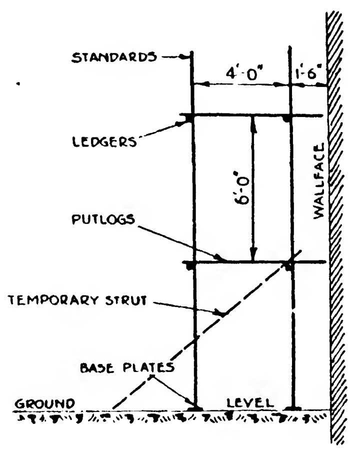


FIG. 22.—BUILDER'S SCAFFOLD (TUBULAR STEEL).

times would be increased by 50 to 100 per cent. owing to the greater number of standards and the introduction of diagonal bracing.

The depreciation of scaffolding equipment due to damage or loss can be minimised by periodical stocktaking. It is desirable, although not always possible, periodically to clean all tubes, couplings, and scaffold planks. The latter represent the major replacement charge as they are often split or sawn and become encrusted with cement.

Most sub-contractors' quotations for the supply and erection of scaffolding are lump-sum prices, but if the period for which the scaffolding will be in use cannot be closely assessed beforehand a quotation may be given for erecting, dismantling, and maintenance for, say, two months, and a weekly or monthly rate of maintenance charged thereafter.

For many reinforced concrete frame structures scaffolds are unnecessary, since supports for platforms and falsework for work at successive heights can be obtained off the completed work as shown in *Figs.* 23 and 24. External

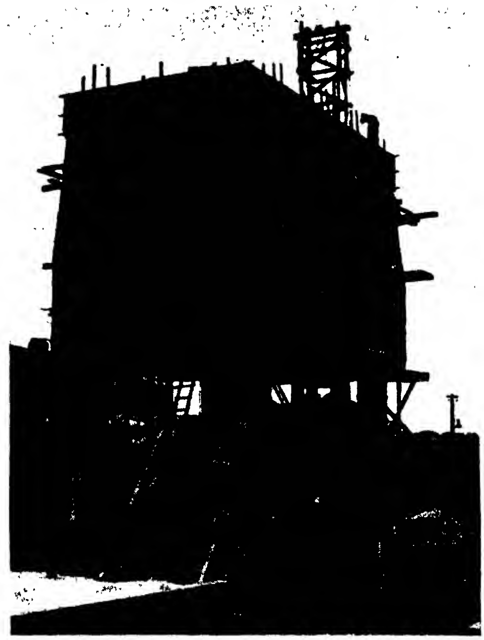


FIG. 23.—BUNKER ERECTION WITHOUT SCAFFOLDING.

FIG. 24.—BUILDING ERECTION WITHOUT SCAFFOLDING.

scaffolding is usually required for structures having concrete panel walls, although for structures like the bunker shown in *Fig.* 23 the necessary support can be obtained by putlogs projecting through holes left in the walls. A working platform of planks can be laid on these putlogs from which also the shuttering can be strutted. It is not necessary to provide a tier of putlogs for each lift of shuttering, since it is possible, as illustrated, to make one tier support at least two lifts. The holes would be made good after the completion of the job, working

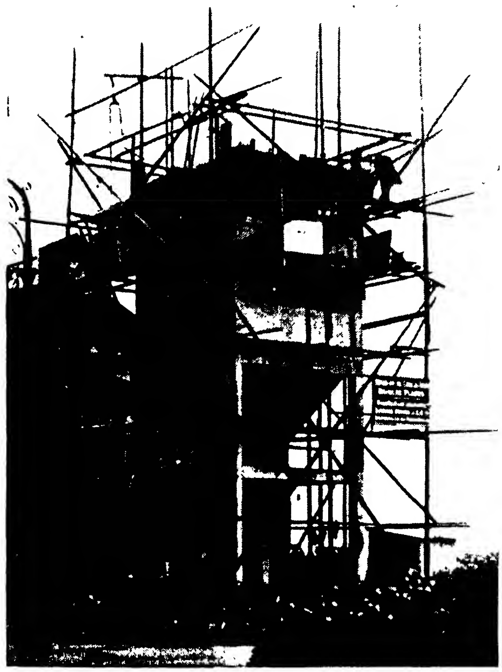


FIG. 25.—BUNKER ERECTION WITH SCAFFOLDING.

from the top row downwards. Owing to the difficulty of making the holes watertight this method is not recommended for impermeable construction. In the case of a building, such as that in *Fig.* 24, each floor forms the working platform for the succeeding floor. A small isolated bunker such as that illustrated on *Fig.* 25 would be erected most conveniently with the aid of an external scaffolding as the amount of scaffolding required is small and the height is moderate. The construction of chimneys with and without scaffolding is considered in a later chapter. The erection of a water tower, as in *Fig.* 12, is facilitated by the use of external scaffolding.

Plant Costs.

The amount that should be charged against a contract for the use of any particular piece of constructional equipment should include all direct and indirect costs involved, including depreciation and similar charges, transport to and from the site, erection, maintenance and dismantling, movement about the site, running costs (including power or fuel, greases, water and other stores), and any temporary stagings, roads, rail tracks and connection of water mains, required for working the plant. The value of such accessories as ropes, boats, temporary timbering, counterweights, and ballast would be assessed and charged against the plant item.

The total cost for all plant used on a given contract bears some relation to the total labour, but it will be understood that the relative proportion varies with the practice of different contractors and to a greater extent with the type of structure. Generally, for an office or warehouse class of building, the plant costs may represent 25 to 35 per cent. of the total labour cost, while for bridges with cofferdam foundations the value may be 50 to 75 per cent.

The amount of depreciation to charge against a given contract, although frequently a major item, is difficult to assess closely. If plant is bought new for a definite job and sold at the conclusion of the work the depreciation would be the full price paid for the plant less any return for salvage value. Generally a piece of plant serves several contracts during its useful life and the net depreciation chargeable against any one item can be based on the contract time for that particular work. For instance, if a concrete mixer is considered to have a useful life of five years, its employment for six months on a job would involve a minimum charge against that job of 10 per cent. of the purchase price. Another method of calculating net depreciation charges is based on the output of the plant. Thus, if a concrete mixer is considered as having a useful life equivalent to an output of 10,000 cu. yd. and a job involves 1,000 cu. yd., the minimum depreciation would be 10 per cent. of the purchase price. Repairs and renewals would be covered by a gross depreciation charge, which would also have to include boiler inspection and similar charges and storage costs during idle periods between contracts.

If the net depreciation is combined with certain other charges there results a gross annual depreciation of 25 to 50 per cent. for a piece of plant that has a useful life of four or five years. For five years' life the net annual depreciation is 20 per cent., to which must be added, say, 5 per cent. for interest on the capital cost of the plant, 10 per cent. for major repairs, and 5 per cent. for insurance and for storage during idle periods, giving a total of 40 per cent. per annum. A good inclusive figure for a petrol-driven mixer would be 5 per cent. per month based on the initial cost.

It is unusual for a piece of plant to be in full use throughout its life. In fact most classes of contractor's equipment may only be in use for seven or eight months in each year, although the average service period for mechanical shovels is about ten months in a year. The economic life and amount of repairs vary with the type of plant considered, and *Table* VI on p. 39, showing average values for various units, assumes that at the end of its economic life the salvage value of the unit is 25 per cent. of its original cost. The annual upkeep charge, expressed as a percentage of the original cost, includes net depreciation, repairs, and 5 to 7 per cent. for financial charges. Repairs may be executed on the site, or for major repairs shop work may be necessary. Site repairs may represent one-quarter to one-third of the total repair bill for concrete mixers and pneumatic drills, and one-third to one-half for pile drivers, pile hammers, pumps, mechanical navvies, and tractors.

TABLE VI.

DEPRECIATION OF CONSTRUCTIONAL EQUIPMENT.

Type of plant	Economic life (years)	Annual depreciation (per cent.)	Repairs (shop and site) (per cent.)	Annual upkeep (per cent.)
Concrete mixers: Petrol	4	19	21	46
" " Electric	6	13	16	35
Hoist towers: Steel	7	11	7	24
Batch boxes	3	25	13	44
Timber bins	2	38	20	64
Steel bins	6	13	12	31
Concrete buckets	3	25	16	47
Concrete carts	2	38	10	54
Distributing chutes	2	38	10	54
Petrol hoists	6	13	7	26
Electric hoists	7	11	6	23
Steam pile drivers	8	10	12	28
Steam pile hammers	7	11	10	27
Concrete pneumatic drills	4	19	28	53
Pneumatic riveters	5	15	12	33
Steam navvies	6	13	13	32
Petrol navvies	4	19	16	41
Electric navvies	7	11	11	28
Pumps: Centrifugal	8	10	10	26
" Reciprocating	6	13	12	31
" Pulsometer	8	10	6	22
Piping	3	25	11	42
Petrol compressor	4	19	15	40
Electric motors	6	13	10	29
Boilers (vertical)	7	11	20	37
Steam rollers	10	8	8	22
Steel shuttering	5	15	8	29
Petrol tractors (wheeled)	6	13	14	33
" " (caterpillar)	5	15	25	46
Rail tracks	8	10	8	24
Jubilee tracks	3	25	15	46
Jubilee wagons	6	13	8	27

Items of plant required only for a single contract of short duration can usually be economically hired on weekly, monthly, or three-monthly terms. In the cost estimate the hiring charges would appear in the place of the gross depreciation, although transport, erection and dismantling, attendance and operation, and similar charges would have to be added. Usually the quotation for hiring will include the transport charge. For example, an air compressor may be let on hire at £7 per week with £5 for carriage each way. To the hiring, transport, and financial charges for the compressor it would be necessary to add the wages of the attendant, and the fuel or power charges, and in the case of an electrically-driven machine the cost of connecting to supply mains. The fuel and oil consumption of a Diesel-driven compressor capable of delivering 140 cu. ft. of free air per minute at 100 lb. per square inch would be approximately 8 gallons of fuel oil and 4 or 5 pints of lubricating oil in a shift of 8 hours. A corresponding petrol-driven machine would consume about 15 gallons of petrol and about 3 pints of lubricating oil in a similar period.

Plant transport charges vary with the nature of the plant as well as with distance. The best rates are obtainable for full wagon loads on a ton-mile basis. Timber framing and loose loading material can be sent by rail at about two-thirds the rate charged for machinery.

The approximate estimated cost of a derrick crane with timber towers in use for six months would be compiled as follows:

	£
Depreciation: net depreciation 20 per cent. per annum on £500	50
Other charges at 20 per cent. per annum	50
Carriage to site	10
Erection	25
Dismantling	15
Carriage from site	10
Timber for towers	20
Ballast: 20 tons use and waste at 7*s.* 6*d.* per ton	8
Fuel: 2½ tons per week at 28*s.* per ton for 20 weeks	70
Water	20
Wages: Craneman, 20 weeks at 70*s.* per week	70
Total	£348

If this crane is employed solely on hoisting concrete the full charge of £348 is charged against the cost of concreting. If the crane's time is equally divided between concrete hoisting and, say, steelwork erection, the charge is allocated in equal parts to these operations.

In the case of items included under the heading of small tools and consumable stores, it is not usually possible to arrive closely at an estimated cost to charge against a particular contract. Quantities and salvage values vary between wide limits, but an on-cost of, say, 4 to 5 per cent. on the net labour charges should be sufficient to allow for miscellaneous equipment and such consumable stores as greases, ropes, odd timbers, tarpaulins, and sackings, but excluding fuel for driving major items of plant. On works of more than a few months' duration the cost of such items would usually be fully charged against the contract. The items of plant that are usually included under the heading of "small tools" are shovels, spades, trowels, steel concrete barrows, wooden navvy barrows, pole ladders, trestles, distemper brushes suitable for grout washing, long-handled tar brushes for applying cement retarding liquids, heads of picks either double-pointed or chisel-and-point type, hickory shafts for use with pick-heads, cast-steel sledge-hammers, galvanised pails, sieves, spanners, etc.

SECTION III.—ARRANGEMENT OF PLANT.

General Principles.

Having decided on the plant to be installed, the efficiency of the subsequent construction is largely influenced by the way in which it is arranged. The first consideration is that the items of plant employed on successive operations should be so disposed that these operations follow naturally and with the minimum of handling of materials. Since a reinforced concrete contract is a temporary factory for making concrete, the machinery for all phases of the manufacture should be installed with the same regard for efficiency as would be bestowed on a permanent factory. The second consideration is that once the major pieces of stationary equipment are erected they should, if practicable, be able to maintain efficient service throughout the contract without being moved from their original positions. Large concrete mixers, hoists, derrick cranes, material bins, access roads, and rail-tracks represent the type of plant of which the positions should be carefully selected at the beginning of the job. Portable mixers, pumps, compressors, bar-bending machines, carpenters' benches, and timber runways are items that can be readily moved from time to time to more suitable positions as the work proceeds.

The position of any piece of plant should be planned with reference to the appropriate materials supply and the position to be occupied in the work by these materials. Where possible independent entrances and exits should be provided for the material wagons or trains. Material dumps should be situated so that the handling and distance from the supply wagons to the dump and from the dump to the operator are reduced to a minimum.

The typical arrangements in *Figs.* 26 to 31 illustrate the application of the foregoing principles to the conditions met with on actual jobs. The special problems encountered in reinforced concrete piling are dealt with in a later chapter.

Small Isolated Structures.—The arrangement of the plant required for constructing isolated structures such as bunkers, water towers, and chimneys is usually a simple matter. For a bunker alongside a permanent railway track on which the material can be brought to the site the arrangement shown in *Fig.* 26(*a*) is suitable. The aggregates and cement are unloaded at one end of the site adjacent to the mixer and hoist. The reinforcement and timber are unloaded at the far end of the site where operations connected with these materials can be conducted without interfering with the mixing and wheeling of the concrete.

The site of the water tower given in *Fig.* 26(*b*) can only be approached by road and only one entrance is available. The surface of the site being soft, it is necessary to lay timber sleeper tracks for the lorries; otherwise the ground would be badly rutted owing to the fact that all vehicles have to follow the same path; the area timbered should be kept to a minimum. The arrangement indicated allows the bulky materials—stone, sand, and steel—to be unloaded directly to the stock heaps. The timber can be unloaded at the end of the sleeper track and taken to stock by hand. The bags of cement would be unloaded at the entrance and taken on wheelbarrows to the cement store. By the same means the concrete materials would be transferred from the heaps to the mixer. Plank runways provide a suitable track for wheelbarrows. The positions of the mixer, offices, water supply, and other essentials are indicated on the plan.

Buildings.—Building sites present a wide range of conditions requiring totally different methods of carrying out the work and a variety of plant arrangements. In *Figs.* 27 and 28 three different problems are represented. *Fig.* 27(*a*) shows an arrangement for the construction of a factory. Modern factories are usually built on open sites with extensive frontages to road or rail or both, and are generally of moderate height. In the example illustrated, road access only

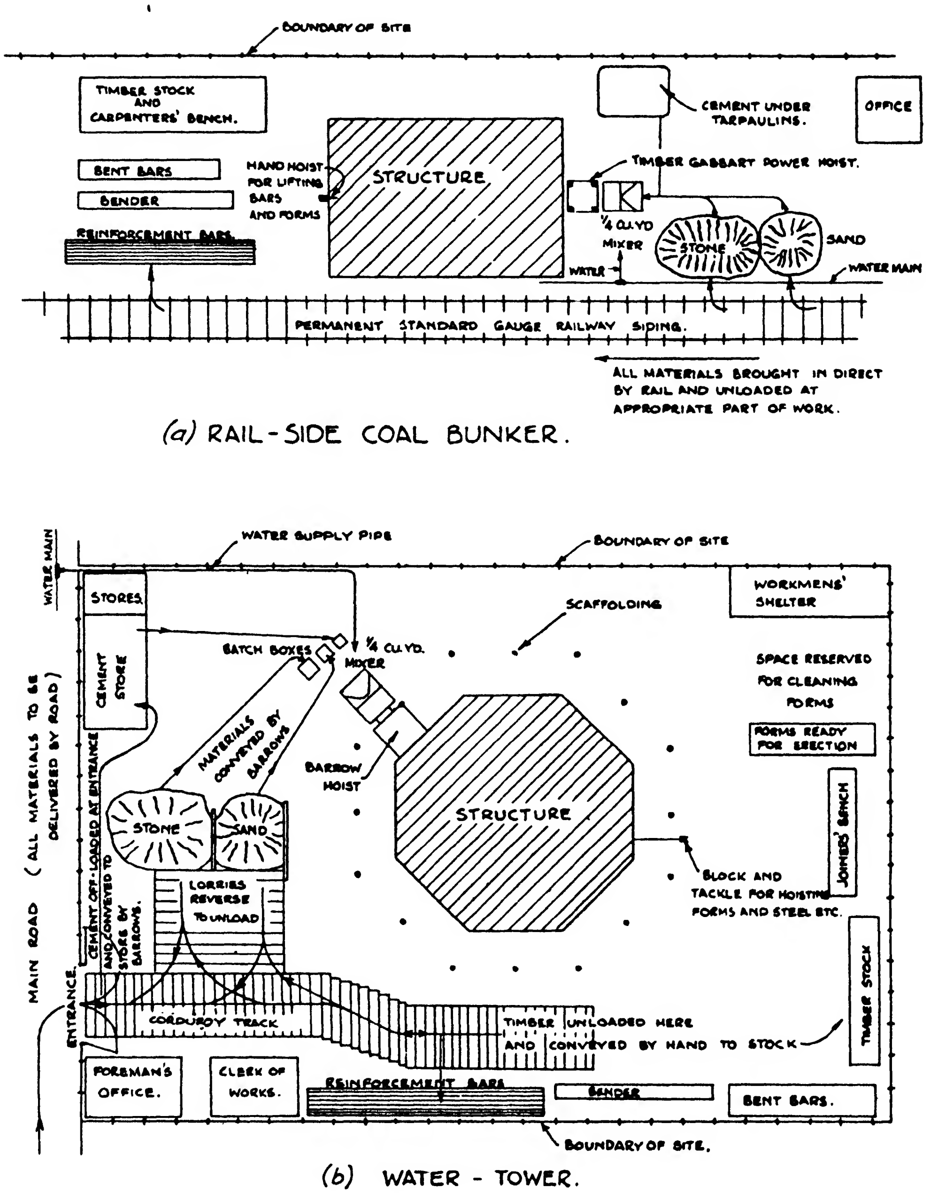


FIG. 26.—ARRANGEMENT OF PLANT FOR ISOLATED STRUCTURES.

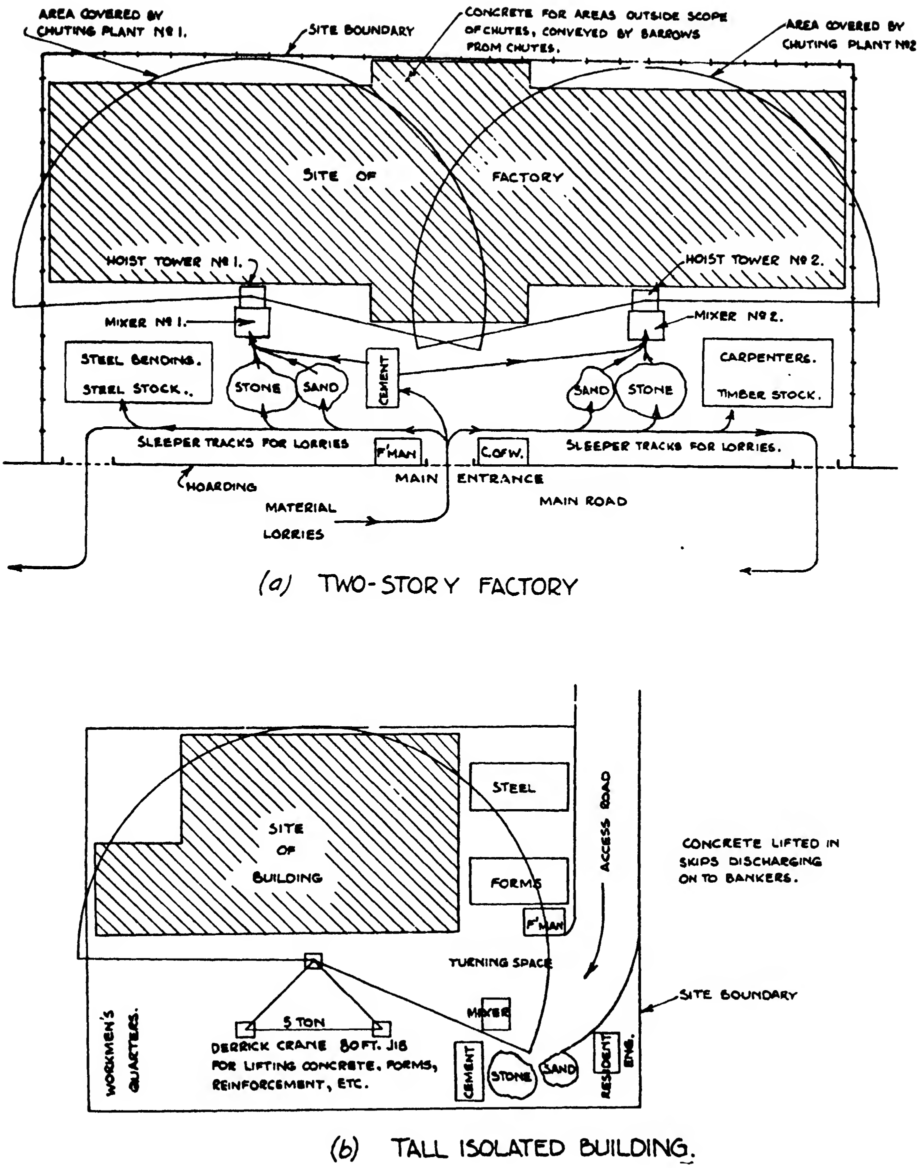


FIG. 27.—ARRANGEMENT OF PLANT FOR BUILDING CONSTRUCTION.

is assumed. Independent entrances and exits are provided and sleeper tracks are installed for temporary lorry roads over the open fields. The concrete is distributed by chuting plants. Since the contract time for most factory construction is usually short, it is necessary to provide two complete mixing and distributing installations in order that as much work as is economically possible may proceed at once. Installation No. 1 would deal with the left-hand side of the job while No. 2 would operate on the right-hand half. It is not essential that both sections of the job should be simultaneously at the same stage of erection, although the greater the lag the longer the overall contract time. On the other hand, a certain amount of lag may enable the contractor to use shuttering that

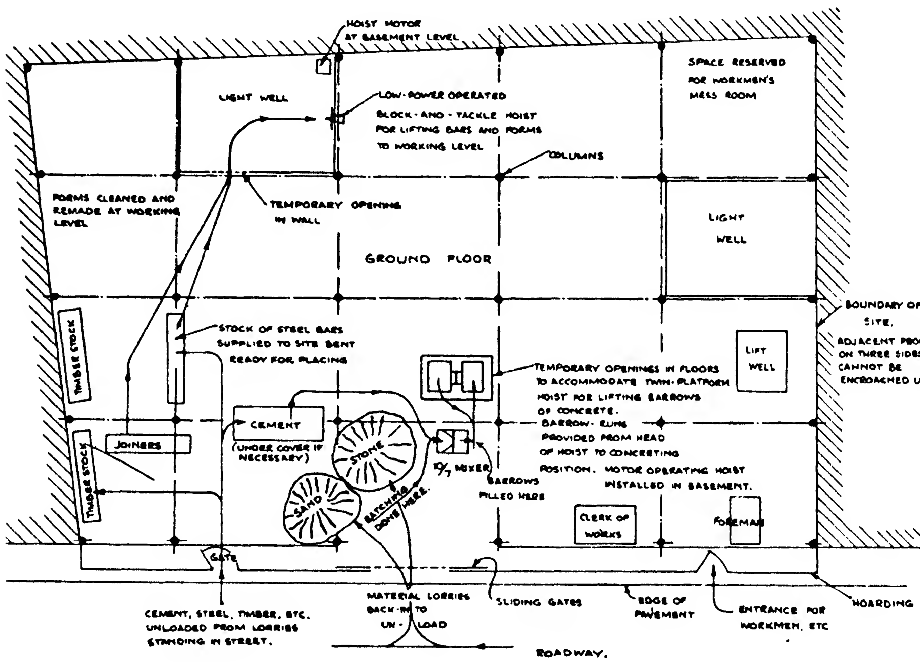


FIG. 28.—ARRANGEMENT OF PLANT FOR TALL BUILDING ON RESTRICTED CITY SITE.

has been struck from one-half on the other half. If there is an approach by railway from the back, the cement and steel might be delivered by rail, although the stone and sand come by road. In such a case it would be better to install the mixers at the back of the site, if space is available, as the cement could then be off-loaded adjacent to them ; tracks could be provided for the lorries delivering sand and stone to enable them to unload at the mixers, thus involving no more handling than in the example illustrated.

For the building considered in *Fig.* 27(*b*), a crane is necessary for handling the excavated material and later for installing permanent machinery, and a 5-ton derrick erected on towers is provided. In addition to its other duties this crane will handle all the concrete, steel, and shuttering and will be in such a position that it can unload from lorries at the end of the access road and at the same time command the whole constructional area. Sufficient space must be provided at the end of the single access road to allow the lorries to turn.

Buildings on confined city sites necessitate special treatment, and *Fig.* 28 shows an arrangement for this case. The special features of this installation are that all operations are carried on in the front of the building to obtain natural lighting as far as possible and to have the mixing plant in a central position on the ground floor. Temporary openings are left in each of the floors for the passage of the concrete hoist; the light wells and lift wells are not sufficiently near the centre for this purpose and extend down to the basement, whereas for the construction of the upper floors it is better to place the mixer at ground level than below, as all materials come in at this level. It might be necessary temporarily to prop the ground floor below the mixer and the material heaps and where the lorries run. Due to the restricted space only small quantities of materials can be stored on the site, but this is not usually inconvenient in city areas where aggregates can generally be supplied at short notice. Sliding gates are preferable to swing gates for large openings as they present no encumbrance to the site or footpath; alternatively a removable section of hoarding can be provided.

Fig. 3 illustrates the early stages of the construction of twin grain silos. Operations are confined to one end of the work where the mixer and a derrick crane are installed. The concrete materials are brought by railway wagon along a temporary siding and dumped alongside the track. Skips running on jubilee tracks convey the material from the dumps to the mixer; from the latter the concrete is lifted in skips by the crane and discharged at the working level on to bunkers or into barrows when the concreting position is beyond the range of the crane. The stock of bent bars is indicated in the photograph. The timber is stacked and the carpentry operations are carried out on the structure.

Bridges.—The construction of almost every bridge requires a different procedure dependent upon such factors as the type of bridge, the size and number of spans, the available working area, and whether traffic has to be maintained on road, rail, or waterway. The arrangement of the plant naturally depends upon the constructional scheme adopted and especially upon whether work is to proceed from one or both ends simultaneously. The examples given in *Figs.* 29 and 31 have been selected from actual jobs as embodying the foremost considerations for this type of structure.

Fig. 29(*a*) applies to a small-span road bridge during the construction of which it is not necessary to maintain traffic on the road or along the waterway below. The work is considered in two stages, the first being during the construction of the foundations. Temporary piers are erected in midstream and upon these is supported a steam derrick crane to handle the material from the excavations and the concrete for the foundation. In the second stage the crane is removed and the temporary piers support the falsework for the superstructure. The concrete for the latter is conveyed in wheelbarrows or carts from the mixer to the required position.

The example in *Fig.* 29(*b*) shows the use of a single crane in the construction of a span of 160 ft. in which the concrete is conveyed in the detachable bodies of jubilee wagons to concreting positions outside the range of the crane. In

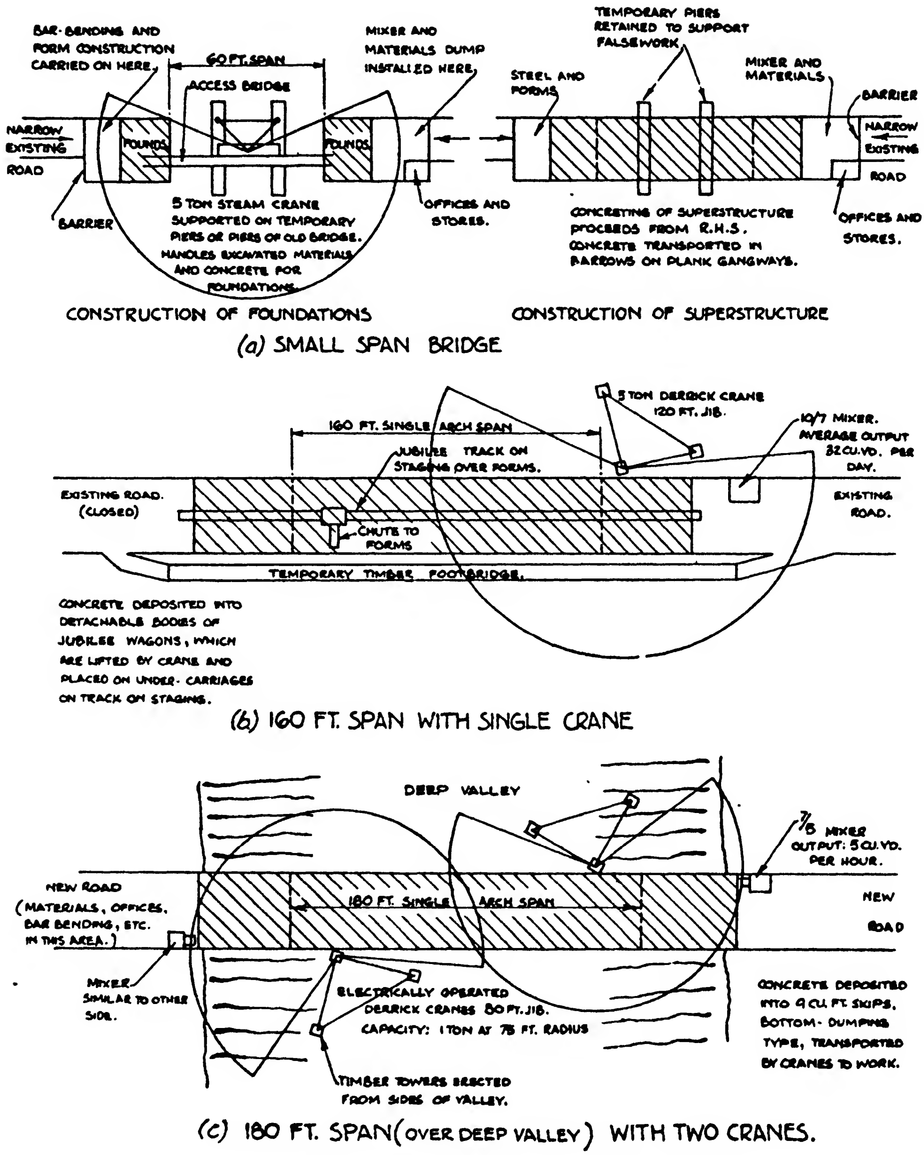


Fig. 29.—Arrangement of Plant for Small Bridges.

Fig. 29(*c*), a bridge of slightly greater span, two cranes are installed completely to cover the work, while in *Fig.* 30 are illustrated two cranes carried on piled trestles which between them cover the site of a three-span structure. The span of the bridge in *Fig.* 31(*a*) is too great to be covered by two cranes only, having regard to the possible positions of the derrick towers. The notes on the diagram indicate the method of concreting the intermediate portion. The construction of the larger bridge represented in *Fig.* 31(*b*) is carried out by an arrangement of eight cranes so covering the work that most portions can be served by two cranes.

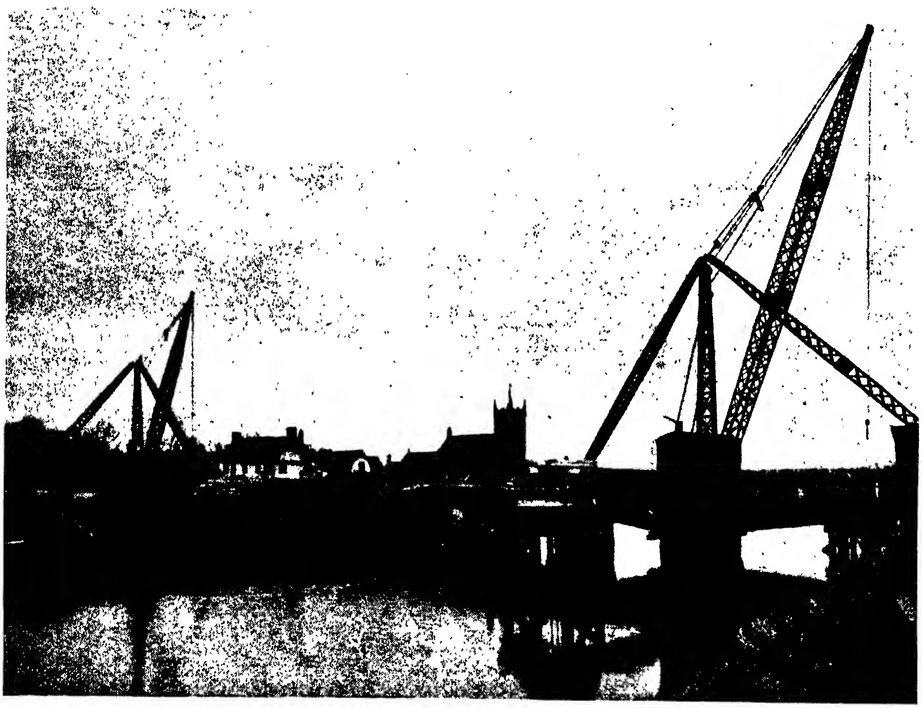
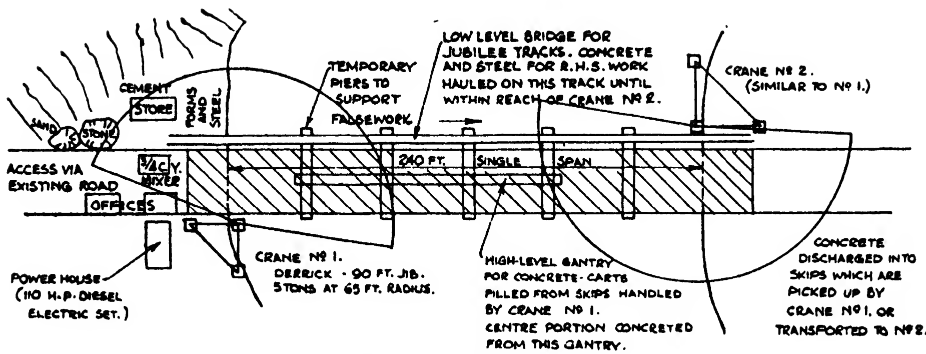
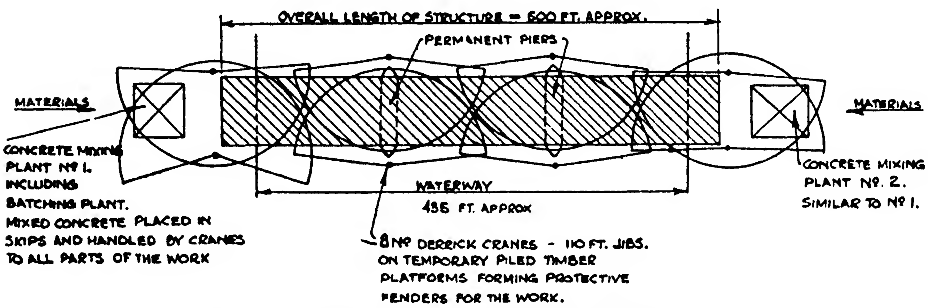


FIG. 30.—TWO CRANES ON BRIDGE CONSTRUCTION.

Other features of the foregoing equipment plans include the provision of the temporary footbridge in *Fig.* 29(*b*) to maintain a pedestrian passageway across the river ; this bridge is also useful for workmen passing from one side to the other in early stages of the work. The valley spanned by the bridge in *Fig.* 29(*c*) being of considerable depth, it is necessary to place the cranes as near the edge as possible (and yet cover the whole work) in order to reduce the heights of the towers. In this case and also in *Fig.* 31(*b*) work proceeds simultaneously from both ends, a mixing plant being installed at each abutment ; materials can be supplied at both ends. In the example in *Fig.* 31(*a*) coal was not economically available, and a power-house was erected for a Diesel engine to supply electric power to the cranes.



(a) 240 FT. SPAN WITH TWO CRANES.



(b) THREE-SPAN BRIDGE WITH 8 Nº CRANES.

FIG. 31.—ARRANGEMENT OF PLANT FOR LARGE BRIDGES.

Wharves, Docks, and Warehouses.—For structures of moderate breadth but of considerable length; different parts of the structure are in different stages of construction at the same time. That is to say, for a wharf surmounted by a transit shed the successive stages that may be on hand simultaneously are construction of wharf substructure, wharf decking, first floor of warehouse and roof of warehouse. With due regard to the disadvantages of crowding one operation on the top of the preceding operation, the work should be planned in such a way that each stage progresses at about the same rate and that activities are confined to as short an overall section as practicable so that supervision, delivery of materials, and mixing are concentrated as much as possible. At the same time all equipment must be such that it can travel forward with the progress of the work.

Space Required for Storing Materials.

A factor in planning is the provision of storage spaces of sufficient area to enable adequate stocks of materials to be maintained. The amount of material in stock at any one time may vary considerably from job to job and depends principally on the rate of consumption in relation to the intervals between successive deliveries and upon the amount in each consignment. The allotment of space for storing steel bars and timber does not usually present much difficulty.

In calculating the area required for a given weight of stone or sand stored in a heap, a natural slope of $1\frac{1}{4}$ horizontally to 1 vertically can be assumed as an average for concrete aggregates. For such approximate calculations an average of 22 cb. ft. per ton of aggregate can be taken. On this basis a roughly-conical heap of material, D ft. in diameter at the base, contains $\frac{D^3}{210}$ tons of material. Similarly a long non-retained bank of aggregate, L ft. wide at the base, contains approximately $\frac{L^2}{110}$ tons per foot run of bank. Any retention afforded to one or more slopes of the heaps or banks by cribwork or otherwise will affect the quantity stored, and special cases should be calculated separately.

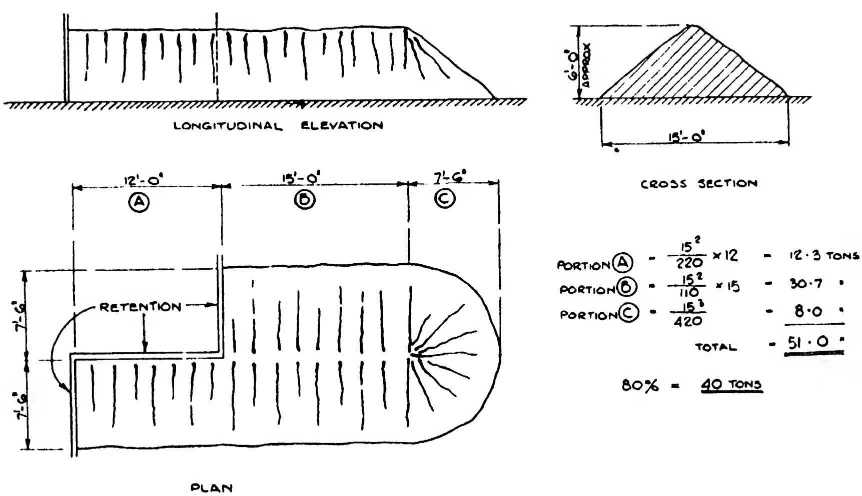


FIG. 32.—QUANTITY OF MATERIAL IN AGGREGATE HEAP.

Consider the amount of aggregate that can be stored in the space illustrated on the plan in *Fig.* 32. The maximum available width at the base of the heap is 15 ft., one end being restricted to half this amount. If the material lies at a slope of $1 : 1\frac{1}{4}$ the approximate height of the heap would be $\frac{7 \cdot 5}{1 \cdot 25} = 6$ ft. The approximate volume of material would be:

$$\begin{aligned}
\text{Portion A} &: 7{\cdot}5 \times 6 \times 12 \times \tfrac{1}{2} = 270 \text{ cb. ft.} \\
\text{,,} \quad \text{B} &: 15 \times 6 \times 15 \times \tfrac{1}{2} = 675 \text{ ,, ,,} \\
\text{,,} \quad \text{C} &: 15^2 \times \frac{\pi}{4} \times \frac{6}{3} \times \tfrac{1}{2} = 177 \text{ ,, ,,} \\
&\overline{\quad 1{,}122 \text{ cb. ft.}}
\end{aligned}$$

At 22 cb. ft. per ton the weight of material would be 51 tons. The same result would be obtained by using the expressions quoted on page 49:

$$\text{Portion A}: \tfrac{1}{2} \times \frac{L^2}{110} \times 12 = \tfrac{1}{2} \times \frac{15^2}{110} \times 12 = 12{\cdot}3 \text{ tons.}$$

$$\text{,, B}: \frac{L^2}{110} \times 15 = \frac{15^2}{110} \times 15 = 30{\cdot}7 \text{ ,,}$$

$$\text{,, C}: \tfrac{1}{2} \times \frac{D^3}{210} = \frac{15^3}{210} \times \tfrac{1}{2} = 8{\cdot}0 \text{ ,,}$$

$$51{\cdot}0 \text{ tons.}$$

This amount might be reduced by 20 per cent. to allow for the heap not being trimmed to the assumed profile.

The area occupied by cement stocks is given consideration in the next chapter, together with remarks on the construction of aggregate storage bins and cement sheds.

CHAPTER II

CONCRETE MATERIALS.

SECTION I.—COARSE AGGREGATES.

Properties of Materials.

A GENERAL clause usually occurs in specifications to the effect that all materials used in the structure " shall be new and of the best quality of their respective kinds and in every respect shall be to the approval of the engineer (or architect) and shall be subject to the tests specified and to such other tests as the engineer (or architect) may deem advisable; any materials rejected shall be replaced by the contractor free of charge and shall be immediately removed from the site." A fair interpretation of such a clause can only be assured if both the site supervisor and contractor are fully conversant with the essential properties required. Since the responsibility of finding suitable concrete aggregate is thus generally laid on the contractor, he should determine, before making up his tender, the source from which such materials can be obtained in sufficient quantities. The foreman or contractor's agent and the resident engineer or clerk of works should be able to decide on the suitability of the aggregates offered by suppliers, and as the work progresses it is equally necessary for those in charge to be able to satisfy themselves that the materials being supplied are up to standard. In this chapter are described the properties, quantities, and methods of handling, storing and testing the materials used in the making of concrete, namely, coarse aggregate (stone), fine aggregate (sand), cement, and water. In the following chapter, methods of measuring and proportioning are dealt with.

Quality of Coarse Aggregate.—The essential properties of coarse aggregates are hardness, cleanliness, suitability of shape, chemical inertness, and, for certain construction, resistance to fire. The material should not contain soft, friable, elongated, or laminated particles, nor be contaminated by admixtures or coatings of alkali, organic matter, or other substances likely to affect the strength or durability of the concrete. The material should be free from adhering particles of loam or clay, reasonably free from crusher dust, and of limited porosity. Where possible it is preferable to take the material from sources that have provided satisfactory coarse aggregates for reinforced concrete over a number of years. Even so the material should be inspected to ensure that it is suitable; and when taken from a new source the aggregate should be inspected and tested to ensure conformity with the specification. The selection is largely determined by what is available locally at a reasonable price, the most common aggregates being various types of gravel and crushed stone, although broken brick, blastfurnace slag, and clinker are also employed for appropriate purposes as discussed later.

Generally, gravel and crushed natural stone should be used for reinforced concrete work, while either of these materials or hard brick or tile can be used for plain concrete.

Materials that arrive on the site in a particularly dirty or contaminated condition can rarely be economically treated on the job to render them fit for use, but small amounts of clay and loam can be washed out. Crusher dust can also, to a certain extent, be removed, while screening may also remove other contaminations or under-size or over-size particles.

The coarse aggregate should be at least as strong in compression as the required concrete. Although the aggregates ordinarily used in reinforced concrete may vary widely in strength, the strengths of the resulting concretes may not vary greatly, other factors being equal, whereas variation in these other factors can result in wide and more serious fluctuations of strength. A minimum crushing strength of 5,000 lb. per square inch is recommended, a figure that enables " third strength " bricks to be used. For mass concrete or unloaded structures, aggregates of considerably less strength are sometimes used.

The shape of the particles is important in obtaining a satisfactory workable concrete; pieces approaching cubical or spherical forms are much preferable to laminated, elongated, or splintered pieces. The surface of the pieces should be hard, non-powdery, and rough enough to ensure that the cement adheres to the stone. Pieces with glassy surfaces should be avoided.

Aggregates such as broken brick and certain broken stones may be porous, and their use should be limited to parts of the structure where there is small risk of corrosion of the reinforcement. A maximum limit of porosity is that after twenty-four hours' immersion in water the material shall not have increased in weight by more than 10 per cent. if the concrete is for use in foundation or mass work or not more than 5 per cent. if for use in impermeable construction or for high strength concrete. Porous aggregates should be soaked long enough in advance of mixing the concrete and with sufficient water to ensure that absorption is complete.

The strength of concrete may be much reduced by small amounts of clay and loam in the aggregates, especially if these adhere to the stone. These also delay the hardening, especially at low temperatures; less harm is done if small quantities are present as free and fairly hard particles. A specified limit for the permissible content of clay, loam, mud, slime or other fines in a free state, is 3 per cent. by weight for first-class work, although 5 per cent. is often allowed for mass concrete and foundations. Most aggregates containing these amounts can be improved by washing. More stringent regulations limit the content of clay lumps to 1 per cent. while others specify 6 per cent.

Loose loam, adhering clay, vegetable earth and other organic substances such as pieces of wood, grass, leaves, or coal may be readily detected by eye. Such foreign substances in small quantities can readily find their way into aggregates heaped directly on the ground. In hardened concrete adhering clay films are indicated in broken pieces by the loose bedding of the stones in the mortar.

The essential properties of a fire-resisting aggregate are a high refractory quality, a low coefficient of thermal expansion, and a low thermal conductivity. When heated and quenched by water the aggregate should not split. Generally, fine-grained stone is better than coarse grained. Hard stones usually show little loss of strength if heated up to 900 deg. F. and cooled in air, but heating above this temperature and quenching with water produce considerable disintegration of most stones, including granite, basalt, sandstone, and limestone. Stone chippings appear to be less affected by extreme temperatures than coarse broken stone. Most bricks produce some of the best fire-resisting aggregates.

Gravel Aggregate.—Pit and river gravel, Thames ballast, and similar materials are some of the most suitable, widespread, and commonly-used aggregates when washed clean and free from loam and vegetable matter. When taken from river beds in colliery districts or near coaling stations, isolated pieces of coal may be encountered, which should be removed. Pit ballast usually needs more treatment than river material, but by washing and screening a well-graded and strong material with particles of suitable shape is obtainable. Sometimes gravel aggregate supplied for reinforced concrete work contains a fair amount of crushed material due to the reduction of over-size stones. In some districts flint constitutes a considerable proportion of the gravel, consequently the pieces of crushed stone are angular and not usually so suitable in shape as uncrushed water-worn pebbles. Gravel aggregate usually contains less voids than crushed stone of the same grading, and is slightly superior to crushed stone for impermeable construction. It has a high resistance to attack by sulphates but tends to split and be unreliable under intense heat. Rounded gravel aggregates are not generally suitable for the surfaces of roads or other slabs subject to abrasion. A standard test aggregate recommended by the Institution of Structural Engineers is an uncrushed washed Thames ballast which has been dried and freed from shells.

Gravel taken out during excavation should seldom be used for reinforced concrete work without previous treatment. Except on large works, treatment is usually more expensive than disposing of the excavated material on the best obtainable terms and purchasing suitable aggregate elsewhere. Where a blinding layer of concrete is specified to be laid below the foundation, excavated gravel can be used if it is free from loam or other overburden.

Beach gravel or shingle is used in some districts and when carefully selected is a good aggregate. Dirtiness and contamination by organic matter are the more common defects of beach shingles and, as with other gravels, uncrushed material is preferable. Old beds of beach shingle often supply a suitable aggregate when the material has been washed and screened. When taken from above high-tide level beach shingle may contain an excess of salt, but the traces of salt present in otherwise suitable material taken from below high-water mark are of little consequence, especially for exterior work. Excess of salt causes efflorescence, when, for example, untreated beach shingle is used in mass concrete. Although the washing may not remove the whole of the chlorides, the residue does no harm if the concrete is free from cracks; if moisture obtains access through cracks to the reinforcement the salt accelerates corrosion. In doubtful cases, some trial specimens of concrete should be made with the material before the work is started.

Broken Stone Aggregate.—In selecting a broken stone, soft or friable materials should be avoided. Granite, whinstone, quartzite, flint, basalt, and certain sandstones are best. Large numbers of sandstones are totally unsuitable for concrete work, and even where a strong sandstone is used the resulting concrete is often porous and therefore not suitable for impermeable construction. Concretes made with soft sandstones tend to be more liable to develop shrinkage cracks.

Where obtainable locally granite is an excellent aggregate, and even in localities at some distance from a source of supply its high resistance to wear may offset the extra cost. For this reason it is largely used for surfacing concrete roads, floors, stairs, and other work subject to abrasion. All granites are not of the same quality, and those containing partially decomposed felspar or large flakes of mica should be avoided. In some parts of the country rocks having the appearance and title of granite are encountered, but these are much inferior to true granites.

Flint is hard and produces a strong concrete; it usually requires crushing and care should be taken to select well-formed particles, excluding sharp splinters. A clean, hard Portland stone, reduced to suitable sizes by crushing, is a good aggregate if free from dust.

Most broken hard stones, with the exception of flint and all sandstones, offer good resistance to attack by sulphates and generally poor resistance to high temperatures. Granite flakes and cracks on heating above 900 deg. F. Flint and quartzite have poor fire resistance and a high coefficient of thermal expansion. Sandstone is a highly refractory material at low temperatures but splits under intense heat.

Whinstone chips are a satisfactory coarse aggregate, but when whin dust has been used as fine aggregate in combination with whinstone chippings failure of the concrete to harden has been experienced, especially when low temperatures prevailed at the time of placing the concrete. It is therefore advisable to screen out the dust and replace it with siliceous sand.

The presence of crusher dust in most crushed stone aggregates is often considered to be detrimental to the concrete. Although this may be so when the stones are coated with the dust, it is questionable whether small proportions (say up to 5 per cent.) are harmful to any practical degree. The presence of dust in certain basalt aggregates is not necessarily harmful.

Broken concrete, which can be considered as broken artificial stone, should not be used in first-class reinforced concrete structures, although in mass concrete foundations or similar work where porosity is not objectionable such aggregate may be used.

Shale or similar materials are not good aggregates.

Hard crystalline limestone is a good aggregate. Unfortunately there is a certain amount of bias against the use of this material, a bias that seems to have been largely fostered by the prohibition in the L.C.C. Regulations (1915) of the use of limestones, marble, and other calcium compounds. The material is extensively used in this country, and selected material has produced concrete of higher strength than that given by inferior gravel aggregates ; good limestone chippings have also proved successful in important marine work. It is essential that limestone should be hard and entirely free from dust. Its use is not recommended for impermeable construction. The fire-resisting qualities of limestone are equal to that of gravel or granite but inferior to that of broken brick, as it has a tendency to calcine at a high temperature. Limestone should not be used for chimney construction, as calcium sulphate may be formed in the reaction between sulphur fumes and the carbonate in the aggregate.

Broken Brick Aggregate.—Broken brick, which in accordance with the London Building Act may not be used for reinforced concrete, is not usually satisfactory for reinforced concrete work unless the bricks are of good quality, hard, and free from lime mortar, lime sulphate plasters, and brick dust. The London County Council Building By-Laws classify bricks according to their strengths, " first strength " having a crushing strength of upwards of 10,000 lb. per square inch and " third strength " a crushing strength of not less than 5,000 lb. per square inch. " First," " second," or " third " strength bricks are suitable aggregates for structural concrete if they conform to the foregoing requirements. Bricks of lower strength may be suitable for mass concrete or other work where dimensions are generous and working stresses low. If crushed by machine, broken brick is frequently cheaper than most aggregates, but a more suitable, although if in large quantities a much more costly, material results from breaking by hand. Concrete of high strength can be made with broken hard firebrick obtained from dismantled coke-ovens or similar sources. Owing to its porosity, brick aggregate should not be used for impermeable construction, and all broken brick used for other concrete work should be thoroughly soaked immediately before mixing the concrete. Broken brick concretes, which are frequently lighter than gravel aggregate concretes, are fire-resisting to a high degree. The most refractory bricks are not necessarily the strongest. The greater the proportion of alumina to silica the greater the infusibility.

Slag, Coal Residue, and Pumice Aggregate.—Coal and coal residues, clinker, ashes, coke breeze, pan breeze, copper slag, forge breeze, dross, and similar materials should not be used as aggregates, as they are liable to reduce the strength or durability of the concrete, while those that contain soluble sulphates including gypsum are likely to result in corrosion of the reinforcement.

Selected broken blastfurnace slag is a good concrete aggregate and is now generally allowed. Carefully selected slags produce concretes that have qualities of strength and durability comparable with gravel concretes, and if well graded to produce a dense concrete render the reinforcement immune from corrosion. If more than ordinary care is not taken with the grading and proportioning, a porous concrete will be obtained that will allow moisture to obtain access to the steel; corrosion will be accelerated by the presence of sulphur in the slag. Boiler and locomotive slags may contain chemical combinations of sulphur, and if the amount of such substances exceeds 1 per cent. the slag should be rejected. Glassy and perforated pieces of blastfurnace slag are detrimental and should be limited to 5 per cent. of the whole. Slag should be tested before use to determine its physical and chemical properties.

A good siliceous sand is preferable to granulated slag for use with broken blastfurnace slag. Slag concrete, which is lighter than gravel concrete, has fairly good fire-resisting properties and is an economical material in iron-working districts. Old deposits of blastfurnace slag often at first sight appear to be totally unsuitable for use as a coarse aggregate, but if all material below $\frac{1}{4}$ in. is screened out and the remainder well washed and mixed with coarse sand a concrete can be produced that will be suitable for mass concrete in engine beds, pit linings, and similar work.

Although not suitable for reinforced concrete construction, clinker and breeze are used successfully as aggregates in other concrete work. Coke breeze, which is practically unused for concrete, must not be confused with breeze and clinker which are residues from the combustion of coal and differ only in the fact that clinker is more thoroughly fused than breeze. Clinker is therefore the superior material, making a concrete that is stronger than breeze concrete although considerably inferior to gravel concrete. All clinker can be improved for concrete making by screening out grains less than $\frac{1}{8}$ in. in size and adding sand. Clinker and breeze concretes are extremely permeable and may contain sulphur compounds, causing rapid corrosion of reinforcement; such aggregates are unsuitable for structural purposes. These furnace residues may also contain a large percentage of combustible material and coal, which are dangerous owing to their expansive properties. Clinker from old sewage beds should not be used, as the bacteria content is extremely detrimental to concrete.

The fire-resisting properties of breeze and clinker concretes are only moderate and uncertain, and their use in constructional work is prohibited by the London County Council By-laws. These materials, however, are largely used for partition walls and fixing bricks, since they produce a cheap and light-weight concrete providing a good key for plaster and a hold for nails.

Pumice is of volcanic origin, and except for a possible small sulphur content it contains no compounds harmful to concrete. Although it is allowed by some Continental regulations, great care must be exercised in adopting this material as an aggregate for load-carrying structures. Unless the pumice is carefully graded, free from dust, and the concrete is mixed rather dry, the strength may be well below that considered satisfactory for ordinary work. Experience in using pumice is, however, leading to its more extensive adoption for light-weight panel work and partitions, nailing strips, precast slabs and bricks, insulating layers and similar unreinforced work. Stairs faced with a concrete made from granite chippings, roof slabs, and vaultings are also being successfully constructed with this material. Pumice concrete exhibits a high shrinkage but fire-resisting qualities are good. Its use in impermeable construction is not generally recommended although damp-proof qualities are claimed for pumice concretes as well as sound and heat-insulating properties.

Gauge and Grading of Coarse Aggregate.

Next in importance to the quality of the aggregate is the size and grading of the particles. For normal reinforced concrete construction the size of the pieces of stone is specified as being not less than $\frac{3}{16}$ in. and not more than $\frac{3}{4}$ in.; that is to say, all the material must pass through a sieve having $\frac{3}{4}$-in. square openings and none must pass through a sieve with $\frac{3}{16}$ in. square openings measured in the clear. The maximum sizes of the particles vary, however, with the type of work. A useful maximum size of aggregate in slabs, walls, and similar narrow work is 20 to 25 per cent. of the thickness of the concrete. Thus a $\frac{3}{4}$ in. aggregate would be suitable for work not less than 3 in. thick. Generally the largest stone should be not too big to pass between the closest-spaced reinforcing bars or between the bars and the face of the shuttering. Some regulations require the maximum size of aggregate not to exceed three-quarters of this clearance. In ordinary design the clearance and cover is usually not less than 1 in. for beams and columns, so that $\frac{3}{4}$-in. gauge material is satisfactory. In slabs and panel walls a minimum cover of $\frac{1}{2}$ in. is normally adopted, but in a well-graded aggregate there would be enough smaller gauge material to enable sufficient concrete to pass between the bars and the shuttering.

When reinforcement is closely spaced, or when the concrete is fairly dry, a richer and wetter mix with $\frac{3}{8}$-in maximum gauge aggregate should be placed under and between the bars in the bottom of beam boxes before placing the bulk of the drier concrete with $\frac{3}{4}$-in. aggregate. In such work as the narrow ribs of hollow floors, or in granolithic or other surface renderings, a maximum size of $\frac{1}{2}$ in. or $\frac{3}{8}$ in. is more suitable than the normal $\frac{3}{4}$ in.

The minimum size of stone usually classed as coarse aggregate is $\frac{1}{4}$ in. or $\frac{3}{16}$ in. and is principally decided by the supplies available, being especially controlled by the maximum size of grain that is economically procurable for the fine aggregate. If the fine aggregate can be obtained graded up to $\frac{1}{4}$ in. then the minimum gauge of the coarse aggregate should not be more than $\frac{1}{4}$ in., but if the fine aggregate does not exceed $\frac{3}{16}$ in. the coarse aggregate should be graded down to at least $\frac{3}{16}$ in.

For heavier construction such as mass concrete foundations, bridge piers, and similar works, a larger maximum gauge up to 3 in. may be permissible, while the insertion of "plums" is also allowable under the condition discussed in a later chapter. For single-course concrete roads a maximum size of $1\frac{1}{2}$ in. to 2 in. is suitable, but in the top layer in two-course construction the aggregate should not exceed $\frac{3}{4}$ in.

The grading of the coarse aggregate plays an important part in the density and strength of the concrete, although perhaps not to the same extent as the grading of the fine aggregate, as deficiencies in the grading of the coarse aggregate can be partly overcome by careful proportioning. Most specifications give only the extreme allowable sizes, say $\frac{3}{16}$ in. and $\frac{3}{4}$ in., and merely state that the material should be well graded between these limits. This is generally unsatisfactory from the point of view of those who have to comply with the specification and those who have to enforce it. The ideal grading is represented by a straight line between the two limiting gauges. Although laboratory-mixed aggregates could be prepared to conform to this ideal, variations from it are inevitable in commercially-supplied aggregates. To prevent argument as to the interpretation of the expression " well graded " it is preferable to specify a limiting percentage for an intermediate gauge, say $\frac{3}{8}$ in.

The grading for a " standard " coarse aggregate in the specification of the Institution of Structural Engineers is as follows:

Size of mesh (in.)	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$
Percentage by weight passing	nil	20	40	60	80	100

Normally, however, this grading would be attained only by expensive screening and mixing, and a material with a grading lying between the following limits should be acceptable when a " well graded " $\frac{3}{16}$-in. to $\frac{3}{4}$-in. coarse aggregate is specified:

Size of mesh (in.)		$\frac{3}{16}$	$\frac{3}{8}$	$\frac{3}{4}$
Percentage by weight passing	Maximum	10	60	100
	Minimum	nil	25	95

Closer analysis than the maximum, minimum, and one intermediate gauge is not necessary except in special cases (see description of proportioning by " fineness

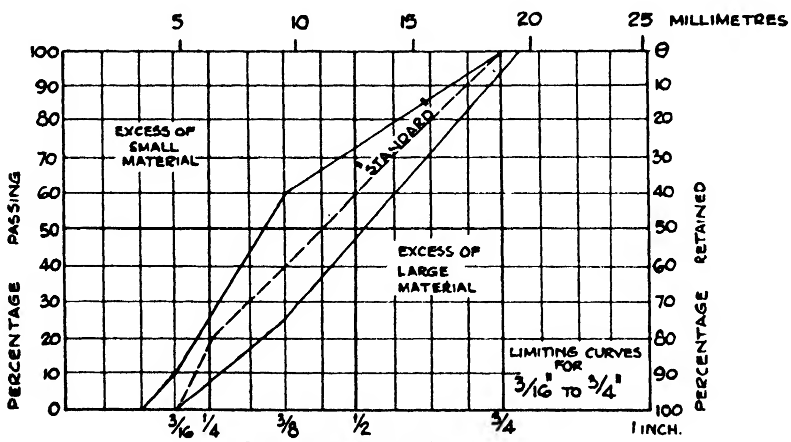


FIG. 33.—GRADING OF COARSE AGGREGATE.

modulus " method). Curves giving the limiting gradings of $\frac{3}{16}$-in. to $\frac{3}{4}$-in. material are given in *Fig.* 33 and the grading curves of satisfactory coarse aggregates should lie between these limits.

For reinforced concrete work where a large-gauge aggregate (say $1\frac{1}{2}$ in.) is allowed the following grading would be satisfactory:

		$\frac{1}{4}$	$\frac{3}{4}$	$1\frac{1}{2}$
Size of mesh (in.)		$\frac{1}{4}$	$\frac{3}{4}$	$1\frac{1}{2}$
Percentage by weight passing	Maximum	5	75	100
	Minimum	nil	40	95

For mass concrete work with a specified maximum gauge of $2\frac{1}{2}$-in., not more than half the weight of a well-graded aggregate should exceed $1\frac{1}{2}$-in.

The remarks on sieve sizes, discussed later in this chapter, should be considered when a grading analysis is being made or specified.

SECTION II.—FINE AGGREGATES.

Quality of Material.

The properties of hardness, cleanliness, and freedom from clayey or organic admixtures that are essential in the coarse aggregate are equally necessary in the sand or fine aggregate. Such material should preferably be composed of hard siliceous grains, and should be chemically inert, and free from excessive quantities of dust, soft or flakey particles, shells, congealed lumps, nodules of clay, shale, alkali, or other contaminations likely to affect the strength and durability of the concrete. The shape of the particles is not of great importance, and with clean and well-graded materials rounded grains are not inferior to sharp grains.

The presence of loam or clay can be detected by shaking the sand in water in a glass container and observing the amount of silt precipitated, while the presence of deleterious organic matter can be revealed by mixing the sand with caustic soda solution. These tests, together with others that determine the suitability of sand for use in reinforced concrete work, are described later in this chapter. Washing will usually remove clay or loam, and a decantation test (also described later) may be made to determine the proportion of silt or clay in the sand.

River (fresh water) sands are usually naturally clean, although cases occur where they are contaminated with mud. Pit sands nearly always require washing, and may contain organic matter. Otherwise sands from either of these sources are generally suitable for reinforced concrete construction.

Sea sand, if not too fine, and if taken from below high-water level, is often suitable; the salt content may be excessive if taken from above high-water mark, and in all cases the sand should be tested for organic impurities due to the presence of marine life. Trial concrete cubes should be made and tested in the case of doubtful materials.

Foundry and silver sands are too fine for concrete work, but fine white Leighton Buzzard sand is used for standard mortar tests.

The screenings from crushed stone are often suitable for fine aggregate if free from an excessive amount of dust and if the particles are not too flakey and splintery. Accurate gradings can often be obtained in these materials.

Sand derived from a stone that is not suitable for coarse aggregate should not be used for fine aggregate. Certain granite screenings are as good as river sand, but neither limestone nor whinstone "sands" should be used for reinforced concrete construction. Sandstone grit is used in some districts but is inclined to be dusty.

Crushed brick is not generally suitable as a fine aggregate, especially if crushed down from old brickwork, as the "sand" may contain powdered lime mortar. If clean hard bricks are crushed and any excess of fine material extracted, the material is suitable for mass concrete work. Crushed unglazed pottery and tiles have been recommended for renderings for impermeable construction.

Although blastfurnace slag may be suitable for use as coarse aggregate, granulated slag is not recommended as a sand for reinforced concrete work as the grains are weak and contain a high percentage of voids.

Grading of Fine Aggregate.

The grading of the fine aggregate has more effect on the strength of the concrete than the grading of the coarse aggregate, although for impermeable construction good grading of both materials is essential. As already explained, the maximum size of the sand particles should bear some relation to the minimum size of the coarse aggregate, and the grading should be as uniform as practicable from this size down to almost dust. Normally $\frac{3}{16}$ in. to $\frac{1}{4}$ in. maximum size is suitable.

For renderings of cement and sand, $\frac{1}{8}$ in. or smaller is often the maximum gauge that can be used consistent with workability.

The grading of a "standard" fine aggregate in the specification issued by the Institution of Structural Engineers is as follows:

Nominal size of mesh (in.)	$\frac{1}{75}$	$\frac{1}{50}$	$\frac{1}{32}$	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{3}{16}$
Percentage by weight passing	nil	20	40	60	80	100

The curve of this grading, which would rarely be obtained under practical conditions, is given on *Fig.* 34. Most natural sands and many crushed stone sands possess too large a percentage of fine grains to be suitable for concrete work without screening or mixing or both. Gradings should lie between the limits indicated by the heavy curves on *Fig.* 34 which mark the fine and coarse limit of suitable concrete sands with $\frac{1}{4}$-in. and $\frac{3}{16}$-in. maximum gauge. Sands of which the sieve analysis curves lie between these limits may sometimes be procurable naturally and economically, while it is often necessary to mix two materials to obtain a satisfactory fine aggregate. The gradings of upwards of twenty samples taken from deposits in various parts of the country and described as "building sands" (as distinct from "plastering sands") were found to lie between the curves marked (*a*) on *Fig.* 34. It will be observed that these samples represent material that lies entirely within the fine zones.

Generally speaking, the coarser the sand the better will be the concrete (other factors being equal). Some specifications require that at least 50 per cent. of $\frac{1}{4}$-in. maximum-gauge sand shall be retained on a $\frac{1}{16}$-in. mesh, while others aim at reducing the amount of fine grains by stipulating that not more than 10 per cent. shall pass a sieve having 50 meshes per linear inch and that

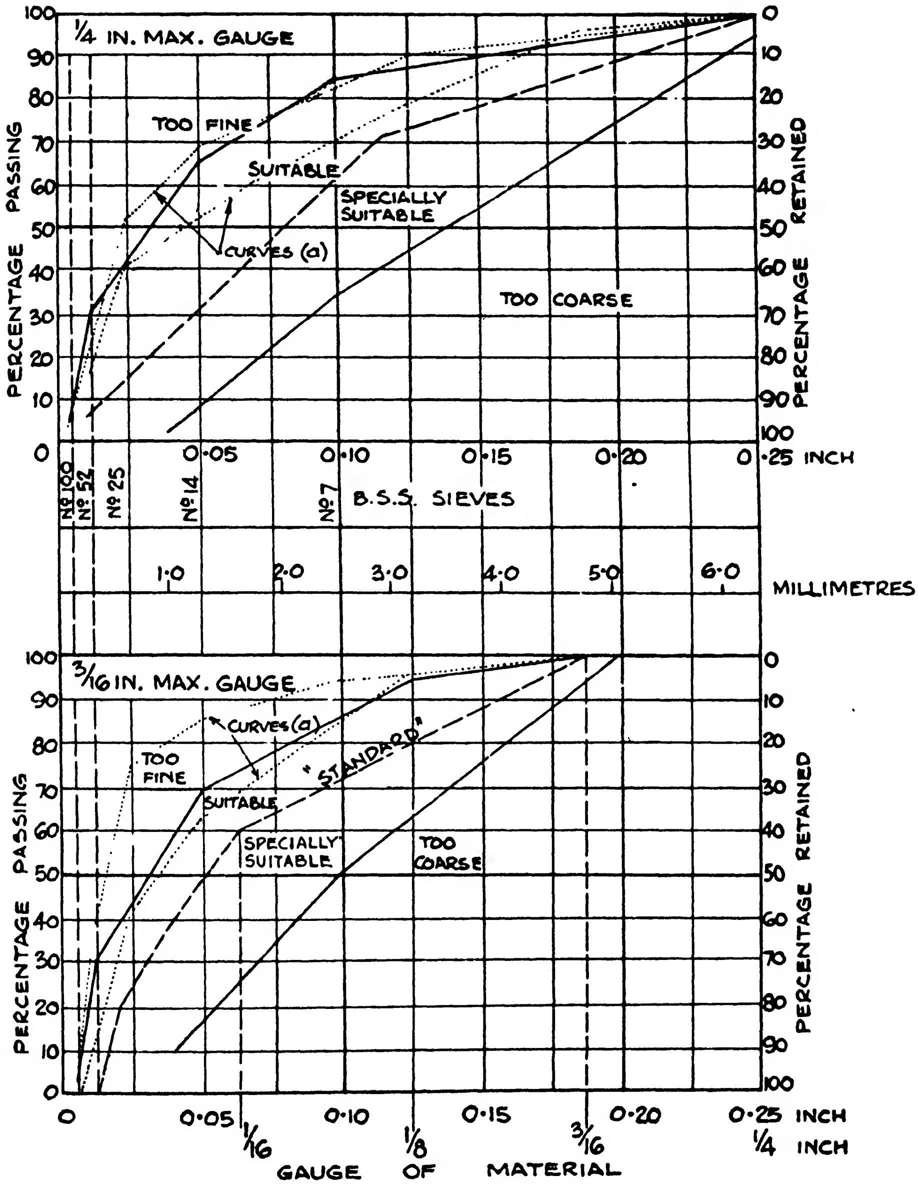


FIG 34.—GRADING OF FINE AGGREGATE.

not more than 3 per cent. shall be under 0·006 in. (No. 100 sieve). The London Building Act (1938 Regulations) stipulates $\frac{3}{16}$ in. maximum gauge with not more than 5 per cent. by weight passing the No. 100 sieve.

Any sand within the fine zone (*Fig.* 34) is suitable for ordinary construction, but for roads or other surfaces subject to abrasion a sand within the coarse zone should be selected. For impermeable construction a sand should not be close to the fine limit and fine sands should be avoided when using finely-ground cements. The use of an exceptionally coarse sand, however, may result in the production of interstices that are not completely filled with cement paste, thus producing a source of weakness and tending to produce a harsh concrete.

Consideration of "mixed aggregates," that is combined fine and coarse materials, is given later.

The current British Standard Specification for Portland Cement requires sand used for testing to have grains of fairly uniform size, that will pass a sieve having 18 meshes to the linear inch (0·0336 in. nominal clear dimension) and that will be retained on a sieve having 25 meshes per inch (0·0236 in. clear dimension).

Mixing Sands.

When an available sand is not graded suitably for reinforced concrete work a satisfactory material can be obtained by mixing two or more sands, and by preliminary study the most economical mixture can be determined. Consider, for example, a job for which the specified grading of the fine aggregate is, as

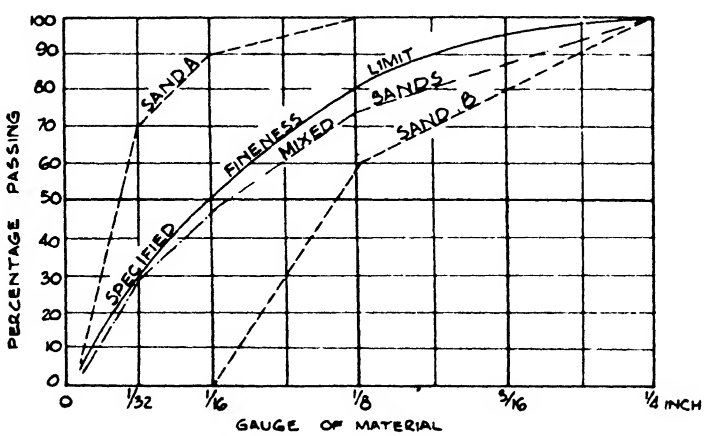


FIG. 35.—MIXING SANDS.

shown in *Fig.* 35, up to $\frac{1}{4}$ in. with not more than 50 per cent. less than $\frac{1}{16}$ in. The only material available locally is a fine sand (A) with a maximum gauge of $\frac{1}{8}$ in. and with intermediate gradings as given below. This sand can be delivered on the site at 6*s.* per ton, and by measurement it is found there are 20 cu. ft. in a ton. A much coarser grit sand (B) with particles between $\frac{1}{16}$ in. and $\frac{1}{4}$ in. can be obtained from a distance at 10s. a ton of 24 cu. ft. delivered, and the grading of this sand is also given below:

Size of mesh (in.)	0·006	$\frac{1}{32}$	$\frac{1}{16}$	$\frac{1}{8}$	$\frac{1}{4}$
Percentage by weight passing: Sand (A)	8	70	90	100	100
Sand (B)	nil	nil	trace	60	100

The grading curves of sands A and B have been plotted on *Fig.* 35, from which it will be seen that by itself neither sand is satisfactory. Since sand B is more costly than sand A, the most economical mixture is that which contains the largest permissible proportion of sand A.

To conform to the specified limit for any size, the relative weights of sand A to sand B will be as $1 : x$ where x is given by $x = \dfrac{A - S}{S - B}$ and A = percentage of sand A passing a mesh of given size, B = percentage of sand B passing a mesh of given size, and S = specified maximum percentage passing a mesh of given size.

Substituting known values for the $\frac{1}{16}$-in. gauge $x = \dfrac{90 - 50}{50 - 0} = 0{\cdot}80$.

Similarly for the $\frac{1}{32}$-in. gauge $x = \dfrac{70 - 30}{30 - 0} = 1{\cdot}33$,

and for the $\frac{1}{8}$-in. gauge $x = \dfrac{100 - 80}{80 - 60} = 1{\cdot}00$.

It appears that the $\frac{1}{32}$-in. gauge material will control, and therefore the proportions will be sand A to sand B = $1 : 1\frac{1}{3}$. That is, for every pound of sand A there would be $1\frac{1}{3}$ lb. of sand B, and such a mixture would have a grading as indicated on the appropriate curve on *Fig.* 35. It must be remembered that the sieve analyses are based on percentage by weight while the measurement of the amount of sand required in the concrete mix is by volume. Thus the price consideration should be based on volume. The resulting cost of the mixture will be:

One ton of sand A	= 6*s*. 0*d*.
$1\frac{1}{3}$ tons of sand B at 10*s*.	= 13*s*. 4*d*.
$2\frac{1}{3}$ tons mixed	= 18*s*. 4*d*.

The volume of this mixture is:

One ton of sand A	= 20 cu. ft.
$1\frac{1}{3}$ tons of sand B at 24 cu. ft. per ton	= 32 ,, ,,
Total unmixed	= 52 cu. ft.

Allowing a small reduction in bulk upon mixing the two sands, the total volume of $2\frac{1}{3}$ tons when mixed would be, say, 50 cu. ft. This reduction would not be generally apparent, as the two sands would be placed separately in the materials skip and would be mixed, together with the cement and coarse aggregate, in the drum of the mixer. The net cost of sand would be

$$\frac{18{\cdot}33s.}{50 \text{ cu. ft.}} = 4{\cdot}4d. \text{ per cubic foot.}$$

Since the required grading is obtained by mixing 1 : $1\frac{1}{3}$ by weight of each sand, the cubic measures necessary are 20 volumes of sand A and 32 volumes of sand B ; that is 1 : 1·6 by volume. If the concrete is specified as 1 : 2 : 4 the relative volumes of each sand per hundredweight bag of cement would be found thus :

Net volume of mixed sand required = 2·5 cu. ft. Allowing 5 per cent. reduction due to mixing, the gross volume would be 2·5 × 1·05 = 2·62 cu. ft.

Volume of sand A $2{\cdot}62 \times \dfrac{1}{1 + 1{\cdot}6}$ = 1·0 ,, ,,

Volume of sand B $2{\cdot}62 \times \dfrac{1{\cdot}6}{1 + 1{\cdot}6}$ = 1·62 ,, ,,

The foregoing calculation implies that the gradings of sands A and B remain uniform throughout different parts of the working face of the pit. Thus constant checking should be made to test the grading of each consignment and to alter the proportions as required.

Sieve Sizes.

When considering the gauge of aggregates, especially sand, it is necessary to note whether the size limitations are expressed as so many inches (or fraction of an inch) or in terms of the number of meshes per linear inch. For instance, a fine aggregate may be specified as not larger than will pass a square mesh measuring $\frac{1}{4}$ in. in the clear, or it may be specified to pass a sieve having four meshes per linear inch. The latter material would be smaller than the former, say, about $\frac{3}{16}$ in., since the clear size of the mesh would be nominally $\frac{1}{4}$ in. minus the wire diameter. Since sieves are usually made to a definite number of meshes per linear inch, a specification having reference to the number of meshes is more practicable than one giving clear dimensions, so long as a standard sieve is specified. The range of standard sieves given in the appropriate British Standard Specification and suitable for sand analyses is indicated in *Fig.* 34, and for comparison the standards for this country and for the United States are given in *Table* VII. In both series the sieve number corresponds to the nominal number of meshes per linear inch.

TABLE VII

STANDARD SIEVE SIZES.

Clear opening of mesh (expressed to nearest 0·001 in.).

B.S.	Sieve No.	100	52	25	14	7	$\frac{3}{16}$
	Mesh	0·006	0·012	0·024	0·047	0·095	0·188
U.S.A.	Sieve No.	100	50	30	16	8	4
	Mesh	0·006	0·012	0·023	0·047	0·094	0·187

The importance of using a standard sieve is evident in view of the variation in the diameter of the wires in ordinary sieves made by different manufacturers. The point is not so serious, however, in considering the sizes of coarse aggregate, say from $\frac{3}{8}$ in. upwards, as the diameter of the wire is smaller in relation to the size of the opening as the latter increases. The gauge of coarse aggregate is sometimes specified in terms of the size of stone that passes through a ring of given diameter. This gives a smaller maximum size of material than that passed by a square mesh of the same clear dimension.

SECTION III.—STORAGE AND SITE TREATMENT OF AGGREGATES.

Contamination of Materials on Site.

When clean aggregates are delivered on the site they must not be allowed to become contaminated by depositing them on loam, mud, or grass, or by exposure to dust from coal or stone crushers. If a clean surface is not available the material should be stored in bins or on a prepared surface such as a boarded, concrete, brick, or hardcore floor. A prepared floor also facilitates shovelling. When a hard surface is not provided the lower part of the heap of material is liable to become mixed with dirt or vegetable matter. None of this lower contaminated material should be used, and a fresh supply of aggregate should be obtained before the bottom of the heap is reached ; if this is done the only waste due to contamination will be in the first consignment.

Some protection from crusher or other dust can be afforded by placing the material heaps as far as possible from the source of the dust and on the windward side. When these precautions are insufficient or when the wind blows from the direction of the crushers or screens, the material should be protected by tarpaulins or other suitable covering. The heaps of fine and coarse aggregate should be kept separate, and if they are tipped adjacent to each other they should be divided by a timber partition.

Storage of Materials in Bins.

On large contracts the expense of installing timber or steel bins for aggregates is usually justified, and complete equipment for handling, storing, measuring, and mixing the materials is often bought or hired. The bins can be filled either directly by tipping from the supply wagons or by crane or other handling plant. Alternative typical bin installations are given in *Fig.* 36, in which (*a*) and (*b*) indicate direct feed while (*c*) and (*d*) involve intermediate handling. If the material is supplied in bottom-opening wagons the latter can be brought over the bin on a staging as in (*a*). The bin, if of timber, may be constructed in conjunction with the staging. If the material is delivered in side-tipping wagons (either of standard gauge, contractor's, or jubilee type) or in end-tipping motor-lorries the bin would be erected to one side of the elevated track or road and fed through a chute. When the site is below the level of the adjacent ground a staging can be dispensed with, but measures must be taken to retain or shore up the bank against the surcharge from the loaded vehicles.

If the supply wagons are non-tipping and can conveniently be run in at a high level they can be unloaded by hand, the material being thrown into a chute. If the supply tracks or roads are at the same level as that on which the bin is erected it will be necessary to elevate the material, and this can be done by a grab as in *Fig.* 36(*c*), which shows the arrangement of equipment for unloading from a train of railway wagons by a travelling steam crane. If the shunting engine is not available to bring each wagon alongside the bin when it is required the crane must travel to and fro between the wagon and the bin. An alternative installation is illustrated in *Fig.* 37 where on a riverside contract a stationary long-jib crane fitted with a grab is used in unloading aggregate from barges

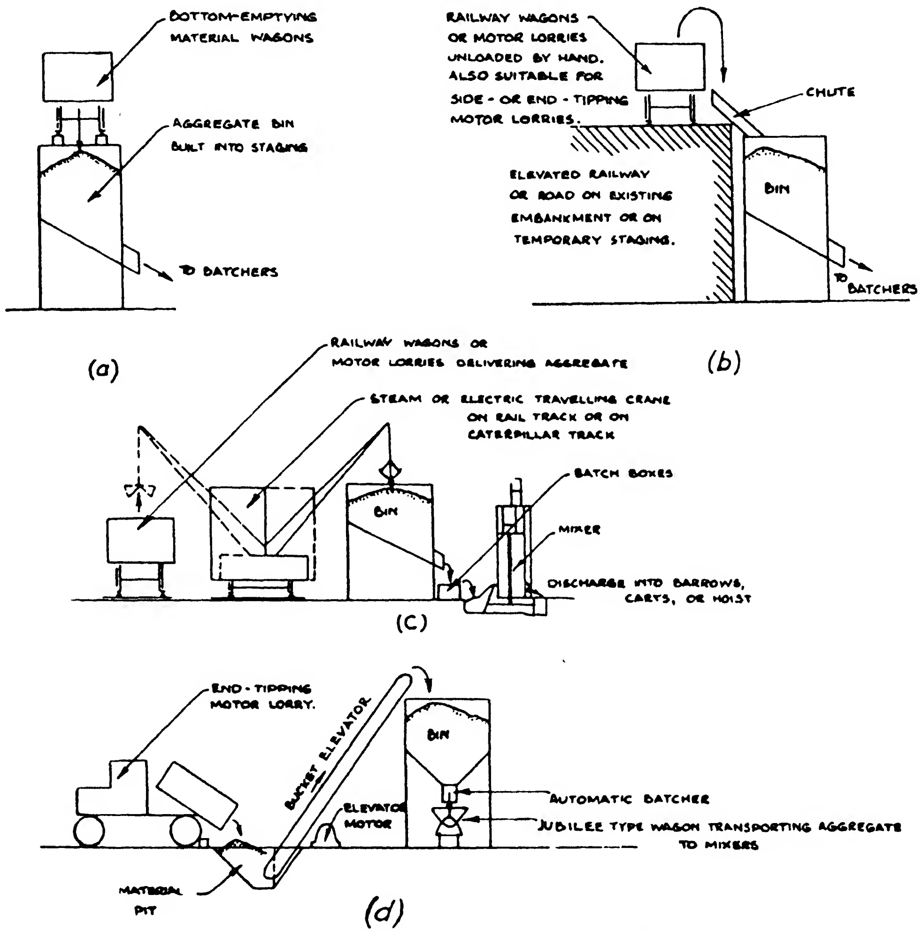


FIG. 36.—EXAMPLES OF BIN INSTALLATIONS.

and depositing it in heaps. The steel bin is provided with an automatic batching plant which is replenished by the crane from the storage heaps. The mixer is installed below the bin and discharges into jubilee wagons. A similar arrangement could be adopted for unloading aggregates from railway wagons.

If the crane in *Fig.* 36(*c*) is only required at intervals to fill the bin, it would be more economical to use a bucket elevator arranged as in *Fig.* 36(*d*), where the material is supplied in end-tipping lorries unloading into a pit at the foot of the elevator. In this case, the automatic batcher discharges a measured

FIG. 37.—AGGREGATE BIN USED IN CONJUNCTION WITH CRANE.

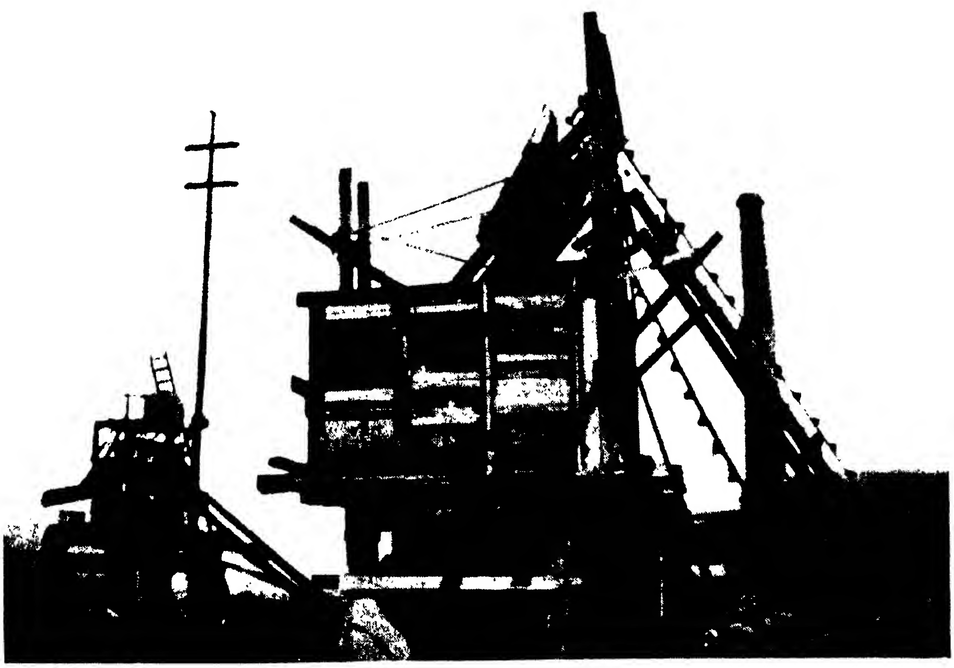


FIG. 38.—LOADING BIN BY ELEVATOR.

amount of material directly into a skip which conveys it to the mixer or mixers at various points on the site. A bucket elevator loading aggregates into a timber bin is shown in *Fig.* 38, a short timber chute being arranged to direct the material from the head of the elevator into the bin. If the problem is one of horizontal travel rather than vertical transport of the aggregates to the bin, a belt conveyor can be used in place of the bucket conveyor. The arrangement incorporating a belt illustrated in *Fig.* 39 may be adopted when, owing to the position of existing buildings, the grab unloading the material cannot discharge directly into the bin. In this instance the grab drops the aggregate into a hopper feeding the belt which conveys the material to the storage bin in the building.

The capacity of the storage and batching plants depends on the size of the contract and study is necessary to co-ordinate the rate of material supply, the

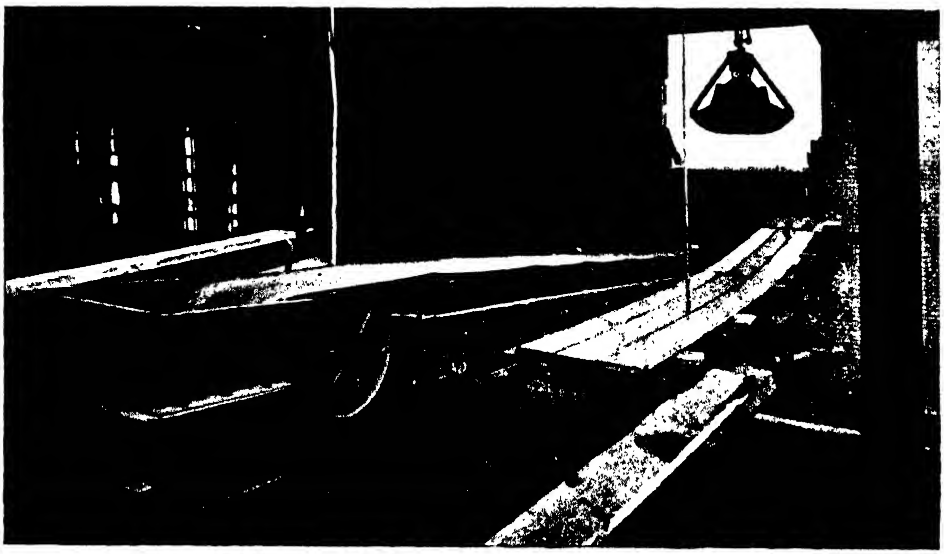


FIG. 39.—BELT CONVEYOR.

bin or other storage capacity, and the rate of consumption. As an example of correlation of supplies and of handling costs, consider a contract involving 15,000 cu. yd. of aggregate which is used at the rate of 100 cu. yd. per 8-hour shift. Each incoming train consists of twenty trucks each containing 20 cu. yd. of sand or stone, that is 400 cu. yd. per train. To maintain the rate of consumption a supply train must arrive in each period of four shifts. The material is unloaded from the trucks by a travelling steam crane fitted with a cubic yard grab, the crane and wagons travelling on parallel tracks. If the bin capacity is 120 cu. yd., 30 per cent. of the material will be deposited directly into the storage bins, the remaining 70 per cent. (280 cu. yd.) being dumped on to stock heaps for transference later to the bins as required.

Upon arrival of the train the crane has therefore to unload six trucks into bins and fourteen trucks on to heaps. Assuming that for the latter operation the time occupied in lowering the grab, grabbing, lifting, swinging, discharging on to heap, and swinging back takes an average of $1\frac{1}{2}$ minutes for 1 cu. yd., the crane can unload one truck in $\frac{20}{1} \times 1\frac{1}{2} = 30$ minutes. It will take longer, say 45 minutes, to unload each truck into the bins, as the crane has to travel backwards and forwards between the trucks and the bins or the train has to be shunted to bring each truck in turn opposite the bins. Thus the total time occupied in unloading a train will be :

6 trucks at 45 minutes each	=	$4\frac{1}{2}$ hours
14 ,, ,, 30 ,, ,,	=	7 ,,
Wasted time, shunting, clearing out corners of trucks by hand, etc., say	=	$4\frac{1}{2}$,,
		16 hours

or two shifts of eight hours.

In the remaining two shifts before the next train arrives the crane has to transfer the contents of fourteen trucks (280 cu. yd.) from the heaps to the bins. Assuming 2 minutes per 1-cu.-yd. grab for this operation, the time taken would be $\frac{280}{1} \times \frac{2 \text{ min.}}{60}$ = say, 10 hours. Allowing for getting-up steam, cleaning, repairs, etc., the crane would be practically fully occupied throughout four shifts in dealing with the contents of one supply train. Thus the crane gang's time would be charged entirely to unloading and handling operations. The cost per 8-hour shift for this gang might be :

Crane driver : 8 hours at 1*s*. 9*d*.	=	14*s*. 0*d*. per shift.
Three labourers clearing wagons, spillings and trimmings ; 24 man-hours at 1*s*. 2*d*.	=	28*s*. 0*d*. ,, ,,
		42*s*. 0*d*. ,, ,,

In four shifts this gang would deal with 400 cu. yd. of aggregate, the handling costs for which would be $\frac{(42 \times 12) \times 4}{400} = 5d.$ per cubic yard, or, say, 6*d*. per ton including supervision and other direct on-costs on labour. To this must be added depreciation and other costs of the crane and the cost of fuel, lubricants, and spares to arrive at a figure comparable to unloading by hand. If the railway terminal charges allow for shunting the wagons on to and away from the siding at the site, no extra allowance is needed for shunting costs. It is important that the crane and the gang should be available to commence unloading immediately upon arrival of the train so as to avoid demurrage charges.

Design of a Timber Bin.

Timber bins can usually be constructed on the site and should be built from stout timbers well braced to resist racking strains due to conveyors or other machines. The timber sheeting should be close fitting to prevent leakage of fine material and the bottom should be sloped to discharge through a central outlet at the bottom of a pyramidical hopper or through a gate in the side. The latter method simplifies the construction of the bottom.

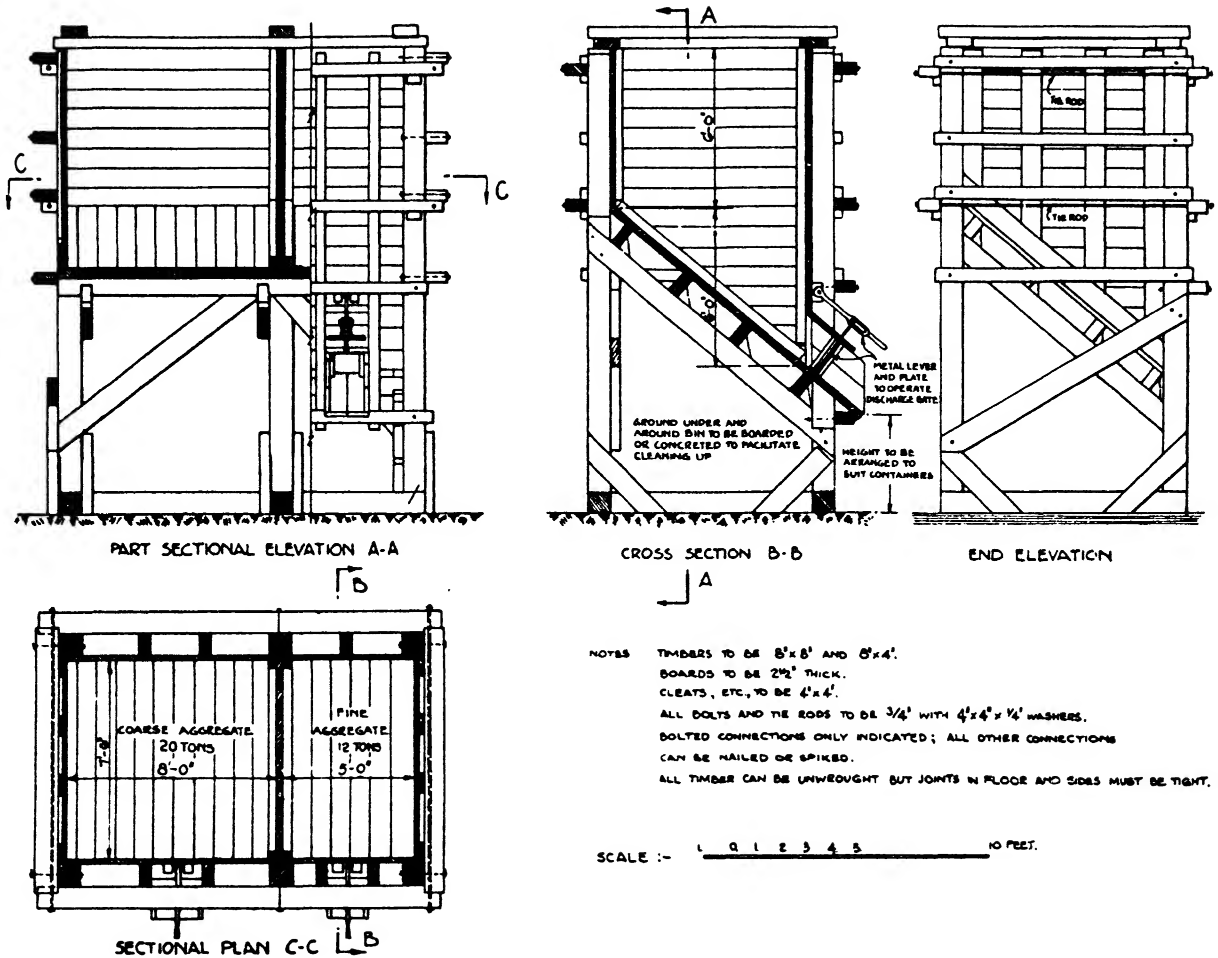


FIG. 40.—CONSTRUCTION OF AGGREGATE BIN.

The bin illustrated in *Fig.* 40 is designed to hold about twenty tons of coarse aggregate and about twelve tons of fine aggregate. The proportions of each compartment are chosen to facilitate discharge and both gates are on one side of the bin. By making the side in which the gate is fixed comparatively short, the bin can be almost completely emptied except for the small amount of dead material in the bottom corners. The bottom boards run along the length of the slope to maintain easier flow when the timbers become worn. Protection against wear can be provided by lining the bottom with corrugated iron, the corrugations being parallel to the direction of flow.

The gates are operated by a hand lever and the discharge is direct into batch measurers. If the height of the lever is above the reach of the workmen, it can be worked from ground level by ropes, levers, or chains. The ground under the discharge chutes and around the bin should be boarded or concreted to facilitate the collection of material overflowing from the batch boxes or spilled while filling the bin. Since ease of emptying the sand-bin depends on the dryness

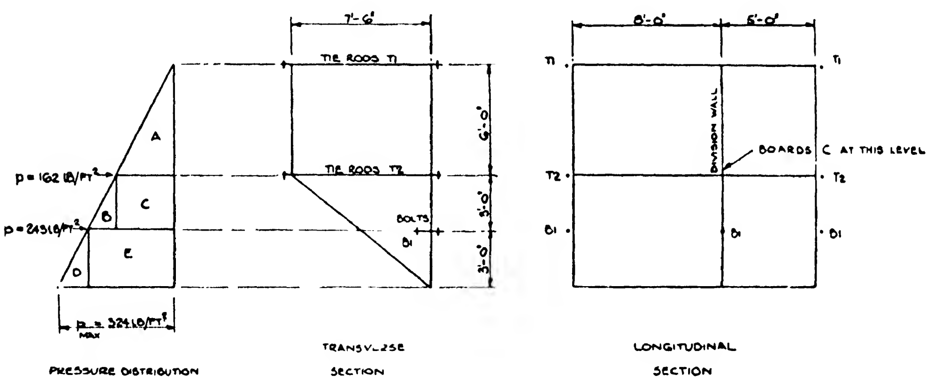


FIG. 41.—PRESSURES ON BIN WALLS.

of the material, a tarpaulin should be available to keep it covered during wet weather in the intervals between successive fillings.

The bin would be filled from the top, top corner braces being omitted and the timbers arranged to present the least hindrance to the material. In addition to stiffening the construction, the top flat timbers provide a narrow walking platform for men trimming the contents. The bin illustrated in *Fig.* 40 is particularly well braced and sturdy to give service on a long contract. Smaller bins would not require to be quite so elaborate, but in designing all such structures attention should be given to the arrangement of the timbers to allow easy assembly and dismantling.

The principal strength calculations for a timber bin of similar design to that shown on *Fig.* 40 concern the tie rods and bolts provided to prevent bursting. The transverse horizontal pressures due to the filling are resisted principally by the tie rods T1 and T2 and the bolts B1 (see *Fig.* 41), although a certain amount is taken on the intermediate frame between the two compartments. Neglect of the full assistance given by this intermediate frame results in the design being well on the safe side.

The intensity of horizontal pressure (p) at any depth (h) from the free surface of the filling for a material like dry gravel or sand can be expressed as $p = 27h$ lb. per square foot (with h in ft.). Thus the maximum pressure when $h = 12$ ft. is $p = 27 \times 12 = 324$ lb. per square foot. The distribution and values of the intermediate pressures are as shown in *Fig.* 41, the total pressure on the front wall of the bin being divided up into the following sections:

	lb.
$A = 162 \times \frac{6}{2} \times 13 =$	6,318
$B = 81 \times \frac{3}{2} \times 13 =$	1,579·5
$C = 162 \times 3 \times 13 =$	6,318
$D = 81 \times \frac{3}{2} \times 13 =$	1,579·5
$E = 243 \times 3 \times 13 =$	9,477
Total pressure $= 324 \times \frac{12}{2} \times 13 =$	25,272

The two tie rods T1 resist

$\frac{1}{3}$ of section $A = \frac{1}{3} \times 6{,}318 = 2{,}106$ lb. $= 1{,}053$ lb. each.

The two tie rods T2 resist

$\frac{2}{3}$ of section $A = \frac{2}{3} \times 6{,}318 = 4{,}212$
$\frac{1}{3}$,, $B = \frac{1}{3} \times 1{,}579\frac{1}{2} = 527$
$\frac{1}{2}$,, $C = \frac{1}{2} \times 6{,}318 = 3{,}159$

7,898 = 3,949 lb. each.

The three bolts B1 take the remainder of the tension, that is

$$25{,}272 - (2{,}106 + 7{,}898) = 15{,}268 \text{ lb.} = 5{,}090 \text{ lb. each.}$$

If, for convenience, all the tie rods have the same diameter, the size required, working to a safe tensile stress of 8 tons per square inch for temporary work, is

$$\text{Area at bottom of threads} = \frac{3{,}948}{8 \times 2{,}240} = 0{\cdot}22 \text{ sq. in.}$$

This will be given by a $\frac{3}{4}$-in. rod. Similarly the size of bolts B1 can be determined from:

$$\text{Area at bottom of threads} = \frac{5{,}090}{8 \times 2{,}240} = 0{\cdot}284 \text{ sq. in.}$$

which is given by a $\frac{3}{4}$-in. bolt. The area of the washers required, adopting a bearing stress on timber of 400 lb. per square inch across the grain, is $\frac{5{,}090}{400} = 12{\cdot}7$ sq. in. Therefore use 4-in. by 4-in. by $\frac{1}{4}$-in. mild steel washers.

Since the width of the bin is only 7 ft. 6 in. compared with its length of 13 ft., the forces on the bolts resisting longitudinal bursting will be considerably less than the forces in the transverse tie rods or in the bolts B1. It is convenient, however, to use bolts of the same size ($\frac{3}{4}$ in.) throughout.

The boarding for the external walls and bottom can be supported at intervals close enough to prevent distortion, calculations therefore being unnecessary, but a calculation for the boarding of the division wall is desirable. Taking as the critical case the boards at the level of the top of the slope, the span L of which is 7 ft. 6 in. and which can be subjected to a horizontal pressure due to about 6 ft. of filling from one side, that is $p = 27 \times 6 = 162$ lb. per square foot. If we calculate for 1-ft. height of boarding, the bending moment will be

$$M = \frac{162 \times 7{\cdot}5^2 \times 12}{8} = 13{,}650 \text{ in.-lb.}$$

Taking a working stress in bending of 1,200 lb. per square inch for timber boarding, the section modulus required is $\frac{13{,}650}{1{,}200} = 11{\cdot}4$ in.^{3} Hence the thickness of boarding required

$$= \sqrt{\frac{6 \times 11{\cdot}4}{12}} = 2{\cdot}4 \text{ in., say } 2\tfrac{1}{2} \text{ in.}$$

The deflection of the boarding can be found from the expression $\frac{5\,PL^3}{384\,EI}$ where I = moment of inertia of section, which with $2\frac{1}{2}$-in. boards is

$$\frac{12 \times 2{\cdot}5^3}{12} = 15{\cdot}625 \text{ in.}^4$$

and P = total pressure on 12-in. board, 7 ft. 6 in. long = $162 \times 7{\cdot}5 = 1{,}215$ lb. If E, the modulus of elasticity, is taken as 1,200,000 lb. per square inch for timber, the maximum deflection is

$$\frac{5 \times 1{,}215 \times 7{\cdot}5^3 \times 12^3}{384 \times 1{,}200{,}000 \times 15{\cdot}625} = 0{\cdot}615 \text{ in.}$$

That is to say, if $2\frac{1}{2}$-in. boarding is used, the calculated deflection under the worst conditions would be nearly $\frac{5}{8}$-in., which for a structure of the nature of an aggregate bin might not be excessive, as it occurs only in one or two boards under certain loading conditions.

The area of end bearing required for the boarding is $\dfrac{\frac{7{\cdot}5}{2} \times 162}{400} = 1{\cdot}52$ sq. in. per foot of height. With the 4-in. by 4-in. bearing strips provided, the area obtained is considerably in excess of this minimum requirement.

The legs supporting the bin are designed as follows. The centre leg at the front of the bin takes the maximum load. The total load on all six legs is

		lb.
Contents:	$20 + 12 = 32$ tons	= 68,500
Timber, etc.:	say 400 cu. ft. at 50 lb.	= 20,000
Superimposed and other incidental loads		= 1,500
	Total	90,000

Assuming that the front row of legs takes five-eighths of the total load and that the centre leg takes 50 per cent. of the load on the front row, the load on this leg is $\frac{1}{2} \times \frac{5}{8} \times 90{,}000 = 28{,}100$ lb.

Assuming a safe load in the direction of the grain of 840 lb. per square inch, which allows a margin for eccentric loading, the required cross-sectional area of the leg is $\frac{28,100}{840} = 33 \cdot 5$ sq. in.

Thus a 6-in. by 6-in. timber would be sufficient if well braced, but a minimum of 8-in. by 8-in. is suggested as a sturdy framework is required to withstand side-racking and other incidental forces not easily calculated.

The safe loading on the ground should be investigated. As shown in *Fig.* 40 the ground-frame is made from 8-in. timbers and has an approximate area of 50 ft. run $\times$ 8 in. $=$ 33 sq. ft. If the load were equally distributed over this area the average ground pressure would be $\frac{90,000}{33} = 28,100$ lb. per square foot, say, $1\frac{1}{4}$ tons per square foot. The ground should therefore be capable of taking a safe pressure of about $1\frac{1}{2}$ to 2 tons per square foot to allow for the pressure under the front edge of the frame being above the average. The pressure is imposed practically at the natural surface of the ground and conditions on most sites would rarely be favourable enough to admit a safe pressure of 2 tons per square foot at this level. In most cases it would therefore be necessary to distribute the pressure over a greater area; for example, by laying a timber apron over the whole of the area occupied by the bin and erecting the ground frame on this apron.

Screening and Washing Aggregates.

Aggregates delivered on the site that are not up to the requirements of the specification regarding cleanliness and grading require treatment before being used. Generally it only pays to treat small quantities on the site and to limit this treatment to the removal of loam or clay by washing and the removal of under-size or over-size particles by screening. If other processes are required it is usually cheaper to have the material properly prepared at the source of supply where more suitable plant is available, or to obtain supplies from another source.

Site screening is most effective when the sand or stone is not damp, and the simplest method for small quantities is to use an inclined screen of the required mesh, the material being thrown in shovelfuls up to the top end of the screen and allowed to run down to the bottom. While passing over the surface of the screen most of the particles smaller than the mesh dimension will pass through. For large quantities a hand or mechanically operated rotary screen is more economical and gives a greater output. A small portable power screen may consist of an inclined mesh, measuring about 2 ft. 9 in. by 1 ft. 9 in. vibrated by a $1\frac{1}{2}$-horse-power petrol engine or electric motor. The screen can be mounted on a steel frame thus raising the discharge height to allow for feeding into barrows, jubilee wagons, bins, or other containers. If working at ground level the screen can be fed by hand but if elevated a bucket or belt conveyor would be necessary to raise the material from ground level to the top of the screen. If fairly dry sand is being screened on a $\frac{1}{4}$-in. or $\frac{3}{8}$-in. mesh, the output of such a screen would be about 5 tons per hour. To maintain this rate at least two men would be required to throw material on to the screen. With damp or loamy sand the heap to prevent accidental mixing. The sand should be allowed to become reasonably dry before use to prevent lumpiness or clogging of the materials in the skip of the mixer.

Washing removes earth and may rid the sand of objectionable proportions of organic matter. To prove that the washing has been efficacious in removing the latter, the caustic soda solution test, described in a later paragraph, should be applied before and after washing.

Another method of washing small volumes of coarse sand is to place a quantity in a close-mesh sieve (say 50 apertures per linear inch) and place it in a tank of water, keeping the sieve in motion. This procedure is not usually suitable for concrete sands which contain a proportion of "fines," but may be adopted for certain surfacing materials.

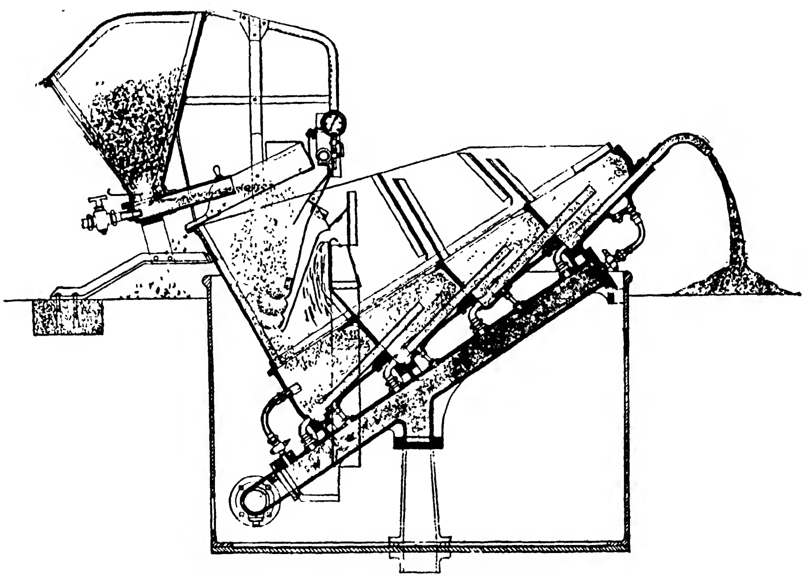


FIG. 44.—SAND WASHING MACHINE.

Quarry dust can be removed from crushed stone by washing, but if the dust is loose and does not adhere to the stones it can be removed more cheaply by allowing the material to fall through a strong current of air which carries away the dust. A fan and casing are required to produce and control the air current, and although suitable apparatus can be purchased for this purpose a satisfactory "blower" can be rigged up on the site if a fan is available. Excessive free dust can be partially removed from small quantities of materials by shovelling the material from one heap to another in a strong wind. The throw of the material should be as far as possible to let the wind play on the material for as long as possible. Allowing the material to fall from a height in a strong wind may also have the same effect. In all methods of dust removal by blowing it is essential that the dust should not settle on clean material, cultivated land, or where it may do harm.

SECTION IV.—CEMENT.

Properties of Cement.

Cement is the most expensive ingredient in concrete and is generally the most reliable, the scientific precision that characterises each stage of the manufacture ensuring a uniformity of quality that would otherwise be lacking. The principal requirement is a cement that will produce a strong, dense, durable concrete with definite setting and hardening characteristics. The chemical composition, the manufacturing processes, and the fineness of grinding determine these properties and no Portland cement should be used which is not guaranteed to comply with the appropriate British Standard Specification. Whereas low-strength concrete can be produced with good cement and poor aggregates, a vastly inferior concrete is produced with good aggregates and a defective cement.

Cements are divided into various kinds dependent on their setting and hardening properties and on their chemical composition. These characteristics determine the use to which a particular kind of cement is put. The most common cements are normal and rapid-hardening Portland cements. High-alumina cement is characterised primarily by its rapid-hardening properties. Some qualities of blastfurnace slag are combined with ordinary Portland cement in the manufacture of Portland blastfurnace cement, the properties and composition of which are set out in the appropriate British Standard Specification.

The terms " setting " and " hardening " applied to cement should be carefully distinguished. The former represents the preliminary phenomenon termed " initial set " in which the liquid or plastic nature of the cement paste in a mortar or concrete begins to disappear ; it represents the beginning of the crystal formation, and the completion of the initial set can be identified by the failure of a pencil point or thumb-nail to scratch the surface. A normal setting cement (originally termed slow setting cement) is one in which the initial set occurs not earlier than half an hour after mixing with water, while with a quick setting cement this stage may be reached in a few minutes. The current British Standard Specification requires initial setting times to be between the following limits: Normal-setting cement: not less than thirty minutes, and not more than ten hours ; Quick-setting cement: not less than five minutes, and not more than thirty minutes.

Ordinary construction requirements do not provide many occasions for the use of quick-setting cements, which should generally be prohibited since the time between mixing and initial set is not sufficient for adequate mixing, transporting, placing, and tamping, unless very special precautions are taken to reduce this period to a few minutes. The initial set of high-alumina cements occurs in 2½ to 4½ hours, but owing to the very considerable decrease in setting time, approaching flash-setting, that occurs when Portland cement is in combination with high-alumina cement, these two materials should not be mixed together in ordinary work.

The " hardening " of cement is the process involving the strengthening of the mortar that commences at the end of the initial set and proceeds first at a fairly rapid rate, slowing down appreciably as time elapses. In sound concrete it has been shown that the hardening process goes on almost indefinitely. The degree of hardening is measured by the compressive strength of the mortar or concrete and the strengths attained by concretes at various ages relative to the strength at the end of one year are approximately in accordance with the following values :

Age	Normal-hardening Portland cement	Rapid-hardening Portland cement	High-alumina cement
9 hours	—	—	20 per cent.
12 ,,	—	—	40 ,,
24 ,,	—	—	65 ,,
3 days	—	50 per cent.	75 ,,
7 ,,	35 per cent.	65 ,,	80 ,,
28 ,,	60 ,,	90 ,,	87½ ,,
3 months	85 ,,	95 ,,	92½ ,,
1 year	100 ,,	100 ,,	100 ,,

These figures are based on laboratory made cubes of 1 : 2 : 4 concrete. The curves given in Chapter VI also illustrate the increase in strength with age. Standard specifications depend on tensile strengths of cement mortar to indicate the hardening properties and soundness of the cement. There are at present no British Standard Specifications for rapid-hardening Portland cement or high-alumina cement ; the former should comply with the B.S.S. for normal Portland cement.

The minimum strength required for normal-hardening Portland cement is 300 lb. per square inch at three days and 375 lb. per square inch at seven days. The tests are made on 1 : 3 (cement : standard sand) mortar and in all cases the seven-day strength must show an increase on the three-day strength. The compressive strength of concrete is considered in a subsequent chapter.

Rapid-Hardening Portland Cement.

The principal advantage to be obtained from the use of rapid-hardening cement is that the shuttering can be struck at an earlier date owing to the high early strength of the concrete. A subsidiary advantage is that the structure is more immune from possible damage due to overloading at an early age. In cases where it is necessary to put the structure into use at an age less than that permissible with normal-hardening Portland cement, the use of rapid-hardening cement may effect the desired end. Rapid-hardening cements may be either Portland cement or aluminous cement.

The chemical composition of rapid-hardening Portland cement is similar to that of normal Portland cement, the rapid-hardening qualities being almost entirely due to the fineness of grinding. The following figures are the average of typical test results of sieve analyses of normal and rapid-hardening Portland cements compared with the requirements of the B.S.S.

	Residue on B.S. Mesh No. 72	Residue on B.S. Mesh No. 170
B.S.S. (minimum)	Not more than 1·00 per cent. by weight	Not more than 10·00 per cent. by weight
Normal Portland cement	0·40 ,, ,, ,,	8·64 ,, ,, ,,
Rapid-hardening Portland cement	0·17 ,, ,, ,,	1·66 ,, ,, ,,

Under normal conditions the tensile strength of rapid-hardening Portland cement at seven days is approximately equal to that of normal-hardening Portland cement at twenty-eight days. Tests show the following to be a common range of strengths of 1 : 3 mortar. The British Standard Specification requirements are also given for comparison.

Age of Specimen	3 days	7 days	28 days
	(lb. per square inch)	(lb. per square inch)	(lb. per square inch)
British Standard Specification	Not less than 300	Not less than 375	Not specified
Normal Portland cement	—	480 to 605	515 to 695
Rapid-hardening Portland cement	505 to 630	530 to 675	595 to 785

The best results cannot be expected from rapid-hardening Portland cement unless coarse sand is used and concreting at low temperatures is avoided. Rapid-hardening Portland cement should always be measured by weight, and since the shrinkage accompanying the setting of concrete made from this cement is greater than with coarser ground cements, there is a greater necessity for adopting the curing methods described in a later chapter.

High-Alumina Cement.

The principal characteristic of aluminous cement (more correctly termed high-alumina cement) is a larger proportion of alumina than is contained in Portland cement. Whereas the latter contains less than 10 per cent. of alumina (Al_2O_3), high-alumina cement may contain 35 to 40 per cent. of this constituent with a consequent decrease in the lime (CaO) content, which may be say 65 per cent. for Portland cement and 38 per cent. for high-alumina cement.

Characteristic analyses of Portland and high-alumina cements give the following percentages of the principal ingredients:

Substance.	Portland cement.	High-alumina cement.
Al_2O_3 (Alumina)	5½	38
CaO (Lime)	64	38
SiO_2	22	5½
Fe_2O_3	2½	12
FeO	—	4½

Most high-alumina cements are more finely ground than specification requirements for Portland cements, but are not always as fine as commercially supplied rapid-hardening Portland cement.

The increased alumina content (the setting agent in cement) leads to rapid-hardening properties, and aluminous cement will harden more rapidly than a rapid-hardening Portland cement and may attain a higher ultimate strength. Final set may occur within an hour of the initial set, that is at 3½ to 5½ hours after mixing.

The London Building Act requirements regarding high-alumina cement can be summarised as follows:

Residue on British Standard Sieve No. 170, not more than 12 per cent. by weight.

Alumina content not less than 35 per cent. by weight.

Ratio of weight of alumina to weight of lime, not less than 0·90.

Initial set, when gauged with 22 per cent. by weight of water, not less than 2 hours nor more than 6 hours.

Final set to be within two hours of initial set.

Tensile strength of 1 : 3 (cement : standard sand) mortar gauged with 8 per cent. of water by weight (of dry materials):

not less than 475 lb. per square inch at 24 hours;
not less than 550 lb. per square inch at 7 days.

The cost of aluminous cement outweighs any advantage to be gained from its high early strength except in such cases as relaying city streets, casting or lengthening piles in situ, and other work where speed is essential and for concrete which may be subject to attack by chemicals against which high-alumina cement is more resistant than Portland cement.

Weight of Cement.

To obtain the correct amount of cement in a concrete mix the cement should be measured by weight. Generally the unit weight of cement in this country is taken as 90 lb. per cubic foot; United States practice takes 94 lb. (or one bag) per cubic foot (in the United States there are twenty-four bags to a ton). These can only be taken as nominal figures as the actual volume of a given weight of cement depends on how it is fed into the container and whether it is shaken down. British Standard Specification No. 648 gives weights of Portland cement varying from 74 to 114 lb. per cubic foot and states that as filled into a gauge box the weight is usually 80 lb. Finely-ground cements bulk more than coarser varieties, and rapid-hardening Portland cement therefore generally weighs less per cubic foot than normal Portland cement. High-alumina cements usually weigh from 80 to 100 lb. per cubic foot.

A test carried out on normal and rapid-hardening Portland cements gave the following values:

Normal hardening Portland cement: One sack (11 sacks to the ton) gave 2·33 cu. ft. $= \dfrac{2{,}240}{11 \times 2 \cdot 33} = 87\frac{1}{2}$ lb. per cubic foot. Rapid-hardening Portland cement: One bag (20 bags to the ton) gave 1·67 cu. ft. $= \dfrac{2{,}240}{20 \times 1 \cdot 67} = 67\frac{1}{2}$ lb. per cubic foot.

When the measure containing the rapid-hardening Portland cement was shaken, the bulk of this material decreased visibly but not sufficiently to approach the weight per cubic foot of normal Portland cement. Although rapid-hardening cement is generally more expensive, it is apparently to the contractor's advantage to use this material (apart from considerations of economy in shuttering) if the mix is specified by volume. This is an unsatisfactory advantage, as can be seen from the following comparisons which are based on the tests mentioned:

Normal-hardening Portland cement:

Cost per ton delivered in jute sacks	40s. 0d.
Allow for return of sacks	2s. 6d.
	42s. 6d. per ton.

$$= \frac{42s.\ 6d.}{11 \times 2 \cdot 33} = 1s.\ 8d. \text{ per cubic foot,}$$

$$\text{or} = \frac{42s.\ 6d. \times 90}{2,240} = 1s.\ 8\tfrac{1}{2}d. \text{ per 90 lb.}$$

Rapid-hardening Portland cement:

Cost per ton delivered in non-returnable paper bags . .	46s. 0d. per ton.

$$= \frac{46s.\ 0d.}{20 \times 1 \cdot 67} = 1s.\ 4\tfrac{1}{2}d. \text{ per cubic foot,}$$

$$\text{or} = \frac{46s.\ 0d. \times 90}{2,240} = 1s.\ 10\tfrac{1}{4}d. \text{ per 90 lb.}$$

Whereas, for equal weights, rapid-hardening Portland cement is more expensive, it can be considerably less costly for equal, non-compacted, volumes. Upon mixing with water the less dense material shrinks to a greater extent but, after making allowance for this, volumetric measurement would give a leaner concrete when using rapid-hardening Portland cement than when using normal Portland cement. Unless the specification gives limiting concrete strengths, volumetric proportions may be dangerous and are likely to result in permeable concrete. Although this is accentuated when using rapid-hardening Portland cements, it is also a factor that must be considered with any type of cement.

Supply and Storage of Cement.

Cement is commonly supplied in jute sacks, four-ply paper bags, wooden barrels, or air-tight steel drums. Barrels or drums are more suitable for long transit and export. Sacks are usually returnable and in cost estimates allowance must be made for hire (usually about 1s. 6d. per sack, which is refunded upon return), cleaning, bundling, and return carriage. Paper bags are not usually returnable but when empty can serve a number of useful purposes on the site.

A paper bag of cement of British manufacture usually contains 1 cwt., a jute sack may contain either 1 cwt. or 2 cwt., and a barrel or drum about 3⅓ cwt. (375 lb.). Packages should be undamaged when delivered.

Cement contained in paper bags is not so susceptible to "air-setting" as material in sacks when stored in the original containers. If the storage place can be kept dry and at a temperature of not less than 40 deg. F., retention of the material in sacks is permissible for a few weeks, but if the cement has to be stored on the site for a longer period it is better to empty the containers and spread the cement on a dry floor in a store. It is best to arrange for delivery of cement to keep pace with consumption, but on remote sites or on foreign contracts storage in bulk is advisable to ensure a reserve against unpunctual deliveries.

Cement stored for long periods is generally adversely affected, but the effect is a reduction in the rate of hardening rather than a decrease in the ultimate strength. Except for some risk of superficial air-setting when cement is stored in jute sacks, there appears to be little difference in the deterioration whether jute sacks or paper bags are used, although storage in wooden barrels gives better results. Finely-ground cements stored in sacks tend to deteriorate more than coarser cements. In this respect normal-hardening Portland cement and high-alumina cements are less affected than rapid-hardening Portland cement. Storage in air-tight drums is accompanied by little or no deterioration after several months if subsequent aeration is avoided.

The cement store should be of weather-tight construction, preferably in timber for freedom from condensation, with a sound wooden or concrete floor raised at least a foot from the ground to ensure that it is damp-proof. Unless well-jointed, timber floors and walls should be protected by tarred felt to prevent wind and rain driving through. Cement stores are best made in sections to facilitate erection, dismantling, and transport from one contract to another. A 6-in. × 2-in. stud framing with $\frac{3}{4}$-in. weatherboarding and with a slightly sloped timber roof covered with good quality proofed felting is the usual construction. With a height of 7 ft. to the lintel timbers a storage shed would cost about 4*d.* per cubic foot constructed and erected. The illustration in *Fig.* 45 shows the general construction of a timber store for housing large quantities of cement. The floor is raised sufficiently high to enable the bags to be unloaded directly from lorries, and roller gravity conveyors are installed to facilitate handling the bags. Wide double doors allow sufficient space for stacking and abstracting the containers.

Cement required for use immediately after delivery to the site may be stored in the open on a raised damp-proof floor so long as it is fully protected by tarpaulins or other weather-resisting covers. Storage under these conditions should be limited to 48 hours. On building contracts the lower stories often provide suitable storage spaces while work is proceeding on upper stories, but precautions must be taken to place the cement away from any rain that may blow in through unglazed window or other openings, and also from water dripping from curing or other operations above.

The storage space required for a given quantity of cement can be calculated from the following data. If spread loose over the floor of a store to a depth of 1 ft., a ton of cement requires about 25 sq. ft. If stored in paper bags laid on their side, the area required is 50 to 60 sq. ft. per ton if laid in a single tier and proportionately less if laid in more than one tier. If the bags are stacked upright, one ton covers about 20 sq. ft. in a single tier. If the bags are stacked in any other manner the space required can be calculated on the basis of 30 to 40 cu. ft. per ton. The minimum area provided should be about 50 per cent. in excess of the minimum stock, to allow space for separating subsequent incoming consignments. If supplies are possible at frequent intervals, it is convenient to store the cement in 10-ton lots, as this amount constitutes a normal load for a large motor lorry.

The cement is usually accompanied by a test certificate. If this is not done samples should be submitted to a laboratory for examination. These samples should be sent at least ten days before the material they represent is to be used. A longer interval is preferable since, in the event of the consignment being condemned, arrangements would have to be made to obtain further supplies, which in turn would require at least a week in which to complete the necessary tests. Most engineers follow the British Standard Specification which requires a test to be made on each 250 tons of material. A sample should consist of twelve

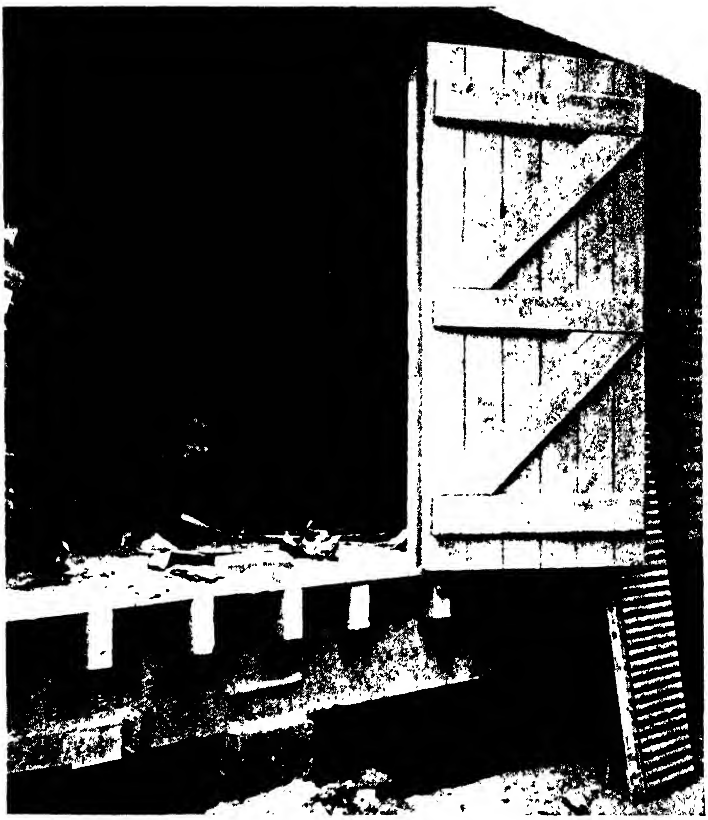


FIG. 45.—CEMENT STORE.

approximately equal portions, each being taken from separate packages. If the cement is delivered in bulk, then the twelve portions should be taken from different parts of the heap. If less than twelve containers are used an amount should be taken from each container to give a combined sample weighing about ten pounds.

For large orders cement manufacturers usually set aside one or more bins of cement for the exclusive use of the particular job. This ensures continuity of supplies and uniformity of quality combined with a more straightforward testing procedure. If successive consignments are not dispatched from a bin for which a test certificate has been issued, a test should be made (or certificate obtained) for each consignment. If a consignment contains more than 250 tons, it is usual to certify each 250 tons. Any cement stored on the site should be labelled to identify it with its appropriate certificate. It is commonly required that these labels should be handed to the resident engineer or clerk of works when the corresponding consignment of cement has been used. This prevents confusion and possible re-use of labels. Sometimes the contractor supplies the resident engineer with a statement relating to each consignment, in which case the statement should indicate the reference mark of, and the amount of, the consignment, the name of the manufacturer, the date of grinding, and a reference to the test certificate.

Consignments should be used in the same sequence as they are delivered. As a rule cement should not be stored on the site longer than three months, and if this time is exceeded it is preferable to have the material re-tested.

Cement that has become lumpy due to storage in damp positions or due to exposure to the weather is generally useless for making concrete and should be removed from the site. Air-set lumps that can be broken down to flour with the fingers are admissible if care is taken to break down all pieces. If such lumps are numerous it is easier to screen them out and discard them. If the proportion of air-set cement is considerable, the fine material after screening should be tested to determine whether it has become defective. Fresh cement that is warm to the touch is not necessarily defective, the rise in temperature being due to the grinding process. The excess heat is generally dissipated during storage or, in any case, when mixed with water and aggregate. The term " hot cement " is often applied to quick-setting cements or to unsound cements. Such cements would be proved defective or unsuitable upon testing in the normal routine.

SECTION V—WATER.

Quality of Water.

The general requirements regarding water for concrete mixing are that it shall be fresh, clean, and free from oil, acid, alkali, loamy, vegetable or organic matter, or other deleterious substances in suspension or solution due to pollution from industrial waste, farmyard or similar drainage. If water is suitable for drinking it is generally suitable for concrete making, but moorland water should be tested before use.

A good test for any water of doubtful quality, is to make concrete cubes with similar materials and water content, one series with the suspected liquid and another with fresh clean water. The strengths and nature of the concrete at 7 or 28 days will give an indication of the detrimental effect (if any) of the proposed water.

Many regulations prohibit the use of sea-water for mixing concrete. Saline waters may produce efflorescence on the concrete surface. If the water contains not more than 5 per cent. of common salt and is otherwise clean, its effect when used with Portland cement may be limited to retarding the setting time by about 10 per cent., which fact should be borne in mind during the curing stage and when estimating the times for striking the shuttering. If moisture can gain access through cracks or pores in the set concrete to the reinforcement, the use of sea-water in mixing the concrete may accelerate corrosion. It is not advisable to use sea-water when mixing aluminous cement concretes.

Generally, water suitable for concrete mixing is satisfactory for other site requirements, an exception being the use of salt water in boilers.

Supply of Water.

In investigating the supply of water for a particular contract it is necessary to consider the probable demand. The quantity of water required for concrete mixing is discussed under "Consistency" in the next chapter, but water for this purpose represents only a small proportion of the total amount consumed. An ample supply of water is required for washing out shuttering, for curing concrete, and possibly for washing aggregates. Supplies for boilers, engine cooling, and workmen's consumption are also required. When the contract involves demolition, a water spray is useful in keeping down dust in dry weather. An allowance of 75 to 100 gallons per cubic yard of concrete is usually sufficient to cover all purposes on concrete construction.

When water is obtained from the mains it is charged for by a rate dependent on the value of the contract or by metering. Where possible the former method is most satisfactory to the contractor, who can then definitely assess the charges when estimating, and because no attempt is made to save water on those operations where copious supplies are essential.

When calculating the value of the contract for the purpose of assessing the water charge, only figures for those trades requiring water for their site work should be given to the water authority. Thus, such trades as excavators, plumbers, steel erectors, carpenters and joiners, steel benders, electricians, heating and lighting should be omitted. Rates on trades using water may be as high as a uniform rate of 20*s.* per £100 of trade work, but in some areas may be 12*s.* 6*d.* per £100 for bricklayers, plasterers, concretors, and terrazzo workers, and 6*s.* 3*d.* per £100 for masons and tilers. When water is paid for by metering the charges vary, and a rate, below that usually charged for domestic supplies, can generally be arranged with the authorities.

When the supply is taken from a main, the latter is tapped at a point arranged with the water authority and iron pipes (usually the contractor's property) are laid to salient parts of the site. From these points, hose pipes, of sufficient length to command the whole area, are connected.

Water is sometimes obtained from artesian wells (if the size and location of the job permits) or, when not taken from public supplies, more commonly from rivers or similar sources, from which it can be pumped into storage tanks. The suction pipe of the pump should be placed so that mud on the river-bed is not stirred up and transmitted through the pipe lines. Similar care should be taken if the water is baled out with buckets and loaded into water wagons which may be either hand, horse, or power-driven.

Storage tanks should be sufficiently elevated to provide an adequate head to transmit the water to the most distant parts of the job and to produce, at the most elevated parts, a sufficiently strong flow to clean the shuttering. In the case of tall structures, where the available head of water from mains is not sufficient to ensure an adequate supply to the upper part of the work, a water tank is sometimes erected at the head of the temporary hoist tower to serve construction needs. The tank is kept full by a pump installed at the foot of the tower or other convenient position

SECTION VI.—SITE TESTS OF MATERIALS.

Apparatus.

The following simple tests can be made on the site to determine the suitability of the materials used in making concrete. The apparatus required includes:

A series of sieves including $\frac{3}{4}$-in., $\frac{3}{8}$-in., and $\frac{3}{16}$-in. clear mesh and standard sieves Nos. 7, 14, 25, 52, 100, or equivalent.

A spring balance or other weighing device capable of measuring accurately to $\frac{1}{10}$ oz.;

A graduated cylinder about $1\frac{1}{2}$ in. in diameter;

A measuring vessel about 3 in. in diameter;

A funnel;

A test tube, glass bottle, or similar container;

A wire probe; and

A glass plate about 3 in. square.

In carrying out the tests as high a degree of accuracy as possible should be aimed at, and all apparatus and the work-bench upon which the tests are made should be scrupulously clean.

By these tests any major defect in the cement or aggregates is detected before they are used; although the quality of the materials may have been satisfactory when dispatched, contamination may sometimes occur or unsoundness may develop during transit to, or handling and storage at, the site.

Determination of Grading of Aggregate.

A known weight of the material is taken and passed through successive sieves starting from the sieve with the larger mesh. From the weight retained on each sieve the weight passing each sieve can be determined and expressed as a percentage of the original weight. The weight of each sample should be about 10 lb. for the coarse aggregate.

It is sufficient if the coarse aggregate is passed through sieves having clear mesh dimensions of $\frac{3}{4}$ in., $\frac{3}{8}$ in., and $\frac{3}{16}$ in., unless the specification allows a larger gauge than $\frac{3}{4}$ in. For ordinary coarse aggregate of specified gauge $\frac{3}{16}$ in. to $\frac{3}{4}$ in., the amount passing the $\frac{3}{16}$-in. sieve and the amount retained on the $\frac{3}{4}$-in. sieve should both be nil; in commercial aggregates there may be traces of oversize and undersize pieces.

The fine aggregate should be passed through sieves having clear apertures of approximately $\frac{3}{16}$ in., 0·095 in., 0·047 in., 0·024 in., 0·012 in. and 0·006 in. If all these sieves are not available, the first, third, and fifth sizes may be used; British and U.S.A. standard sieves have these sizes.

A convenient method of computing the grading percentages is given in the following example. The result of passing a 4-lb. sample of a given sand through successive sieves, commencing with the largest, is as follows:

Size of aperture (in.)	0·188	0·095	0·47	0·24	0·012	0·006
Weight retained (lb.)	nil	0·40	0·48	0·36	1·96	0·80
Hence total weight greater than aperture of particular sieve (lb.)	nil	0·40	0·88	1·24	3·20	4·00
Percentage of 4 lb. greater than aperture of particular sieve	nil	10·0	22·0	31·0	80·0	100
Hence percentage of total passing any gven sieve	100	90	78	69	20	nil

This grading should be compared with the specified grading, preferably by plotting the corresponding curves similar to those on *Fig.* 34.

Voids Test.

This test, which is often used in connection with proportioning a concrete mixture as described later, and which is applicable to fine or coarse aggregates, is carried out as follows. Pour a known volume (V_1) of the aggregate into a

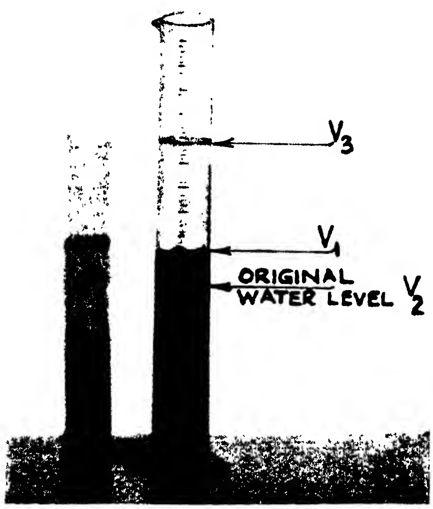


FIG. 46.—DETERMINATION OF VOIDS.

graduated cylinder or similar measuring vessel containing a known volume of water (V_2). Read off the raised level of the water, that is the combined volume of material and water (V_3), as indicated on *Fig.* 46. The percentage of voids in the material is then given by

$$V_0 = 100\left\{1 - \frac{V_3 - V_2}{V_1}\right\}$$

To measure the voids in a sample of fine aggregate, a glass graduated cylinder or bottle of not less than $1\frac{1}{2}$ in. in diameter should be used, but for coarse aggregate a much wider vessel should be employed. For the latter about 500 c.c. of water and stone should be used (i.e. $V_1 = V_2 = 500$ c.c.) and for sand about 100 c.c. should be used (i.e. $V_1 = V_2 = 100$ c.c.). The material should be poured slowly into the water so that only a minimum amount of air is carried down with it into the water. This method is preferable to the alternative of placing the material in the vessel first and adding water, as among the particles much air is trapped that cannot be effectively released by merely probing and shaking. The immersed stone should be probed with a wire probe, however, to release the few air bubbles that may have been carried down with the material.

A given sample of crushed stone was tested for voids with the following results:

Volume of material	600 c.c. $= V_1$
Volume of water	400 c.c. $= V_2$
Combined volumes	750 c.c. $= V_3$

$$\text{Percentage of voids} = 100\left(1 - \frac{750 - 400}{600}\right) = 42.$$

The voids test is susceptible to a number of sources of error, and if the test is to be of any value care must be taken to eliminate or reduce these potential inaccuracies. Several samples from different parts of the consignment of aggregate should be tested, each sample being obtained by "quartering" as large a volume of material as convenient. The error due to "surface voids" may also be high unless a large vessel of such proportions that the surface/volume ratio is a minimum is used. For example, if a cylindrical measure is employed the depth of the material in the vessel should be about twice the diameter for one-tenth of a cubic foot of aggregate. The volume of material, and therefore the size of the vessel, should be as large as possible when testing coarse aggregate.

Determination of Porosity of Aggregates.

To determine the absorption or porosity of a given coarse aggregate, take a sample weighing not less than 1 cwt. and dry it thoroughly over a fire or stove or by spreading it thinly on a clean surface exposed to the sun. Carefully weigh the dried sample when cool and then immerse in water for twenty-four hours. Drain off the surplus water (taking care that the smaller particles are not washed away) and weigh. The difference in the weight when dry and when soaked represents approximately the amount of water absorbed by the sample and can be expressed as a percentage of the dry weight for the purpose of comparing with the specification limitations. This information may also be required for determining the amount of mixing water (see Chapter V).

Determination of Wetness of Aggregates.

To weigh a sample of the aggregate as used and then to dry the sample and weigh it again, does not lead to very accurate results, especially if the moisture content is only slight. When information with regard to the wetness is required for the purpose of assessing the correct amount of mixing water the following method is preferable.

Take two or three samples of the fine and coarse aggregates. Dry one sample of each material and mix with cement in the right proportions and with the correct amount of water. Measure the slump as described in Chapter V. The other samples in their natural state should also be mixed with cement in proportions identical with the first sample but with different amounts of water each less than the correct amount. By making a slump test on each of the resultant concretes, the amount of water that must be used with the wet aggregates to give the same slump as with the dry aggregates can be determined. The difference between this amount of water and the correct amount represents the quantity of free water contained in the aggregates which must be allowed for.

Determination of Bulking of Sand.

Since the amount of moisture contained in fine aggregate considerably affects the volume of the material, when measuring the latter by volume for proportioning of mixes (see Chapter V) it is essential to know what actual quantity of sand is contained in a given amount of damp material. A " Rapid Sand Tester " is best used for this purpose. This patented apparatus comprises a canister and a calibrated scale, from which the correct amount of sand, of any moistness, to give a required volume of dry sand can be read off immediately. With this apparatus there is no need to dry the sand or to ascertain its water content before being able to correct the proportions of a concrete or mortar to allow for the bulking of the sand due to moisture content. The apparatus is particularly useful where large quantities of sand and fine aggregate are usually stored in the open and subject to variations in volume due to different moisture contents in wet and dry weather, or due to variable periods of storage after dredging. The uniformity of proportioning the apparatus makes possible results in more uniform strengths, and economy in cement.

Determination of Loam Content of Fine Aggregate.

Place some sand in a glass vessel (test tube, bottle, or measuring cylinder) until it is about half full, and fill to about three-quarters with water. Shake vigorously and allow the contents to settle for about an hour, when the water above the sand should be quite clear. An examination will show that the loam, silt, and clay have settled on the top of the sand as a clearly defined sediment and the relative quantities can be approximately obtained by inspection. If a more accurate measurement is required, a graduated cylinder should be used. Sand should be placed in the cylinder up to the 100-c.c. mark and water added up to the 150-c.c. mark. After shaking and allowing to settle, the number of cubic centimetres of silt will correspond to the percentage of this material in the sample under test.

The decantation test constitutes a more accurate determination of the actual proportion of silt, loam, and soluble material. A quantity of damp sand, about 1 lb., is dried by heating to a temperature not exceeding 110 deg. C. (230 deg. F.). A known weight, W_1, of the dried material is placed in a dish about 9 in. in diameter and 4 in. deep and sufficient water is added to cover the contents. The dish is agitated for a quarter of a minute and then allowed to settle for a similar period, after which the water is carefully poured off without losing any sand. It is advisable to pour the water through a No. 200 British Standard sieve and to return to the dish any material retained on it. Water is again added to the sand, the dish is shaken and the water poured off, this cycle being repeated until the wash water is quite clear. The sand remaining in the dish is then dried at a temperature not above 110 deg. C. and weighed (W_2). The percentage of silt, etc., in the sample is then given by $\frac{100(W_1 - W_2)}{W_1}$

Detection of Organic Impurities in Fine Aggregate.

A quantity of sand is placed in about an equal volume of 3 per cent. caustic soda solution (sodium hydroxide = NaOH). Shake and allow to settle for 24 hours. Then examine the colour of the liquid above the sand. If this is quite clear or pale yellow, the sand is tolerably free from organic impurities. If the liquid is dark yellow, the sample of sand should be washed and again tested. If now the liquid is pale yellow the consignment represented by the sample would be improved by washing. If the dark yellow persists or if in the original test the liquid assumes a brown or darker tinge, the amount of organic impurity is considerable and the expense of satisfactory treatment may not be worth while. The material should be replaced by a sand from another source. In doubtful cases, trial concrete cubes should be made and tested.

A standard colour solution can be prepared by adding 2½ c.c. of a 2-per cent. solution of tannic acid in 10 per cent. alcohol to 97½ c.c. of a 3-per cent. sodium hydroxide solution. This mixture should be vigorously shaken and allowed to stand for twenty-four hours. Any sand test showing a liquid not darker than the standard solution can be considered as satisfactory as regards organic impurities.

Detection of Loam and Dust in Coarse Aggregate.

There is not usually much free loam or clay in a sample of commercially supplied coarse aggregate. If this admixture is in the form of small particles its presence could be detected by visual inspection and could probably be removed by screening. The presence of clay or loam adhering to the pieces of stone is more detrimental to the strength of the concrete and this can also usually be detected by inspection. It is clearly shown in hardened concrete by looseness of the stones in the mortar.

The presence of loose dust will be detected in the grading test, but only examination of pieces of stone can detect dust adhering to the particles. Dust is not uncommon in crushed stone and can be effectively removed by washing. Defective concrete due to failure of the stone to adhere to the mortar results if excessive dust adhering to the particles is allowed to pass uncorrected.

Tests for Cement.

The tests stipulated in the British Standard Specification for Portland Cement are usually carried out at the works or at an independent laboratory, a certificate of the results being forwarded to the interested parties. The additional tests to be described here can be made on the site in special circumstances, and it is not suggested that they should invariably be made, as under proper transport and storage conditions the cement should not suffer deterioration.

A periodical routine application of test (*a*) is advisable if cement is stored for long periods or in bulk on the site. If concrete shows any signs of defectiveness for no apparent reason and the aggregates prove satisfactory, tests (*b*) and (*c*) should be applied to prove the soundness of the cement. When the cement is suspected of being seriously defective, samples should be submitted to a qualified tester for a thorough examination and test.

Test (*a*).—Make one or two pats of neat cement gauged to a plastic state with clean water. The pats, which should not exceed $\frac{3}{8}$ in. in thickness, should be placed on a clean plate of glass and after initial set should be placed in a vessel of water. Having been kept immersed for seven days the pats should be examined. If the cement is sound there should be no evidence of cracking or buckling. When broken the sections should be of uniform colour and hardness and without any indication of surface flaking.

Test (*b*).—Pats of neat cement should be made and should be kept for four hours in a moist atmosphere, for instance, over a bowl of water in a warm room. At the end of this time, the pats should be placed in a vessel containing water at 115 deg. F. (comfortably warm to the touch). After immersion for twenty hours the pats must show no signs of cracking or buckling.

Test (*c*).—Take about 1 lb. of cement with just sufficient water to make a paste and place it on a piece of glass. For a period of four or five hours test at intervals with a pencil point or other similar object. The point should not be pressed into the specimen but allowed to rest vertically on the surface and penetrate under its own weight. To obtain comparable results the same pencil should be used each time on one specimen and should be fairly heavy. For the first hour, tests can be made at intervals of ten minutes to determine the time of initial set. The tests can then be made each half-hour but with a heavier pointed instrument to give an indication of the rate of hardening. The slab should then be set aside under cover for two days when it should be immersed in cold water ; heat this to boiling point and keep it boiling for four hours. If the specimen remains free from cracks or other defects, the material can be considered as sound.

Throughout these tests it is essential that the cement specimen when not immersed should be protected from draughts and extremes of temperature.

CHAPTER III

REINFORCEMENT

SECTION I.—SUPPLY OF STEEL.

Quality of Material.

Reinforcement may be in the form of rolled mild steel bars (either plain or deformed), cold-drawn mild steel wire, twisted bars (either single or double), or expanded metal. In work of large dimensions as in heavy foundations, old rails of various sections or colliery winding ropes are sometimes used when available. In some designs rolled sections, joists, channels, or angles are used.

Rolled mild steel bars of round section are the most usual, although when they are available square bars are occasionally used. Rolled mild steel supplied for reinforcement in most work is required to be in accordance with the current British Standard Specification. The main provisions of the present specification are an ultimate tensile strength of 28 to 33 tons per square inch and an elongation not greater than 20 per cent. on a test piece eight diameters in length. Without fracturing, the bars should be capable of being bent double when cold to a radius not greater than 1½ diameters if the bar is over 1 in. in diameter, or to a radius equal to the diameter if it is 1 in. or less in diameter.

Cold-drawn mild-steel wire, generally supplied in sheets or rolls of electrically-welded mesh, is widely used as reinforcement in suspended floors, ground floors, foundation rafts, and roads, and in wrappings for concrete encasing steel beams and columns.

The essential difference between cold-drawn and hot-rolled mild steel is the higher elastic limit and ultimate tensile strength of the former. The current British Standard Specification for hard-drawn steel wire for reinforcement sets the limits of the ultimate strength as 37 and 42 tons per square inch, and prescribes as the bending test that the wire, without fracture, shall be bent cold to a radius equal to the diameter of the wire through an angle of 90 deg., back through 180 deg., and then back to the original position.

Expanded metal is used as reinforcement for horizontal and vertical slab construction, pipes and culverts, and for wrapping concrete-encased structural steelwork. If in accordance with British Standard Specification it will be manufactured from mild steel having an ultimate tensile strength within the range of 26 to 32 tons per square inch.

"Isteg" bars or similar material consisting of twisted twin mild steel bars are manufactured by clutching the two bars together at both ends at a fixed distance apart, and then twisting together cold, and consequently stretching. The cold working produces an increase in the yield point, the minimum value of the latter being 58,500 lb. per square inch compared with 32,000 lb. per square inch for ordinary rolled mild steel. This increase allows a working tensile stress 50 per cent. higher than for rolled mild steel to be adopted with a consequent saving of one-third in the weight of the steel required.

Whichever of the foregoing materials is used, the mills or other suppliers will usually give test certificates for each consignment of material. Bars intended for principal reinforcement in beams, columns, or similar work of restricted dimensions should be straight and must be out of winding. Bars, wire, or other material used for reinforcement in any work should be free from surface defects such as pitting due to corrosion and, although moderate superficial rusting is not objectionable, the bars should be free from loose rust, scale, paint, oil, acid, or loam. Vigorous wire brushing will remove mill scale and rust scales due to exposure, and any adhering earth. Improvised reinforcement such as old wire ropes, old structural sections, or rails are particularly liable to coverings of oil, grease, or paint. Burning by blow-lamp or by passing through a fire will usually effectively remove these coatings.

Size and Length of Bars.

Unless the contractor is preparing his own reinforcement details and steel lists the lengths and diameters of the bars required would be specified by the designers, but the following remarks may be used as a guide when the site engineers have some control over the supplies of reinforcement.

The general sizes of round bars are $\frac{1}{4}$ in. to $\frac{5}{8}$ in. diameter advancing by sixteenths of an inch, and from $\frac{5}{8}$ in. to $1\frac{1}{2}$ in. advancing by eighths of an inch. Bars as small as $\frac{1}{8}$ in. and up to 2 in. in diameter are permitted by most regulations, and even larger bars are sometimes used. Bars less than $\frac{1}{4}$ in. in diameter are frequently used in manufactured mesh reinforcement where wires of small diameters are closely spaced. In these cases the diameters of the bars usually conform to standard wire gauges. Mild steel bars of $\frac{1}{8}$ in. and $\frac{3}{16}$ in. diameter are used as binding in columns of small section, posts, lintels, and special construction.

Bars greater than $1\frac{1}{2}$ in. in diameter are only common in large girders, counterforts, or heavy foundations. For normal building work the limits of $\frac{1}{4}$ in. to $1\frac{1}{2}$ in. are the most economical. Loose bars of very small diameter, in addition to being more expensive, increase the number required, resulting in more pieces to be bent and fixed. On the other hand, very large bars may require special rollings and the provision of more powerful bending machines, and are only justified when the tonnage is large.

The longest bar normally obtainable is 40 ft., but the maximum lengths adopted should bear some relation to the diameter to avoid whippiness during handling. For this reason bars $\frac{5}{16}$ in. in diameter and less should not exceed 20 ft. in length ; $\frac{3}{8}$-in. and $\frac{1}{2}$-in. bars should not exceed 25 to 30 ft. ; $\frac{5}{8}$-in. bars should not exceed 40 ft. ; larger diameter bars can exceed this figure (say up to 80 ft. for $1\frac{1}{2}$-in. bars) if the circumstances justify their use. Bars of small diameter can be conveniently handled in longer lengths than those mentioned if several are bundled together, but bars $\frac{3}{8}$ in. or $\frac{5}{16}$ in. in diameter erected in vertical positions should not project more than 10 ft. or 12 ft. upwards unless means, as discussed later, are taken to avoid distortion.

Within these limiting lengths, bars should be as long as possible for economy, thereby avoiding lapping and consequent duplication of bars and cutting charges. Short bars, that is less than 5 ft., should be avoided where possible owing to the increased cost. To aid easy identification of bars on the site and to avoid unnecessary sorting, as few different diameters as possible and a minimum number of different lengths of each diameter should be specified. For the same reason, the lengths should be in multiples of 3 in.

The question of bar lengths is essentially related to the available transport facilities and facilities for handling them on the site. Work on restricted sites, as within existing buildings, or on jobs difficult of access, as in underpinning, may make it imperative to keep the length of the bars abnormally low, and this matter should be given attention in the preliminary stages of the work and before detail designs are prepared.

Bars transported by road will be limited in length by the capacity of the lorries available ; although not usual practice, bars up to 40 ft. long can be carried on road vehicles. Since it is the overall length of the bar that controls the lengths ordered, shortening of over-length bars can sometimes be effected by bending or forming hooks on the bars in accordance with the bending schedule before they are dispatched to the site.

Railway transport of bars not more than 18 ft. long does not present any particular problem either in freight charges or obtaining wagons. Bars from 18 to 24 ft. require special wagons, but lots of not less than two tons are not subject to delays or increased freight charges. Special wagons are required for bars over 24 ft. and especially over 42 ft., and unless the consignment consists of four tons or more it is subject to what is in effect a higher tonnage rate and may be delayed while awaiting the necessary wagons. The increased transport rate for consignments of long bars of less than 4 tons weight is somewhat as shown in the following example : A 2-ton load of bars not exceeding 24 ft. long is transported a certain distance for 20*s*. a ton. A consignment of the same weight but involving bars over 24 ft. and not exceeding 42 ft. long would be charged as 3 tons, that is the freight charges would be increased to 30*s*. a ton. If the bars exceeded 42 ft. and were less than 60 ft. in length, the charge would be equivalent to 40*s*. a ton. Bars of over 40 ft. long are also subject to an increased charge on the basic purchase rate.

Methods of Supply.

Reinforcement may be delivered to the site in one of several forms and may come either from stock or directly from the rolling mills. If the bars are supplied in stock lengths from the stockyards, they require to be cut on the site to the lengths specified on the descriptive steel lists. This method is usually expensive as there may be a fair amount of waste in cutting the lengths supplied, but avoidance of delay due to later delivery of bars supplied from the mills may offset the extra cost of obtaining bars from stock, cutting them on the site, and the consequent waste; another advantage is that the steel can be sent to the job before or while the detail drawings and steel lists are being prepared.

An alternative method, and one that is most frequently adopted, is to order the steel in the precise lengths given on the steel lists. In urgent cases this is supplied from stock, or in normal cases, and more cheaply, directly from the mills. This necessitates the descriptive lists being available before ordering the steel and is only efficient if the designs can be prepared sufficiently far in advance to prevent holding up the work on the site through lack of bars.

In either of the foregoing methods, small diameter bars, say $\frac{5}{16}$ in. and under, which may be required in lengths of 2 ft. or 3 ft. for column and beam binders, present a problem owing to the increased costs of short lengths. It is therefore often better to order these bars either in maximum stock lengths or in random long lengths and cut them into short lengths as required on the site.

A third method, and one that commends itself on congested sites, is to have all the bars delivered bent to the required shape, with cages ready assembled, but this again necessitates the completion of the final drawings for each section of the work before any steel can be ordered. An advantage in this method is that when long bars are involved they may be bent in more convenient overall lengths for transport. Bent bars of small diameter may be distorted during transit and subsequent handling, but if they are bundled together they are less likely to be damaged. It is in any case better that bars sent bent to the site should be bundled together and labelled in accordance with items in the steel lists and on the drawings. As bundling and labelling are usually included in the bending and delivery charges, the practice of supplying bent bars to the site has much to commend it from the point of view of the engineers in charge of the construction.

Coiled Material.

Bars of $\frac{5}{16}$ in. diameter and less can be supplied more cheaply in rolls than in straight lengths, but to their first cost must be added the cost of straightening prior to bending to the required shapes. The straightening can be done by hammering out the kinks on a wooden bench. By this means the material can be straightened sufficiently for use as distribution bars in slabs, reinforcement in panel walls, or, if in short lengths, for bending into binders for columns and beams.

If the supply and use of coiled material are to be continuous throughout a large contract, the provision of a straightening machine is advisable. The principle of such a machine is shown diagrammatically in *Fig.* 47. A reel or drum carries the coil of material, and the bar as it unwinds is passed through groups of rollers, each consisting of three grooved rollers. There may be three or five sets of rollers arranged in vertical and horizontal planes alternately. The machine is operated by hand or by motor, the rotation being applied to one of the pulley axles through suitable gearing. A cutter is frequently incorporated with the machine, enabling any length of straightened bar to be cropped off.

A similar form of bar straightener may be attached to a bending machine, and as shown in *Fig.* 48 consists of a group of four staggered rollers followed by

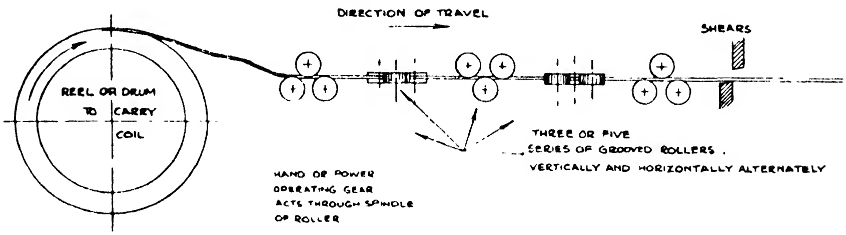


FIG. 47.—DIAGRAMMATIC ARRANGEMENT OF STRAIGHTENING MACHINE.

a pair in alignment. This attachment can be used to straighten bars supplied in short lengths as well as in coils and will take bars up to $\frac{3}{4}$ in. in diameter.

Material supplied in coils is very suitable for the manufacture of helicals for piles and octagonal columns. The wire does not require preliminary straightening as it can be run directly from the coil on to the mandrel of the helical-forming machine as shown in *Fig.* 49, which illustrates helicals being made for pile skeletons. For the normal spacing of helical binding, that is about 3-in. pitch, it is sufficient to wind the material closely on the mandrel as illus-

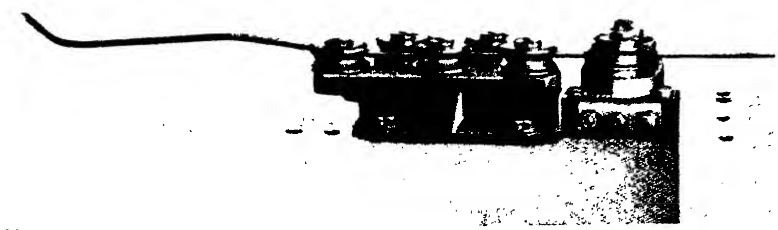


FIG. 48.—BAR STRAIGHTENER ATTACHED TO BENDER.

trated. When it is fixed to the main longitudinal rods of the skeleton the helical can be stretched to give the required pitch. It is best to chalk-mark one of the longitudinal bars to indicate the spacing.

The principle of continuous helical binding may be successfully extended to cylindrical tanks of 20 ft. diameter using coiled material of $\frac{3}{8}$ or $\frac{5}{16}$ in. diameter. The bar is wound around the circumference, using the ring of vertical bars as a template and support. The circumferential bars are fixed with tying wire or by welding at each intersection with the verticals, thus preserving the diameter and pitch. This method, which is as effective as a number of loose bars bent to a given radius, is recommended on the score of economy, as laps and hooks are reduced to a minimum and separate bending costs are avoided. Offset against this saving is the cost of extra supervision while placing the reinforcement and the increased cost of the small-diameter coiled bars; larger (and cheaper) bars could be used at a wider spacing if machine bending of separate short bars were adopted as in the more conventional designs.

FIG. 49.—FORMING HELICALS FROM COILED MATERIAL.

Mesh reinforcement of the lighter gauges may be obtained in coils or rolls. In work extending over small areas the material is unrolled sufficiently to enable small sheets of specified sizes to be cut from it and carried to the work. For large areas of slabs, either on the ground or suspended, when one or more complete rolls will be used in the slab, the material is placed on the shuttering, or on the blinding layer in the case of slabs on the ground, and unrolled in advance of the concreting operation.

Cutting.

Delivery of the bars in the required lengths avoids the provision of cutting machines on the site, but a pair of hand shears or bolt croppers is necessary on jobs of any size to make occasional adjustments in length to small bars during the progress of the work, or to cut bars for binders if these have been supplied in stock or random lengths. When large bars projecting from concrete already hardened are found to be incorrectly placed and are in such a position that a cutting machine cannot be brought up to the work, they can be cut with a flame or by a hack-saw. The latter method is laborious and costly and is preferably avoided. If it is at all possible, it is better, instead of cutting, to bend such bars so that they will be incorporated in the body of concrete yet to be placed. In bending, care should be taken not to strain the fresh concrete in which the bars are embedded. If an acetylene flame is available the labour of cutting large bars in position may be materially reduced, although it would not be economical to provide a flame-cutting apparatus for this purpose only.

If bars are supplied in stock lengths and require cutting on the site, provision for marking-out preparatory to cutting must be made. For this purpose a bench, say 30 ft. long, 3 ft. wide, and waist high should be provided adjacent to the cutting machines. The bench should be marked in feet and inches throughout its length, and at the zero end a metal stop should be fixed to enable the bars to be rapidly measured and marked off. Heavy bars and bars longer than

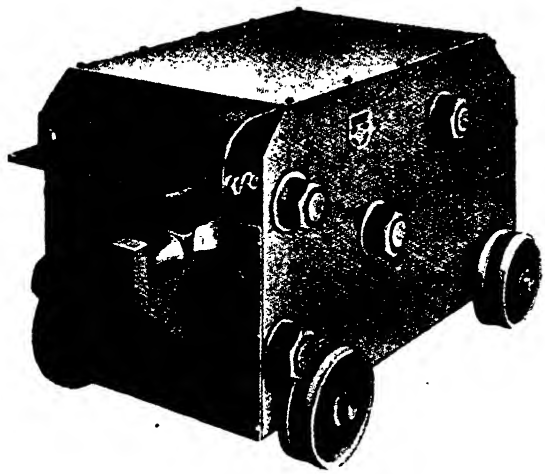


Fig. 50.—Portable Bar Cutting Machine.

the bench can be marked while lying on the ground, using a linen or steel tape for the longer dimensions.

The size of bar to be cut determines the type of cutter required, and for large bars the quantity to be dealt with also affects the choice of apparatus. Bars up to ⅜ in. or even ½ in. diameter can be cut by portable hand shears. Larger bars require a simple hand-lever machine securely bolted down to a base ; this machine can often be conveniently fixed to the end of the marking-out bench.

Bars 1 in. and upwards in diameter require a power-operated shearing machine, which can only be profitably installed if several tons of such bars are to be cut. Power-cutters are operated either hydraulically or, most commonly, electrically, and are usually fixed in position. An electrically-operated machine that can be conveniently moved about the site is illustrated in *Fig.* 50. The machine, with the motor combined, is mounted on four wheels and for bars up to $1\frac{1}{4}$-in. diameter may weigh $7\frac{1}{2}$ cwt. complete. Machines to cut bars up to 2 in. diameter may weigh up to 15 cwt. In some cases cutters are combined with power benders, the power unit for the bending operation also operating the shears. An example is illustrated in *Fig.* 51. With this machine the operator can cut and bend bars without altering his position.

The relative speed and economy of various types of cutting devices are difficult to assess, as the major factor is the amount of labour required for handling the bars up to the cutters and transferring them from the cutters to stock or to the benders. Thus the rate of feeding the machines largely determines the rate of cutting, as the time required to make the cut is approximately the same for all sizes and types of shearing machines. Obviously such cutting methods as hack-saw, cold chisel, or flame are much less speedy and can only be considered as emergency methods. The cost of providing, installing, and operating power

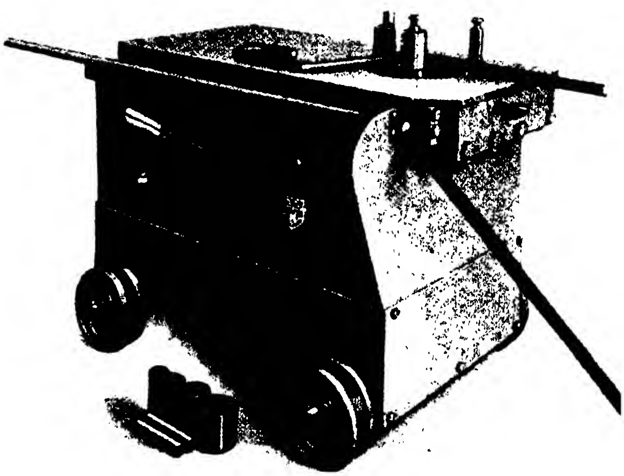


FIG. 51.—COMBINED CUTTER AND BENDER

cutters is necessarily higher than that for the more simple hand-operated shears, but if power machines are working to full capacity on large bars the tonnage dealt with is higher in a given time.

Portable cutters of the hand-shearing type can be carried about the job and used wherever convenient, but with the machines required for larger bars it is necessary to take the bars to the machine. The position of the latter should therefore be such that handling both before and after cutting is reduced to a minimum. Turning the bars around lengthwise should be avoided, especially in restricted spaces. The two most convenient arrangements of plant for bending and cutting reinforcement are shown in *Fig.* 52; in neither case is any turning required. A general view of a steel bending yard is shown in *Fig.* 53 where the stocks of bars are in the foreground, the bending bench in the centre, and the reinforcement assembled ready for erection in the work is shown in the background.

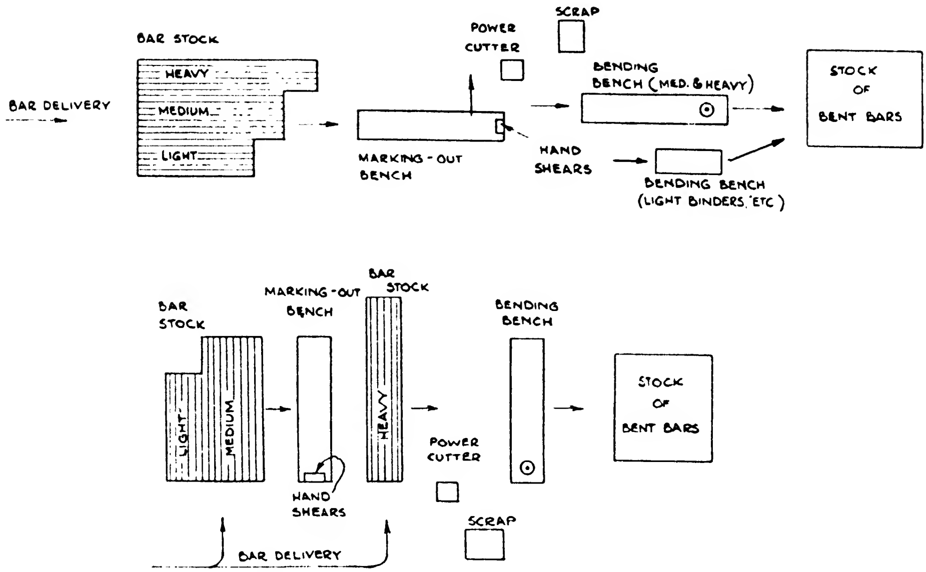


FIG. 52.—ALTERNATIVE ARRANGEMENTS OF REINFORCEMENT PLANT.

FIG. 53.—STEEL BENDING YARD.

Site Storage of Bars.

In making arrangements for storing bars on the site, there are three factors to consider: (1) easy identification of separate bars, (2) avoidance of distortion, and (3) prevention of contamination and corrosion. The methods of storing described facilitate identification and, by providing sufficient intermediate supports, prevent distortion of long bars. To avoid fouling the bars with earth they should be kept off the ground and should be clear of any running or standing water that would cause corrosion. In addition, they should be placed where they can be easily handled during unloading and stacking, and when transferring them from the stack to the bending machines. Stacked material should not impose loads on existing structures in excess of the safe loads for the latter. Special care is required in this direction when large quantities of reinforcement are stored on the floors of buildings or on ground immediately behind the heads of retaining walls.

Bars may be stacked horizontally or vertically, the former being the better. If they are stored vertically (as may be necessary when space is restricted) the length of the bars must be restricted, since long bars will distort under their own weight besides being very difficult to handle in an upright position. Limiting lengths are about 18 ft. for ½-in. bars or under, and 20 to 25 ft. for larger bars. With vertical storing the ends of the bars should be prevented from sinking into the ground by providing a bearing surface of boards, bricks or lean concrete.

When ample space is available the bars should be stored on a mattress formed by parallel timber sleepers placed over the whole storage area. The bars will then be clear of the ground, and if the bearers are not more than 5 ft. apart adequate support is given to all lengths of bars and distortion is avoided. Bars of different diameters should be separated and different lengths kept apart by stakes driven into the ground at each end of the row. Each batch of bars should be labelled with the diameter and length, thereby saving time in identification. When consignments of bars arrive on the site, as unloading proceeds they should be placed directly in their correct batch in the store, thereby avoiding the cost of double handling due to sorting later. It should be made somebody's duty occasionally to check the stored bars to see that no odd bars returned to stock have been placed in the wrong batch.

If the space available for storage is insufficient to allow all the bars to be spread out horizontally in a single layer, they can be stored in horizontal racks arranged in tiers, the racks being constructed in timber or, more sturdily, from light steel angle section or tubular steel scaffolding. The upper tiers should be reserved for small diameter and short bars and the lower tiers for the heavier bars. Each compartment of the rack should be plainly marked with the diameter and length of bars contained. As on an ordinary contract there may be about eight different sizes of bars and a dozen or more different lengths of each diameter, the provision of a separate compartment for each type of bar may be impracticable. To limit the number of compartments, a convenient method is to store together all like lengths of bars of obviously different diameters—say ¼, ½ and 1 in.; when they are stored in racks it is almost impossible to tell by inspection the length of a bar in any compartment, whereas different diameters are readily identified on sight.

SECTION II.—BAR BENDING.

Bending Dimensions.

Much time and effort can be saved on the site if rational dimensions for shaping the bars are supplied to the benders. These dimensions should be so given that a minimum amount of calculation is necessary before marking off the bars and setting the machine and stops. The method of dimensioning given in *Fig.* 54 is one suitable convention and is based on the principle that if any tolerance is permissible the dimensions A and B must not be exceeded. Any

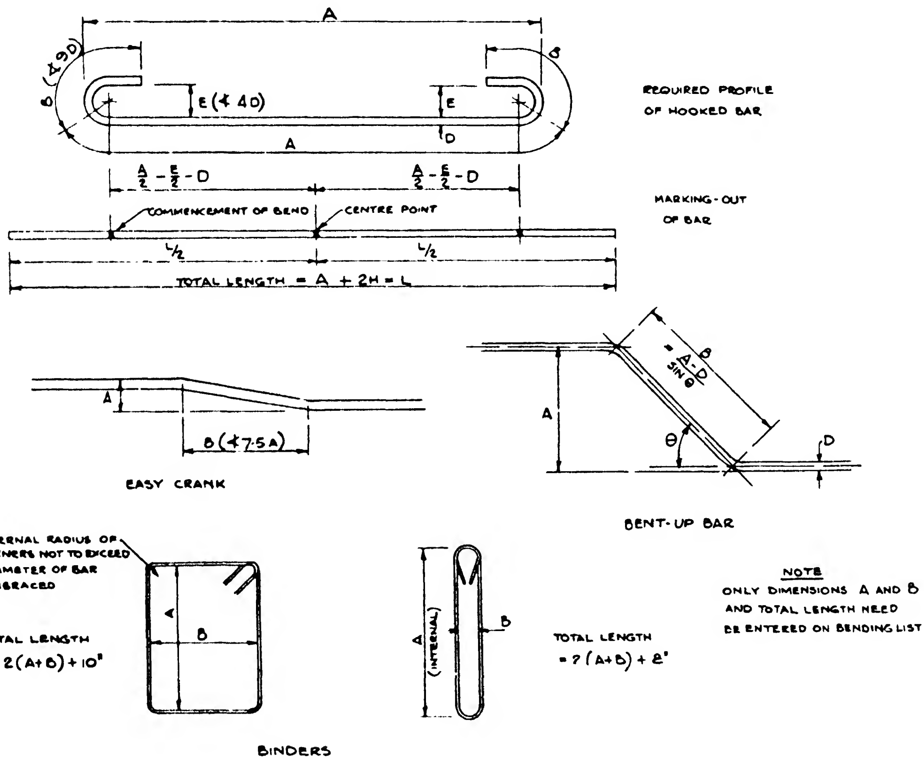
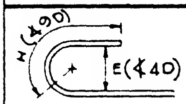
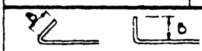


FIG. 54.—BENDING DIMENSIONS.

adjustments in length are made in the length of the hooks. *Table* VIII gives the minimum dimensions for hooks, together with practical values that can be conveniently worked to when they are not specified. Bending dimensions involving angles expressed in degrees should be avoided, since if the machine is not marked in degrees the steel bender in working out the dimensions may make a costly error. The bending schedule should give the length of bar and offset. When bending large-diameter bars care should be taken to ensure that the overall length of the bent bar does not exceed the theoretical or calculated length, as the latter may be based on a limited length of concrete member which cannot be exceeded. There is a tendency for bent bars to "grow" an inch or more during the bending process, and thus exceed the calculated length, unless steps are taken to counteract this possibility.

TABLE VIII.

DIMENSIONS OF HOOKS.

DIAMETER OF BAR			1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1 1/8"	1 1/4"	1 3/8"	1 1/2"
	H	PRACTICAL VALUE	3"	3"	4 1/2"	4 1/2"	4 1/2"	6"	7 1/2"	9"	9"	10 1/2"	12"	13 1/2"	13 1/2"
		MINIMUM	2 1/4"	2 13/16"	3 3/8"	3 15/16"	4 1/2"	5 5/8"	6 3/4"	7 7/8"	9"	10 1/8"	11 1/4"	12 3/8"	13 1/2"
	D	PRACTICAL VALUE	2"	2"	2"	2"	2"	3"	3"	4"	4"	6"	6"	6"	6"
		MINIMUM	1"	1 1/4"	1 1/2"	1 3/4"	2"	2 1/2"	3"	3 1/2"	4"	4 1/2"	5"	5 1/2"	6"
		MINIMUM VALUES OF B	1 1/2"	3"	3"	3"	4 1/2"	4 1/2"	4 1/2"	6"	6"	7 1/2"	7 1/2"	9"	9"

All bends should be "easy," and minimum internal radii, when no guidance is given to the contrary by the designers, should be two diameters for rolled mild steel and four diameters for hard-drawn steel wire, although four diameters are sometimes recommended for all bends. The diameters of bending machine mandrels to give minimum internal radii of two and four diameters are given on *Table* IX. The internal radius of twice the bar diameter gives the correct bend for a semi-circular hook of conventional profile which conforms also with the hook requirements of the building regulations.

TABLE IX.

DIAMETER OF MANDRELS FOR BAR BENDING.

Diameter of bar (inches)		¼	⅜	½	⅝	¾	1	1¼	1½
Minimum diameter of mandrel (inches)	Rolled mild steel bars	1	1½	2	2½	3	4	5	6
	Hard cold-drawn steel wire or bars . .	1½	2¼	3	—	—			—

The radii of bar bends (excluding semi-circular hooks) given on *Table* X are those specified in the London Building By-Laws and the Memorandum thereon, and vary with the quality of the concrete and with the working stress in the reinforcement.

Two conditions are considered, the first being when the thickness of the concrete adjacent to the bar does not exceed the minimum permissible cover, in which case there is a possibility of splitting the concrete if the bend is too sharp. The second case is when the bar is embedded on all sides by sufficient thickness of concrete to prevent splitting, and for these conditions the permitted radii of bends are two-thirds that of the first case. The table gives values of a factor K which when multiplied by the diameter of the bar gives the minimum internal radius of the bend in inches. In a normal building the working stress on the steel would not usually exceed 18,000 lb. per square inch for plain rolled mild steel bars, and for "Ordinary-Quality" concrete the minimum radius allowed on a 1-in. bar would be 7½ in. if the bar is near the surface of the concrete and 5 in. if the bar is well buried in the concrete. If "Quality A" concrete is used, these radii can be reduced to 6 in. and 4 in. respectively.

TABI E X.

RADIUS OF BENDS IN REINFORCING BARS.

VALUES OF K.									
CONCRETE MIX		1 : 2 : 4 (III)		1: 1⅔ : 3⅓		1 : 1½ : 3 (II)		1 : 1 : 2 (I)	
QUALITY		ORD.	A.	ORD.	A.	ORD.	A.	ORD.	A.
TENSILE STRESS IN BARS AT BEGINNING OF BEND. WITH MINIMUM COVER.	10,000	4·2	3·3	3·8	3·0	3·7	2·8	3·2	2·5
	12,000	5·0	4·0	4·6	3·6	4·4	3·4	3·8	3·0
	14,000	5·8	4·6	5·4	4·2	5·1	4·0	4·5	3·5
	16,000	6·7	5·3	6·1	4·8	5·9	4·6	5·1	4·0
	18,000	7·5	5·9	6·9	5·4	6·6	5·1	5·8	4·5
	20 000	8·3	6·6	7·7	5·9	7·4	5·7	6·4	5·0
WHEN NO RISK OF SPLITTING.	10000	2·8	2·2	2·6	2·0	2·5	1·9	2·1	1·7
	12 000	3·3	2·6	3·1	2·4	3·9	2·3	2·6	2·0
	14 000	3·9	3·1	3·6	2·8	3·4	2·7	3·0	2·3
	16,000	4·4	3·5	4·1	3·2	3·9	3·0	3·4	2·7
	18,000	5·0	3·9	4·6	3·6	4·4	3·4	3·8	3·0
	20,000	5·6	4·4	5·1	4·0	4·9	3·8	4·3	3·3
MINIMUM INTERNAL RADIUS = K (DIA. OF BAR). VALUES OF K PRO ?ATA FOR OTHER STRESSES.									

The radius of bends of beam and column binders should be considered in relation to the diameter of the main bars around which they pass. The defects produced by making the corner radii too large or too small are illustrated in *Fig.* 55.

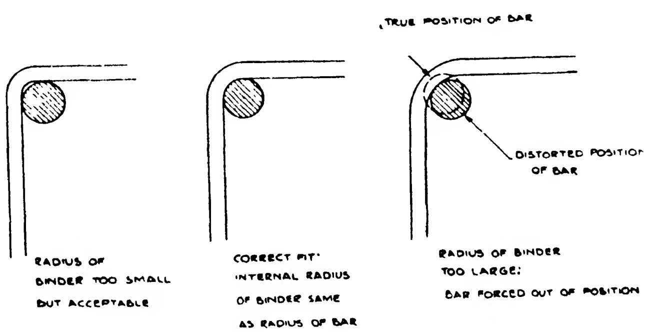


FIG. 55. RADII OF BENDS FOR BINDERS.

When the designing engineer does not supply a bar-bending schedule, or in other circumstances when a schedule is to be prepared, a convenient form is that given in *Fig.* 56. It is essential that the lists should be accurate, fully

JOB No. DATE.

POSITION IN STRUCTURE	No. OF BARS	DIAM. OF BARS	LENGTH OF BARS	DIMENSIONED SKETCH OF BARS	REMARKS
				WAREHOUSE C - COLUMNS - 18" SQUARE	
DRG. No. 6103-3					
BASES TO	24	3/4"	7'-0"	7½" 5'-9" 7½" In footing at 6" crs	20 No.
COLUMNS ① TO					Bases
⑳ INCLUSIVE	8	1"	4'-3"	4'-0" 3" Splice Bars	Thus
	3	5/16"	4'-9"	12" 12" Binders to ditto to fit 1" dia. bars	
1ST LIFT OF	8	1"	17'-6"	13'-3" 1'-9" 2'-6" 3" Main Bars	
COLUMNS ① TO					20 No.
⑳ INCLUSIVE	30	5/16"	5'-9"	15" 15" Binders to Ditto to fit 1" bars	Columns
(FOUNDATION LEVEL					Thus
TO 1ST FLOOR)	30	5/16"	5'-3"	13" 13"	
				WAREHOUSE C - 1ST FLOOR	
SECONDARY					
BEAMS (B1)	2	3/4"	13'-0"	7½" 11'-9" 7½"	
14" × 7" net				1'-3" 1'-3"	12 No.
	1	3/4"	14'-3"	7½" 22½" 16" 6'-9" 22½" 7½"	Beams
					Thus
	2	½"	12'-6"	4½" 11'-9" 4½"	
	17	5/16"	3'-9"	16" 5" Binders at 6" and 9" centres to fit 3/4" bars	

FIG. 56.—BAR BENDING LIST.

descriptive, easy to follow, systematic, and consistent as regards dimensioning. The items should appear on the list as far as practicable in the order in which the bars will be required, and every effort should be made to render perfectly clear the position of each bar. Reference letters are not usually necessary unless the work is very complicated. In dealing with a building, for instance, the bending list would be taken off in the following order: (1) Column footings and other foundations including splice bars for columns and walls; (2) Columns, walls, and stairs up to first floor; (3) First-floor main beams; (4) First-floor secondary beams; (5) First-floor slab; (6) Columns, walls, and stairs up to second floor; (7) Second and subsequent floors and roof in the same order as the first floor.

The example on *Fig.* 56 shows the sequence of bars for the footings and the first lift of a series of columns, together with a typical example of bars for a beam. The order of listing the latter items is: (i) Straight bars in bottom; (ii) Bent-up bars in bottom; (iii) Top bars over supports and elsewhere; (iv) Binders.

The primary object of the bending list is to enable the benders to bend the bars, and although only such information as is necessary for this purpose and for the identification of the bent bars need be put on the lists, nothing is lost by inserting the spacing of binders and bars in slabs and walls and stating whether the bars are in the bottom or top of the slab, on the inner or outer face of walls, or vertical or horizontal. The diameter of the bars with which column or beam binders are in contact should be included in the bending lists.

All information required for assembling and fixing the reinforcement in the work should be repeated and amplified on the detail drawings.

Bending Operations.

Bending for small bars may be done either by improvised means or by hand-operated machines, and by power benders for all sizes. For bars of ½ in. diameter

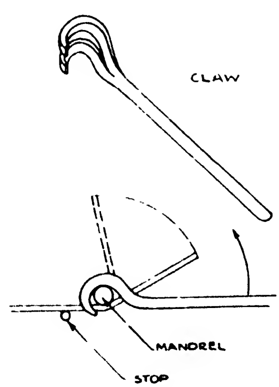


FIG. 57.- BENDING BY MEANS OF CLAW.

and less in small quantities a simple method is to hold the bar in a vice and obtain leverage by a length of gas-pipe slipped over the protruding end of the bar.

Alternatively small bars can be placed between two stops and bent with a gas-pipe sleeve in the same way. A third and common method is to use a " claw." This tool and the method of using it are indicated in *Fig.* 57.

There are numerous bar-bending machines on the market and the following descriptions apply to models of well-known makes. The simplest machine

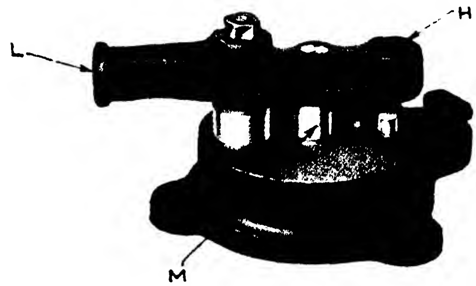


FIG. 58.—SIMPLE BAR-BENDING MACHINE.

suitable for bending bars cold consists of a direct-acting lever as shown in *Fig.* 58 and can be used for bars up to ⅝ in. diameter. The lever, which is rotated through the same angle as that of the bend, is inserted in the socket (L). The centre mandrel (M) is 1 in. in diameter so that for all bars greater than ¼ in. a radius former of the required size and placed over the mandrel is necessary. As shown

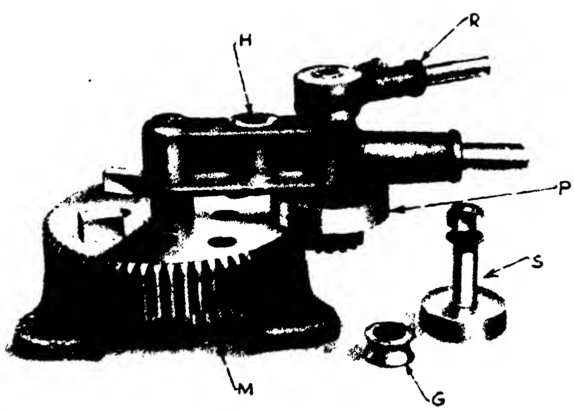


FIG. 59.—GEARED BAR-BENDER.

in the illustration the machine is set for bending small bars, but for ½ or ⅝-in. bars the arm is removed and replaced so that the centre mandrel passes through the hole (H).

For bars larger than ⅝ in. geared benders are required. The machine illustrated in *Fig.* 59 will bend cold bars up to 1 in. diameter. Since the standard centre mandrel is 1¾ in. in diameter, radius formers are required for all bars likely to be bent on this machine. The machine is operated by the backward and forward motion of the ratchet lever (R). As illustrated, the machine is arranged for bending ¾-in. bars, but if larger bars are to be bent the plain roller (P) on the ratchet spindle is replaced by the grooved roller (G). Bars of ⅝ in. diameter and smaller can also be bent on this machine, and for this purpose direct lever action is obtained by removing the ratchet spindle, roller, and gear pinion and inserting the plain roller and spindle (S) in the hole (H) in the rotating arm. The machine is then operated exactly as is that shown in *Fig.* 58.

A third machine that can be used for all sizes of bars up to 1½ in. diameter is, as shown in *Fig.* 60, operated by a worm and pinion rotated by a handle (H).

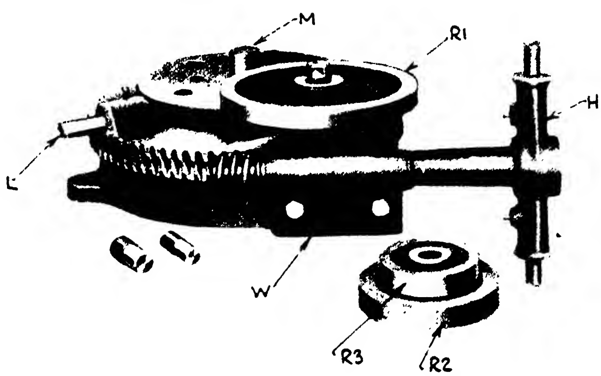


FIG. 60.—WORM-OPERATED BAR-BENDER.

The large roller (R1) shown in position on the machine is 12 in. in diameter and is used for bars up to 1 in. diameter. For 1¼-in. bars the 8-in. diameter roller (R2) is used, while for 1½-in. bars the 5½-in. roller (R3) is fixed to the rotating pinion. Since the centre mandrel (M) is 2 in. in diameter a radius former would be required for most bars bent on this machine to give the minimum diameters specified on *Table* IX. If the bar is ½ in. or less in diameter the centre mandrel would ordinarily give a sufficient internal radius of bend, and for such small bars the bending can be done by direct leverage if the worm bracket (W) is removed and a lever inserted at L.

More than one man can operate a direct-lever machine if a pipe of sufficient length is slipped over the lever, thus allowing a greater leverage to be exerted. By removing the arms of a handle-type machine and inserting a length of 1-in. diameter bar, bent at right angles and with sufficient length outstanding horizontally for two men to obtain a hold, the power of the machine can be increased. When adopting either of these or similar expedients care should be taken to ensure that the machine is not strained by being worked beyond its capacity.

Hand-operated machines of the types described are usually bolted to a bench, examples being illustrated in *Fig.* 61.

Bending bars 1½ in. and upwards in diameter requires special equipment. If only a few bars are to be bent, easy bends can be formed by a "jim-crow" or rail-bender. This appliance is illustrated in *Fig.* 62 and consists of a forged bow with a mild steel square-threaded screw. A suitable size for bending reinforcement bars up to 2 in. in diameter would be 18 in. across the bow and fitted with a 2-in. or $2\frac{1}{4}$-in. diameter screw.

If a considerable weight of large diameter steel has to be bent the provision of a power-operated bender may be advisable. These machines are usually

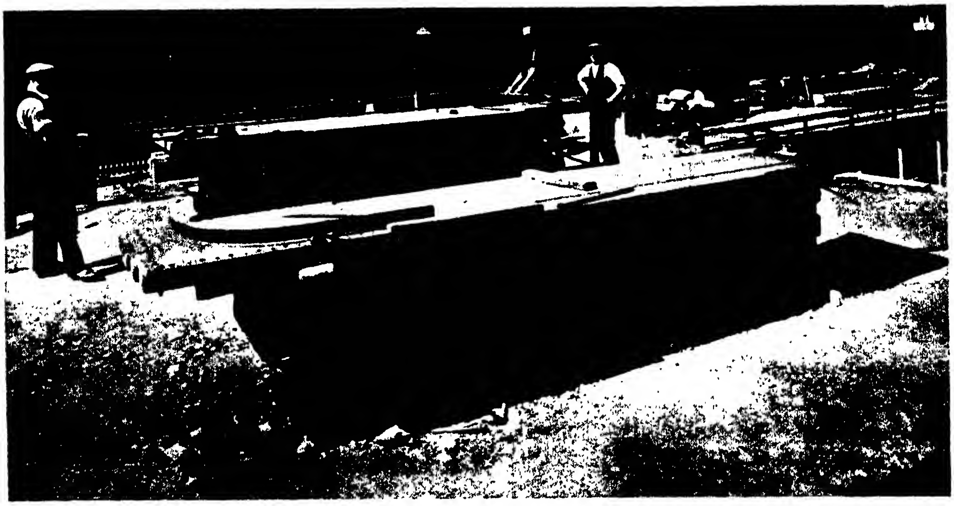


FIG. 61.—BAR-BENDING BENCHES.

operated by electric power and can be used for bending all sizes of bars required on a contract. An operator of average skill can reduce labour charges for bending to a very small percentage of hand-bending costs, since each bend is more quickly formed and one, two, or more bars, depending on the diameter, can be bent

FIG. 62.—JIM CROW.

at a single operation. Some power machines can form hooks on two $1\frac{1}{2}$-in. diameter bars simultaneously, and curved bars can be bent several at a time true to radius and without the formation of the kinks associated with hand-bending of this type of bar. Offset against the labour saving are the costs of obtaining, installing and running the machine. The cost of providing the machine for a given contract depends on whether it is necessary to buy or hire the machine for a single contract or whether the machine is already owned by the contractor and can be kept in constant use on successive contracts. The power consumption is about 1 kw. per ton of bars, and the performance about 40 tons in an 8-hour day, and on a machine having this output it would be possible to bend simultaneously four 1-in. bars, three $1\frac{1}{8}$-in., two $1\frac{1}{4}$-in., or a single 2-in. bar.

Electrically operated machines may be controlled either by a hand lever or a foot pedal and can be fitted with an attachment that automatically controls the angle through which the bar is bent. On some types the centre rotating bending plate automatically returns to the starting position upon completion of the bend.

The power bender illustrated in *Fig.* 63 bends single bars up to $2\frac{3}{8}$ in. diameter and the provision of two bending discs, the smaller of which rotates faster than the larger, allows smaller bars to be bent at the same tonnage rate as large bars. A machine of this type is illustrated in actual use bending 2-in. bars in *Fig.* 64.

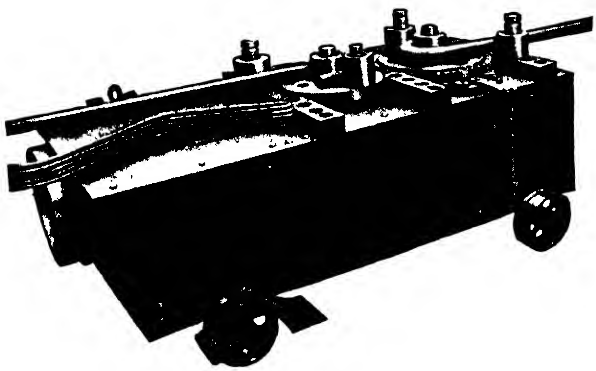


Fig. 63.—Power Bender.

Although bending machines are usually of robust construction, they should be given the same care and attention as other mechanical equipment. Moving parts should be kept oiled or greased, but this should be done so that surplus grease does not come in contact with bars being bent. Machines in the open should be covered at night with a tarpaulin or other means of protection. If a machine is to be used in one position for some weeks it is advisable to rig up a housing consisting of a tarpaulin supported on a framing of 4-in. by 2-in. timbers. This protects the machine, and also enables work to proceed during wet weather.

Whatever bending method is adopted all bends should be made gently, avoiding all jerky action. The bars may be bent hot so long as the strength of the material does not depend upon cold working as does that of cold-drawn wire reinforcement.

The bar should be heated to the cherry-red colour associated with a temperature of about 1,550 deg. F. and should be bent while at this temperature. It is imperative that the hot bar should be allowed to cool slowly, quenching by immersion in water or otherwise being prohibited. Unless these conditions can be rigorously complied with, bending hot should be forbidden. Usually hot bending should be limited to bars 1⅛ in. and upwards in diameter; smaller bars can usually be bent more conveniently while cold, and cold-drawn steel is commonly only used in small diameters.

FIG. 64.—BENDING LARGE BARS.

Special attachments are required successfully to bend twisted twin bars of Isteg or similar type. Such an attachment in the shape of grooved formers fixed to a power bender is illustrated in *Fig.* 65, by which means one or two bars

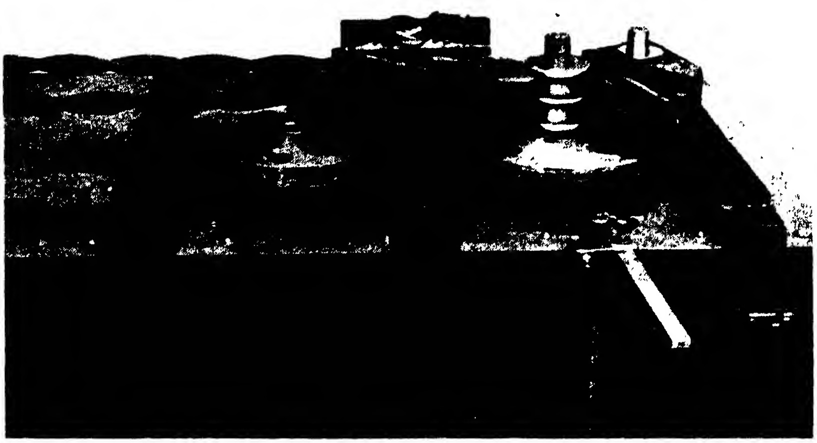


FIG. 65.—BENDING TWISTED TWIN BARS.

can be bent simultaneously without distortion. The grooves in the formers are so proportioned that the twin bar can be placed on the machine without having to fit the bar to the former. The length of the clamps and rests is sufficient to extend over two or more twists in the bar, thus ensuring a support parallel to the axis of the twin bar.

The sequence of operations in bending a bar by machine or otherwise needs preliminary consideration if accuracy, convenience, and minimum handling are to be attained. *Figs.* 66, 67 and 69 indicate recommended methods for three bars of characteristic profile. The dimensions given assume the bar to have been dimensioned in accordance with the conventions given in *Fig.* 54.

Before bending the symmetrically bent-up beam bar shown in *Fig.* 66 the

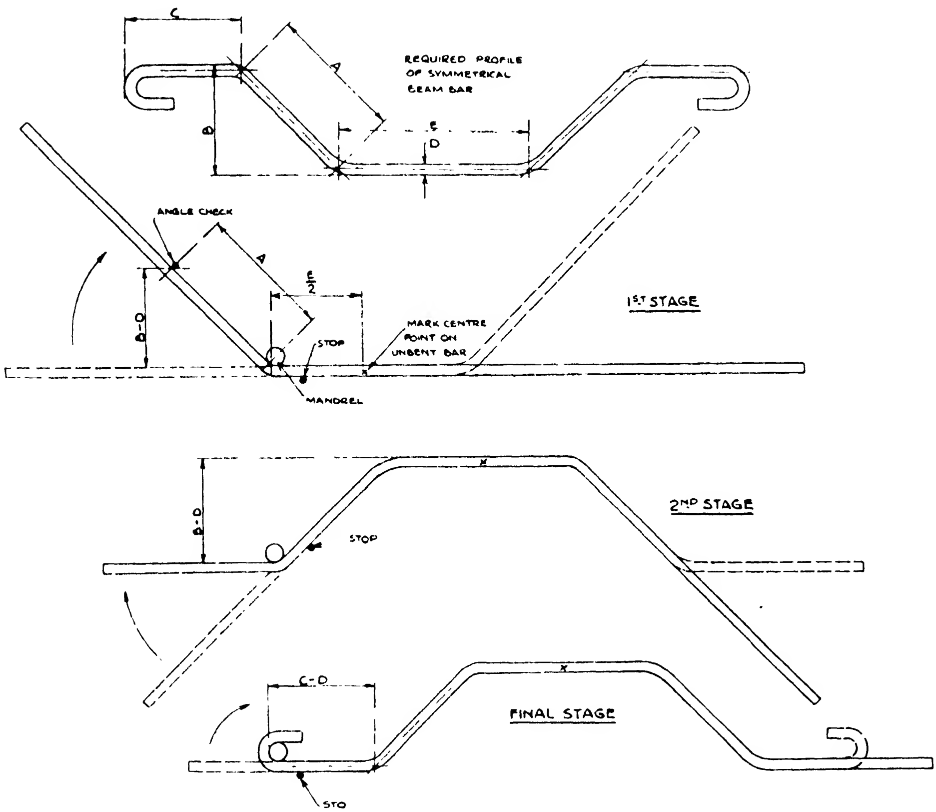


FIG. 66.—BENDING BEAM BARS.

centre point on the straight bar is chalk-marked, and from this mark the points of bending are marked off in each direction. The diameter of the mandrel or radius former attached to the machine is related to the bar diameter as in *Table* IX, and the angle check should be secured in the position indicated on the diagram. The first operations are to form the crank at one end of the bar, reverse the bar, and form the crank at the other end. The second operations are to form the horizontal lengths at each end of the bar, and finally to bend the hooks at each end, using for this purpose the surplus length of bar. The length of the straight portion of the hook will not be identical with the length specified in the bending diagrams if the length of the original straight bar is not exact.

When a large number of small diameter slab bars or binders of identical shape has to be bent the process of marking every bar can be avoided, the following

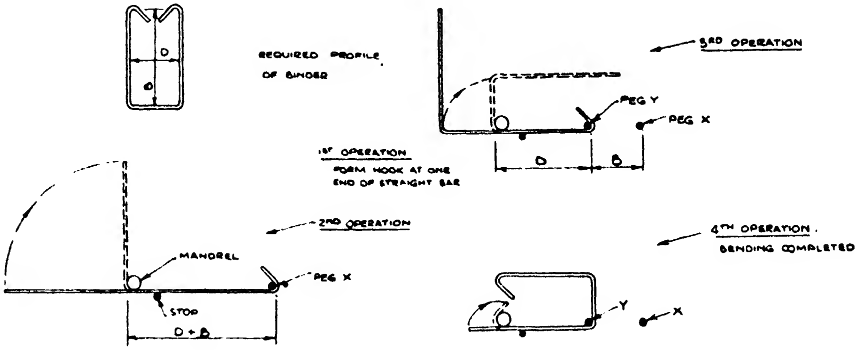


FIG. 67.—BENDING BINDERS.

procedure, as shown in *Figs.* 67 and 69, being more convenient. A series of nails or short pieces of bar to act as stops or pegs is driven into the bench or into a piece of timber fixed to the bench, the positions being determined from

FIG. 68.—BENDING LINKS.

the finished dimensions of the bar as indicated. Either of the bars illustrated can be bent by machine or by a claw or similar tool.

In the case of the beam binder shown in *Fig.* 67 the preliminaries to bending involve securely fixing pegs X and Y in relation to the mandrel and the dimensions

B and D. By means of a claw a hook is bent at one end of the straight bar, and when this is placed over peg X the bar is in the correct position for forming the first bend. The bar is then shifted so that the hook embraces peg Y and is now correctly placed for the second bend, leaving only the final operation of forming the second end hook.

Similarly with the slab bar indicated in *Fig.* 69, from the given dimensions

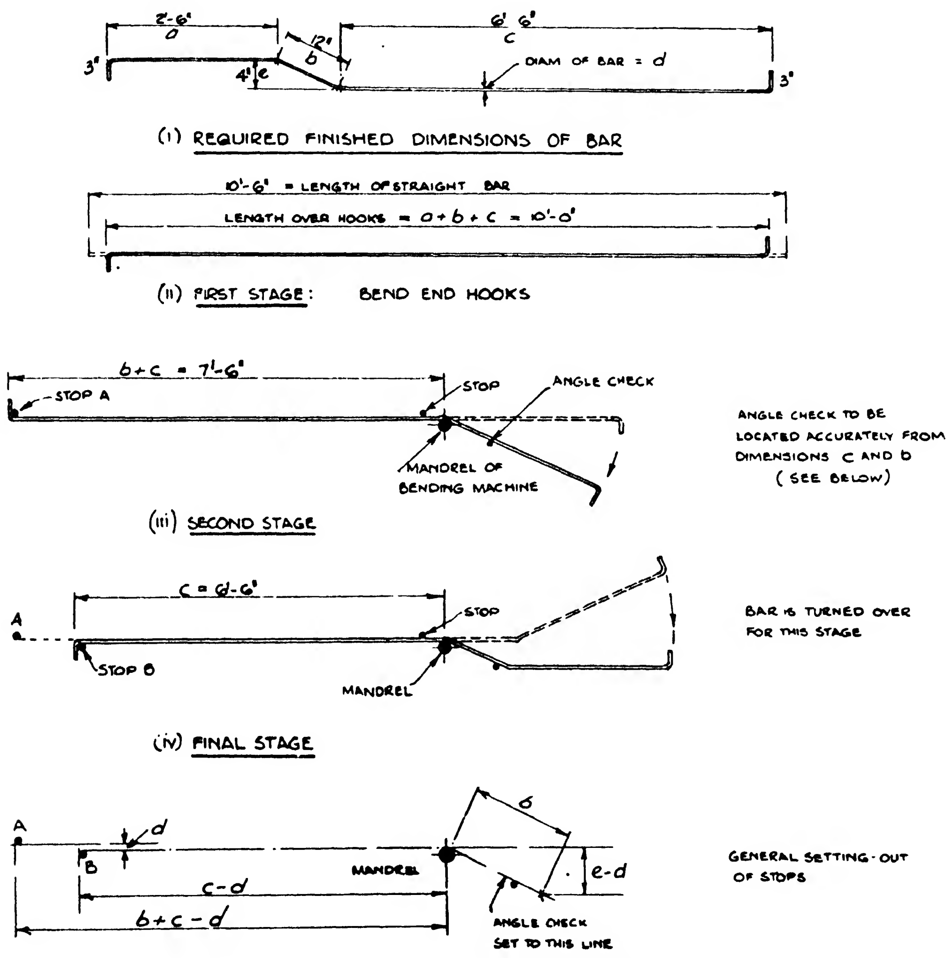


FIG. 69.—REPETITION BENDING FOR SMALL BARS.

the pegs or stops A and B and the angle check are fixed, it being essential that the latter should be accurately set out. In this case both ends can be formed before bending commences; if only one is formed it should be that at the end of dimension *c*. With the end hook embracing stop A, the first bending operation is to form the bend between dimensions *a* and *b* and then, by turning the bar over and engaging the hook with stop B, the final bend between dimensions *b* and *c* can be formed. If there are no hooks on the ends of slab bars it is necessary to fix a block of wood or other stop at the positions A and B, against each of which in turn the end of the bar should be held during the formation of the two bends.

The first step in the process of bending links as illustrated in *Fig.* 68 is to form a hook at one end of the bar. The adjustable stop in the metal plate is set at the correct distance from the mandrel to allow the bottom loop to be bent as indicated. The surplus length of bar after completing this operation is bent to complete a link of the shape of the finished units lying on the bench. In this case the diameter of the mandrel would be equal to the clear distance between the arms of the link, that is dimension B on *Fig.* 54.

In any method that involves setting out pegs for repetition bending, it is advisable to check the final dimensions of the first bar bent to make sure that the setting of the pegs will give the required shape.

For forming helical binding for columns or piles, an improvised machine as illustrated in *Fig.* 49 can be adopted or a device such as that shown in *Fig.* 70 can be attached to certain types of power bar bending machines. The relative

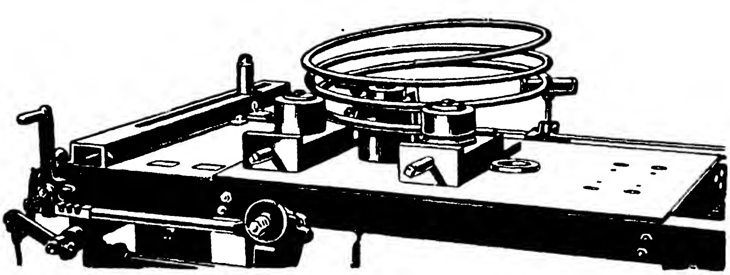


FIG. 70.—FORMING HELICAL BINDING.

position of the three rollers and support is adjusted to the required radius of the helical.

In place of hooks at the ends of bars, fish-tails are sometimes specified; these can be formed by cutting the end of the bar with a hacksaw and bending the two pieces outwards by means of a nailbar. If a flame is available the slit can be cut much more readily than with a saw. A flame is especially suitable for this purpose when fish-tails have to be formed on the ends of bars projecting from piles already driven.

Handling Bars.

Heavy bars, whether in straight lengths or bent, are handled singly, long bars being carried on the level by two or more men. When bent bars are being hoisted to the fixing position, the lifting tackle can be attached to one of the end hooks. Light bars, however, can be more conveniently handled if bundled together, but it is necessary to wrap sacking or jute cement bags around the bundle to prevent the slings slipping during the process of hoisting. Means of hoisting bars from ground level to working levels are discussed in Chapter I.

If the bending or fixing of the reinforcement is let to a sub-contractor, the contract should clearly indicate whether the labour, plant, and power for hoisting and other handling of the bars between the bending machine and the shuttering are included in the sub-contract. On large jobs this item may be quite considerable and is easily overlooked or assumed to be included. The same condition applies to unloading bars that are supplied by merchants or others.

FIG. 71.—ERECTING HEAVY BARS.

When handling bars adjacent to finished concrete work, care should be taken not to damage the corners of columns and beams. All columns adjacent to any hoist should be protected from damage by timber guards.

Handling large bars when erecting them in their permanent position in the work may also present problems as individual bars in long-span beams may weigh several hundredweights. In *Fig.* 71 is shown a method of hoisting 2-in. diameter bars 90 ft. long into position in the top of a beam.

SECTION III.—FIXING REINFORCEMENT.

Assembling Bars.

The principal factors in steel placing are precision and convenience. The bars should be placed accurately in the positions indicated on the detail designs and maintained in these positions. When delay occurs between assembling the steel and depositing the concrete, the placing of the reinforcement should be again carefully checked immediately prior to concreting. If they have been exposed for a considerable time, bars may need brushing to remove loose rust or other deposits. Any material brushed off that falls into the shuttering will be washed away when the latter is cleaned out. If it is anticipated that the

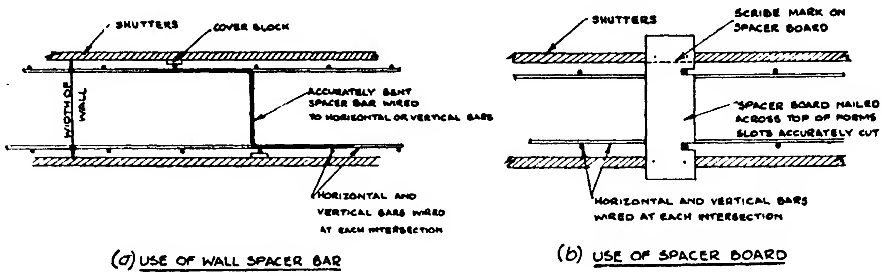


FIG. 72.—MAINTAINING WALL BARS IN POSITION.

assembled bars will be exposed to the weather for more than a few days, a cement grout wash should be applied and brushed off before concreting.

Certain precautions should be taken to prevent displacement during concreting ; generally the provision of cover blocks, as described later, will be sufficient to maintain the correct position of the bars in columns, beams, ordinary floor slabs, and similar members. In walls and slabs further measures than are afforded by this means or by the shapes of the bars are often needed. In *Fig.* 72

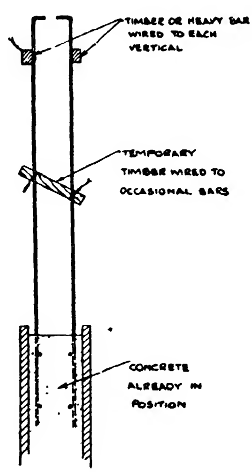


FIG. 73.—STIFFENING PROJECTING VERTICAL WALL REINFORCEMENT.

two methods of maintaining the position of wall bars are indicated ; in (*a*) the two layers are separated and kept in their correct relative position by being attached to an accurately bent bar that acts as a distance piece, while in (*b*) the same effect is obtained by placing the vertical bars in slots accurately cut in the timber with the additional advantage of maintaining the correct cover.

The slots can be replaced by pairs of projecting nails, and a scribe mark on the timber will indicate the correct alignment with the inside face of the shutters. The bar in (*a*) remains permanently in position, but the timber in (*b*) must be removed before the shutters for subsequent lifts of wall can be erected.

Distortion of projecting vertical bars in double rows in walls can be prevented prior to the complete assembly of the horizontal reinforcement and erection of the shutters by the arrangement shown in *Fig.* 73. This consists of a longitudinal timber or heavy bar placed along each row of vertical bars and wired to each bar. Between every fifth or sixth pair of opposite bars a cross-piece (either a length of wood or steel bar) is wired. For single vertical

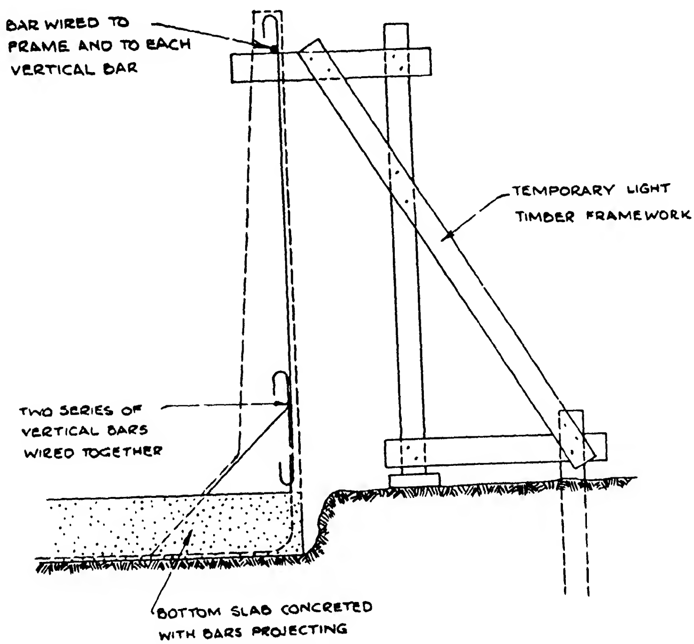


FIG. 74.—SUPPORTING VERTICAL WALL STEEL.

rows of heavy steel in walls it is usual to erect a timber frame constructed on the lines indicated in *Fig.* 74. The dimensions, details, and spacing of the frames are determined by the nature of the work, and it is generally left to the foreman steel-fixer or supervisor to decide the minimum of support required to ensure the rigidity of the assembled reinforcement. The frame could often be part of the shuttering, the soldiers in wall construction, for example, forming the vertical posts of the frame.

In the case of the small chimney shown in course of erection in *Fig.* 75 the ring of vertical steel projecting above the finished level of the work is maintained in position by a wooden template. The bars are secured to the latter by wiring, or light nails or staples may be used. The correct spacing of the vertical bars is secured as successive rings of horizontal reinforcement are fixed.

Where double reinforcement occurs in slabs, as in heavy foundation rafts, the method shown in *Fig.* 76 for the support of the top layer is adaptable. Here additional support bars are inserted, but in a first-class design the provision of

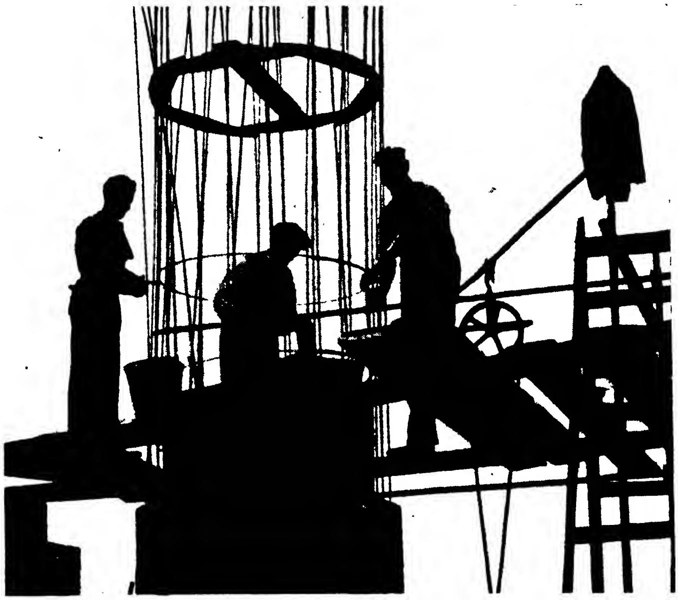


FIG. 75.—SUPPORTING VERTICAL REINFORCEMENT.

these extra bars would not be necessary as the designer would so arrange the reinforcement that the supporting bars would be part of the principal reinforcement. *Fig.* 77 shows bars supported by a timber strut. In the case of normal

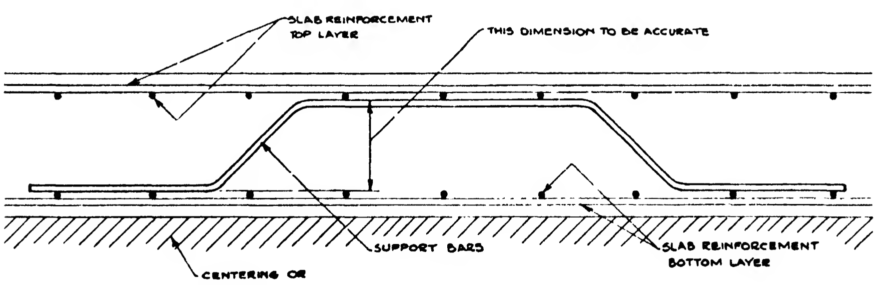


FIG. 76.—DOUBLY-REINFORCED SLAB.

beam-and-slab construction the top layer of slab reinforcement is usually adequately supported on the top bars in the beams. Where this is not the case, as is illustrated on *Fig.* 78, and where the top layer consists of light bars only, the latter can be supported on small timber props which are removed one at a

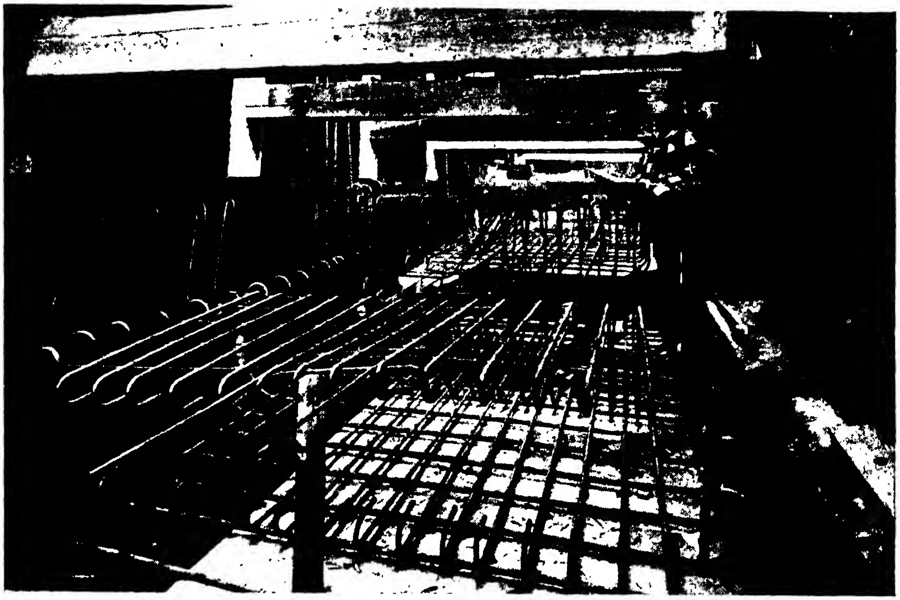


FIG. 77.—SUPPORTING TOP LAYER OF BARS.

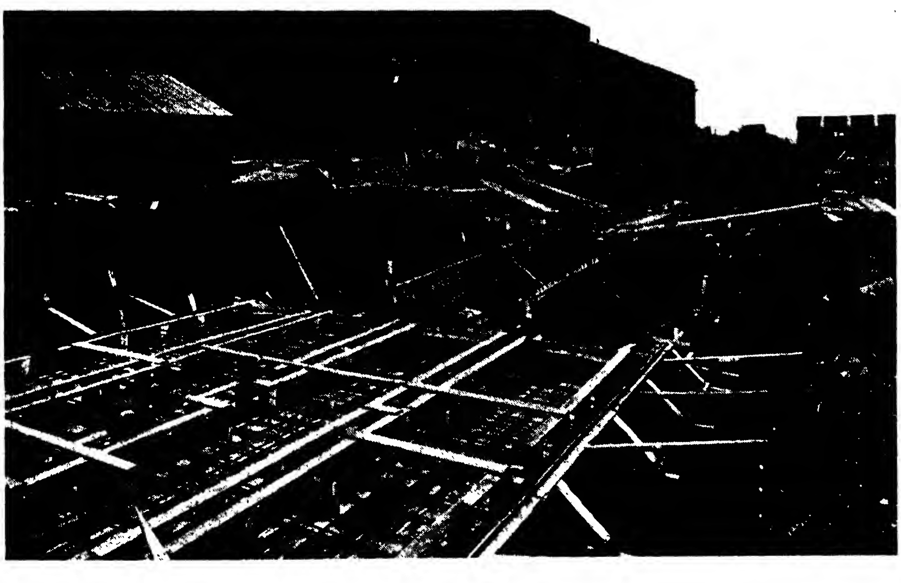


FIG. 78.—SUPPORTING TOP LAYER OF SLAB STEEL.

time as concreting proceeds. The height of the props must be accurately determined to give the specified cover over the top bars. In the illustration all the bars have not yet been supported, but this must be done before concreting is commenced. Only a sufficient number of props is required to maintain the top layer in position without sagging. To prevent disturbance of the steel after it has been finally placed, elevated walkways formed from timber boards are used as in *Fig.* 78. If they are set at the correct height, the longitudinal boards can act as screed guides, and where openings have to be formed through the slab these boards can be laid so as to keep stable the boxes forming the holes. These boxes can in turn support the boards.

The operations involved in assembling reinforcement are closely related to the erection of the shuttering and the concreting procedure, and all three must

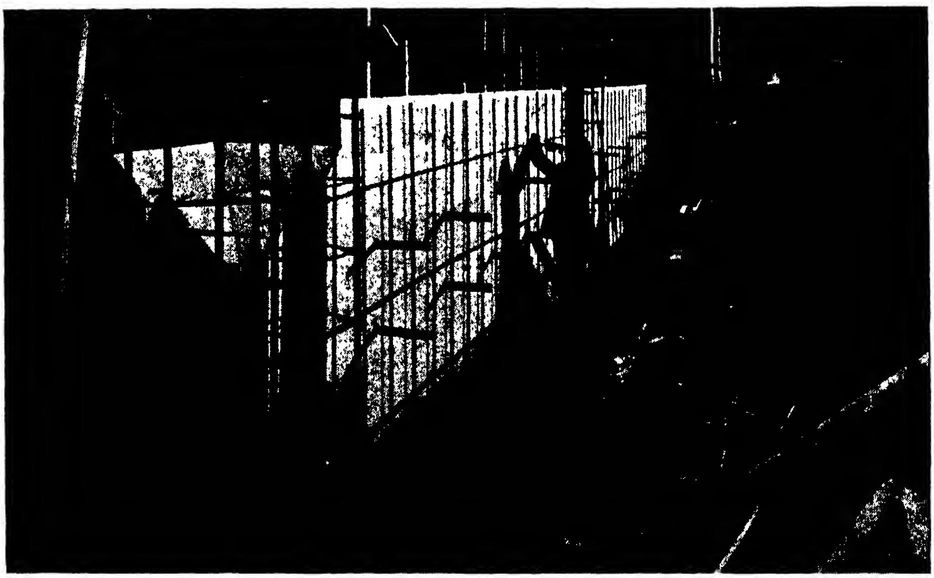


FIG. 79.—ERECTING WALL REINFORCEMENT.

be considered together. Steel fixing usually precedes the erection of shuttering in the case of foundations, columns, and walls, and follows the shuttering of slabs, although sometimes for walls and wall columns the shuttering for one face is erected before the reinforcement as shown in *Fig.* 79. For beams the shuttering is partly erected before placing the steel, the bottom and perhaps one side of the beam box being built up to provide a support on which the bars can be assembled with ease.

For such members as pre-cast piles and braces, the fabrication of the bars and binders into rigid skeletons before they are placed in the shutters is usually the most convenient method ; this can also be extended with advantage to the reinforcement in light columns, single-span beams, and lintels. Mats of foundation and road reinforcement also are often more conveniently made up before they are placed in position in the excavation. The benefit of this method is realised when the reinforcement is to be placed in awkward positions, such as within an existing building or over water, which make it desirable to do as much as possible of the preliminary work in more spacious surroundings.

The skeleton is assembled adjacent to the bar bender, and is carried to the work and deposited in the shutters when required. In all cases when the unit contains any considerable weight of bars, the gross weight of each skeleton should be ascertained beforehand to ensure that the capacity of available cranes or other lifting devices is not exceeded. The construction of the skeleton must be such

FIG. 80.—ASSEMBLING PILE REINFORCEMENT.

that during handling it will not sag or twist to such an extent as to produce permanent deformation, and stiffeners, such as the diagonal forks commonly placed in pile skeletons, should be introduced if necessary. In the background of *Fig.* 49 skeletons for octagonal piles are shown in the course of fabrication. One of the longitudinal rods in each skeleton, with the necessary number of helical units threaded on, is suspended from the timber frames. This leaves the interior of the helicals clear, thus enabling the remaining longitudinal rods to be inserted and wired in position on the perimeter of the binding.

A method of assembling pile skeletons which is equally applicable to heavy column reinforcement is shown in *Fig.* 80, where the main bars are carried on a frame formed of rails, the top bars being supported by a small metal frame and rod as illustrated until the diagonal forks or struts, together with the supplementary ties, are in position. The binders are bent while being wrapped round

FIG. 81.—REINFORCEMENT SKELETON FOR RECTANGULAR COLUMN.

the main bars, a claw being used for this purpose. The rods from which the binders are formed are supplied to the fixer in straight lengths with a small bend at one end. This method is advantageous when the main steel is heavy

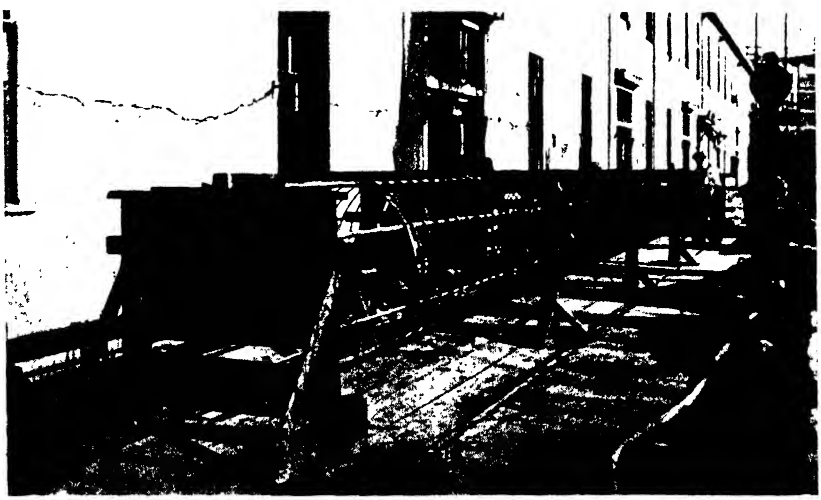


FIG. 82.—REINFORCEMENT SKELETON FOR HELICALLY-BOUND COLUMN.

and needs several points of support, as in this case the binders cannot be easily threaded on from one end. The process shown enables the binders to be put on in such a way as to embrace the longitudinal bars tightly.

In *Fig.* 81 is illustrated a skeleton of heavy column reinforcement ready to be transferred to the shutters while a similar skeleton for a large column with a helically-bound core is shown in *Fig.* 82.

A common type of frame used for assembling a similar skeleton for a light four-bar column is illustrated in *Fig.* 83. The frame consists of two self-supporting movable trestles, across which two of the longitudinal bars are placed with the binders loosely threaded on. The remaining two lower bars are inserted in the corners of the hanging binders. One of the top bars is chalk-marked to indicate the spacing and the binders are pushed along to their correct position

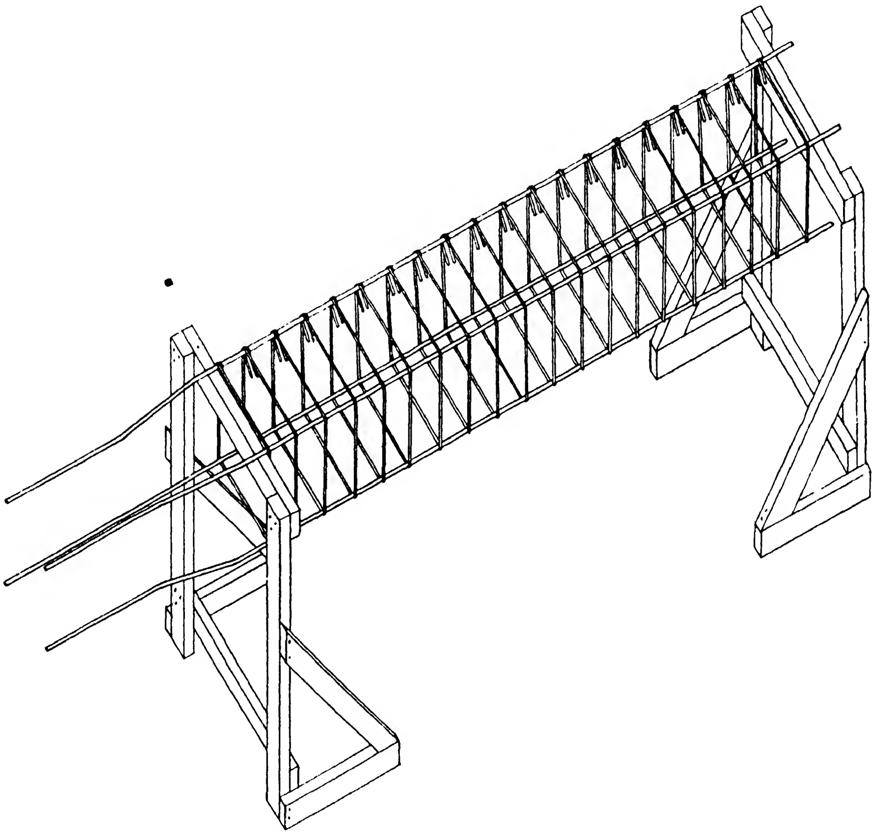


FIG. 83.—ASSEMBLING COLUMN SKELETON.

where they are wired to each of the longitudinal bars. If the ends of the latter are cranked or hooked it is necessary to ensure that they are turned in the direction indicated on the detail drawings before any wiring is commenced. This method of assembly can be easily extended to columns of more than four bars and with double or treble systems of binders. If the length of the column is such that the skeleton deflects unduly between the end supports, stiffness during assembly can be obtained by spanning a loose board between the trestles. The binders will then be threaded on this board and will be suspended therefrom before inserting the longitudinal bars.

For lighter skeletons a convenient support can be made by driving pairs of bars firmly into the ground as shown in *Fig.* 84. The upper end of each bar is bent to a double hook which supports a short length of cross bar, the latter being

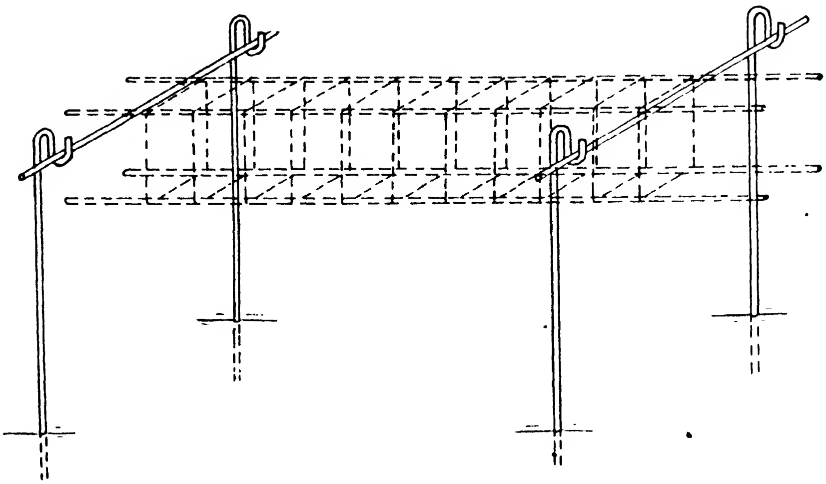


FIG. 84.—TRESTLES FOR ASSEMBLING COLUMN REINFORCEMENT.

about waist-high. Bars of $\frac{3}{4}$-in. diameter are suitable for both uprights and cross-pieces.

An example of a skeleton of a heavy beam for a foundation assembled ready for placing in the excavation is shown in *Fig.* 85.

FIG. 85.—BEAM SKELETON.

The erection in situ of the reinforcement for tall and heavy columns usually requires some form of staging, and in *Fig.* 86 is illustrated a convenient method of providing a working platform for the fabrication of the upper part of such a column. If this is adopted care must be taken not to distort or displace the binders in the lower part of the skeleton. It is essential in column construction to ensure that the upper ends of cranked or straight vertical bars, whether main bars or splice bars, are in the correct position to fit in with the vertical bars and binders in the subsequent lift of concrete. This must be attended to before concreting is commenced, and the upper ends of the bars are best maintained in their relative positions by providing a template. As illustrated in *Fig.* 87, the template, in the form of a rectangular piece of wood or metal, is drilled or notched

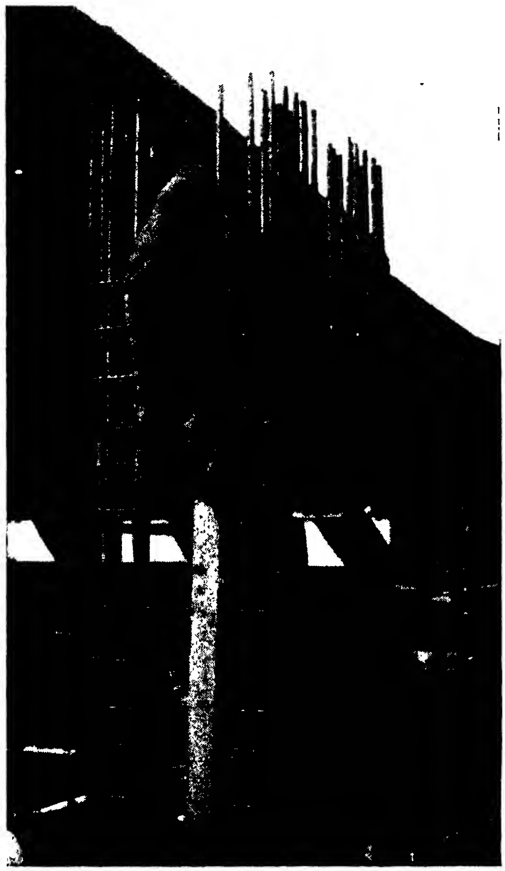


Fig. 86.—Erecting Column Reinforcement.

to take the projecting ends of the bars and is removed after the concrete has set and before the reinforcement in the subsequent lift is placed. As an alternative and simpler form of template, a wooden frame, as indicated also on *Fig.* 87, can be used.

Beam reinforcement may be kept in position during concreting by one of the methods shown in *Fig.* 88. All longitudinal bars are wired to the binders. In the case of the tee beam, concreting in the first instance proceeds up to about the level A—A, so that a piece of wood to support the whole skeleton of beam steel can be inserted under the top bars as shown. This support, which can be either a length of bar packed up as shown in *Fig.* 89, or a length of wood, bears on the slab sheeting and must be removed before concreting the slab. During the

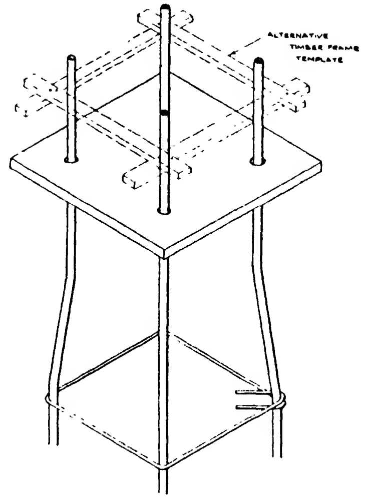
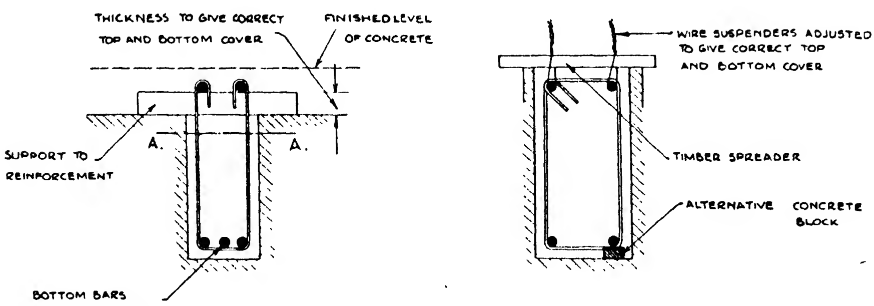


FIG. 87. COLUMN BAR TEMPLATE.

latter operation the reinforcement is rigidly held in the set concrete in the stem of the beam. For a rectangular beam which is concreted in a single operation, the reinforcement is suspended by wire from the timber spreader attached to the sides of the beam box. Alternatively it may be supported from below by



(a) TEE BEAM (b) RECTANGULAR BEAM

FIG. 88.—SUPPORTING BEAM STEEL.

resting on a concrete cover block. The correct side cover is obtained by inserting cover blocks between the shutter faces and the longitudinal bars. The top bars in the heavy reinforcement in the bridge girders illustrated in *Fig.* 90 are suspended from a series of substantial timber frames by wires, the correct position

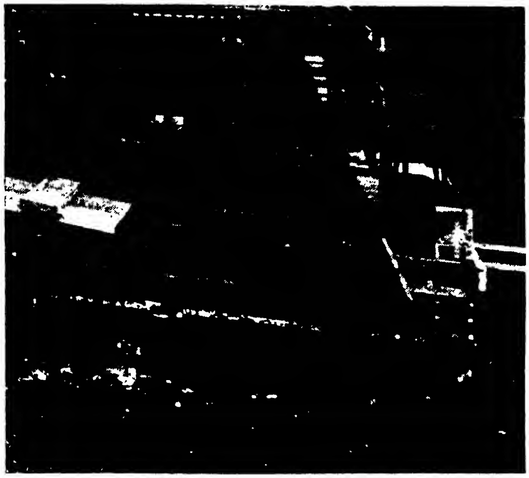


FIG. 89.—SUPPORTING BEAM BARS.

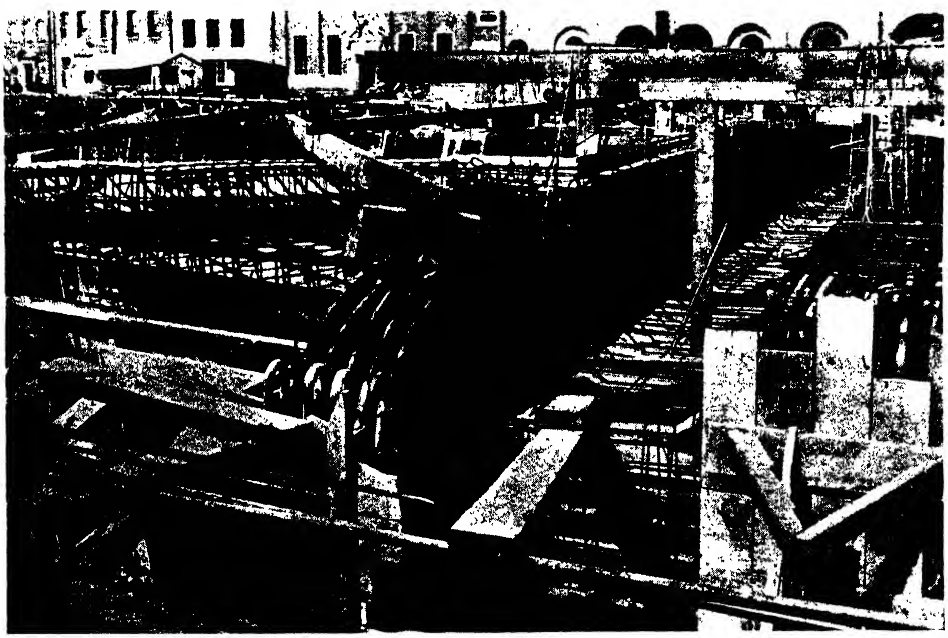


FIG. 90.—SUSPENDING REINFORCEMENT FOR BRIDGE GIRDER.

being obtained vertically by the tourniquet action of a short length of bar inserted between the two strands of twisted wire. The wire should be No. 9 gauge An

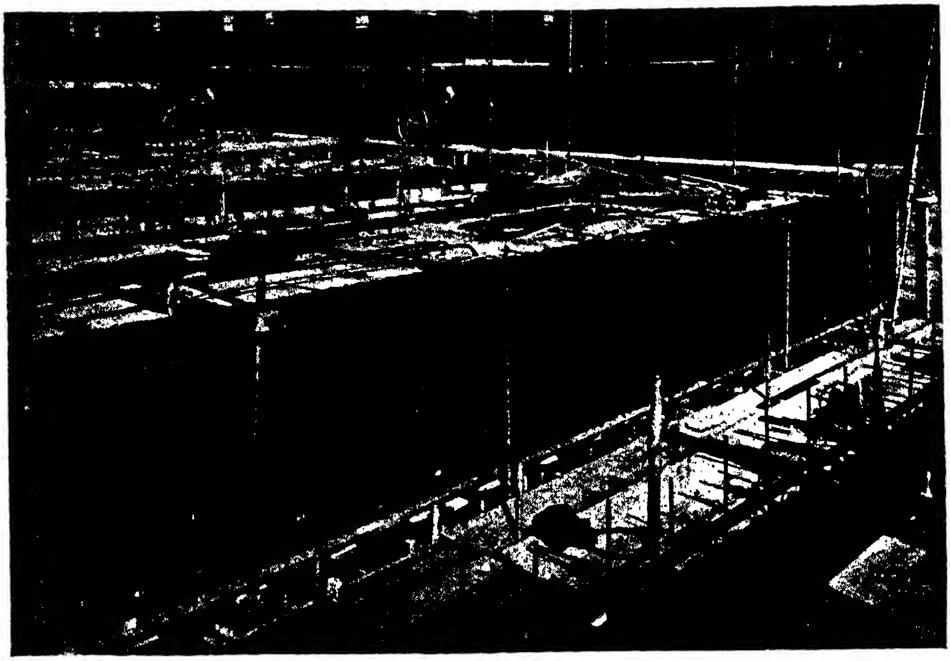


FIG. 91.—SUSPENDING BEAM REINFORCEMENT.

alternative method is shown in *Fig.* 91, where the wire ties suspending the heavy beam reinforcement are attached to a frame constructed from tubular steel scaffolding.

Wire Ties.

To assist in supporting and maintaining assembled loose bar reinforcement rigidly in position, intersections of bars are secured by wire ties. A typical simple arrangement of bars for a slab laid directly on the ground is illustrated in *Fig.* 92 where alternate intersections are wired. Soft iron black wire of No. 16 gauge (about $\frac{1}{16}$ in. diameter) is usually employed for this purpose, although other gauges are sometimes used. A lighter gauge than No. 18 (about $\frac{3}{64}$ in.) is not recommended, nor is a heavier gauge than No. 14 (about $\frac{5}{64}$ in.) necessary for normal work.

The amount of tying wire required varies considerably with the character of the reinforcement, being less per ton of bars if the latter are of large diameter than when small diameters predominate. For general building work an allowance of 5 to 7 lb. per ton of bars is ample. This gives 150 to 200 yd. run of No. 16 gauge wire ; allowing for some waste, this would be sufficient for wiring about 1,000 intersections. The wire is usually supplied in coils containing $\frac{1}{2}$ cwt. or 1 cwt. of material from which short lengths are cut as required.

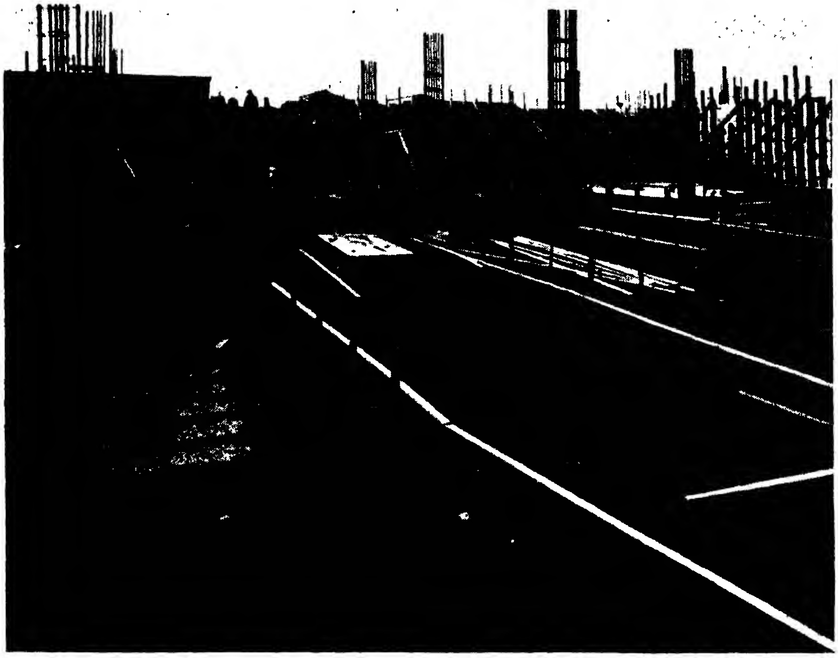


Fig. 92.—Ground Slab Reinforcement.

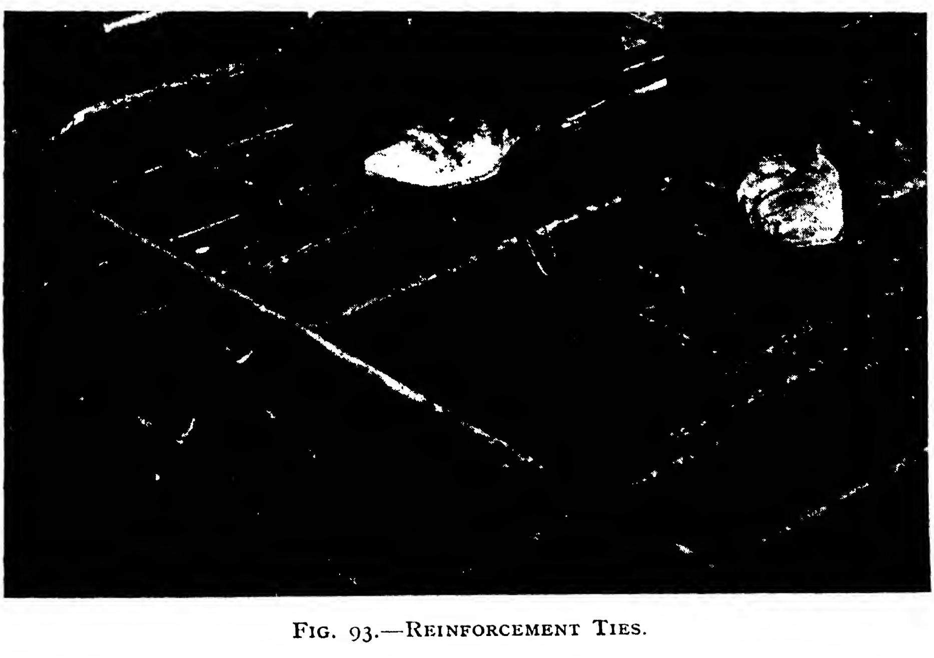


Fig. 93.—Reinforcement Ties.

In the simplest work, say for wiring the intersection of two slab or wall bars, the length of wire required would be 4 to 6 in. This is bent into the form of a **U** which is placed around the intersection of the two bars, the protruding ends of the wires being then twisted together with a pair of pliers until the tie tightly embraces the bar. If the reinforcement is accessible a steel fixer can wire up to a dozen intersections per minute. This work is facilitated by the use of special wire ties and clips, one type of which is shown in position, together with steel chairs supporting slab bars, in *Fig.* 93.

It is advisable to see that the projecting ends of wire ties are not in contact with the shuttering, otherwise objectionable rust stains, which are difficult, if not impossible to remove permanently, will appear on the face of the concrete. This defect can be avoided by not using more wire than is necessary to make a secure fixing, and by bending the projecting ends of the wire away from the shuttering, or better, by twisting the projecting ends around one of the adjacent bars. With the latter method there is little danger of the wires being forced against the shuttering when the concrete is tamped.

Concrete Cover and Bar Spacing.

The minimum thickness of concrete between any reinforcing bar and the face of the concrete and the minimum distance between adjacent bars are usually specified or determined by the reinforcement details. Otherwise the following minimum covers should be observed:

Slabs and Walls: Not less than the diameter of the bar or less than ½ in. for floors and panel walls. Not less than ¾ in. for work subject to water pressure, in contact with filled earth, or subject to abrasion. Not less than 1 in. for slabs laid directly on the ground.

Beams: Top and bottom cover over main bars not less than the diameter of the main bars or less than 1 in. Side cover to main bars not less than 1 in. for all diameters of bars, or preferably not less than the diameter of the bar. Not less than ½ in. over binders.

Columns: Not less than the diameter of the bar or less than 1 in. for columns less than 12 in. square and not less than 1½ in. for columns 12 in. square and upwards. Not less than ½ in. over binders.

Piles: Not less than 1½ in. to main bars.

Sea Work: Not less than 2 in. over any reinforcement.

The durability of any structure depends considerably upon adequate cover being provided; unsatisfactory concrete work is often due to insufficient protection to the reinforcement, especially in water-containing structures, sea and river work, or structures associated with certain industrial processes. The space between adjacent parallel bars, as in beams, should be sufficient to ensure that the concrete can pass between and around the bars and to enable the bar to obtain adequate bond in the hardened concrete. The clearance horizontally between any two bars should therefore be at least equal to the maximum size of the aggregate plus a nominal tolerance to allow easy passage, or equal to the bar diameter, whichever is greater. Thus with ¾-in. maximum gauge aggregate the clearance should be a minimum of 1 in. This clearance should also be sufficient if 1-in. diameter bars are used, but must be increased if larger diameter bars are provided. Some regulations require the minimum clearance to be equal to one-and-a-half times the bar diameter and other specifications are content with $\frac{3}{4}$ in., apparently irrespective of the bar diameter.

The minimum vertical distance between any two horizontal bars is usually specified as being $\frac{1}{2}$ in., but where $1\frac{1}{4}$-in. diameter or larger bars are concerned it is preferable to increase this minimum to $\frac{3}{4}$ in. or 1 in. In order to maintain this clearance it is advisable to introduce spacer bars at 3 ft. or 4 ft. centres between superimposed layers of beam bars. Where the bars from intersecting beams thread between the layers of beam bars, special spacer bars may be neither necessary nor effective. Where horizontal bars lap vertically one above another

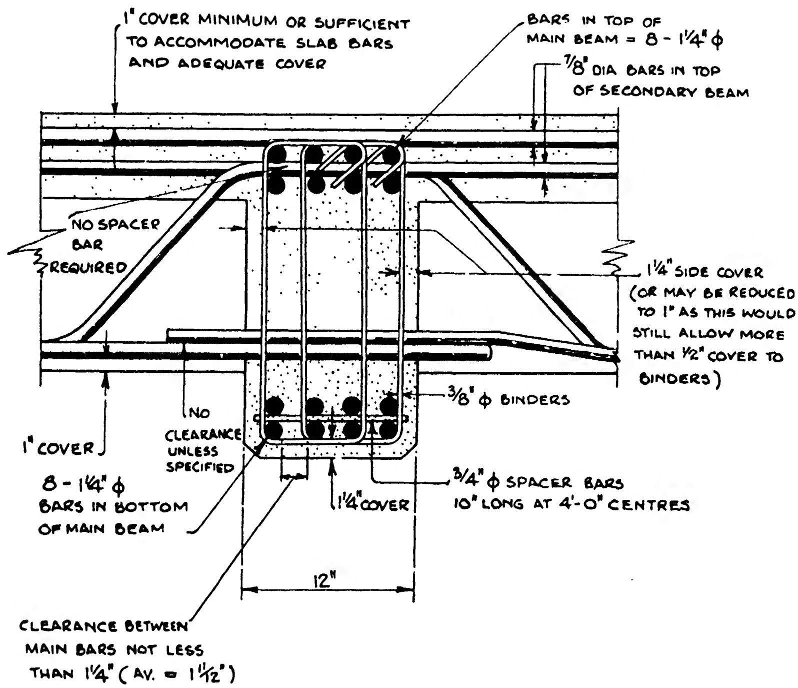


FIG. 94.—BAR CLEARANCES AND COVERS.

the clearance should be at least equal to the diameter of the smallest bar, although in certain cases the designer will permit such bars to be in contact. Where bars pass each other approximately at right angles there is no objection to their being in contact. *Fig.* 94 illustrates a beam intersection and indicates the various clearances, spacer bars, and covers necessary for this particular case.

After reinforcement has been assembled it is necessary to insert "stops" or cover blocks between the bars and the shutters to secure and maintain the requisite cover of concrete. The best method is to use blocks of concrete or mortar, which can be readily made on the site to thicknesses conforming to the minimum specified covers. If a long strip, about an inch wide, is cast, pieces can easily be broken off if the surface is scored by a trowel while the mortar is still soft. Alternatively, if rectangular blocks ½ in. thick by 1 in. wide by 1½ in. long are cast they are adaptable for slab, beam, or column work requiring ½ in., 1 in. and 1½ in. of cover respectively as shown in *Fig.* 95. If a piece of wire is cast into the block the latter can be securely fixed to the reinforcing bar as illustrated, thus preventing the block from falling out of position during concreting of columns, walls, or other vertical work. Concrete cover-blocks can be left permanently in position, and are therefore preferable to wood or steel wedges which must be withdrawn as concreting proceeds with consequent risk of reduction of cover due to movement of the bar. Metal hangers or asbestos collars are sometimes used, and can also be left in position. The practice of using a piece of aggregate is not recommended on account of the uncertain amount of cover attained and the difficulty of preventing the stone from falling out of position.

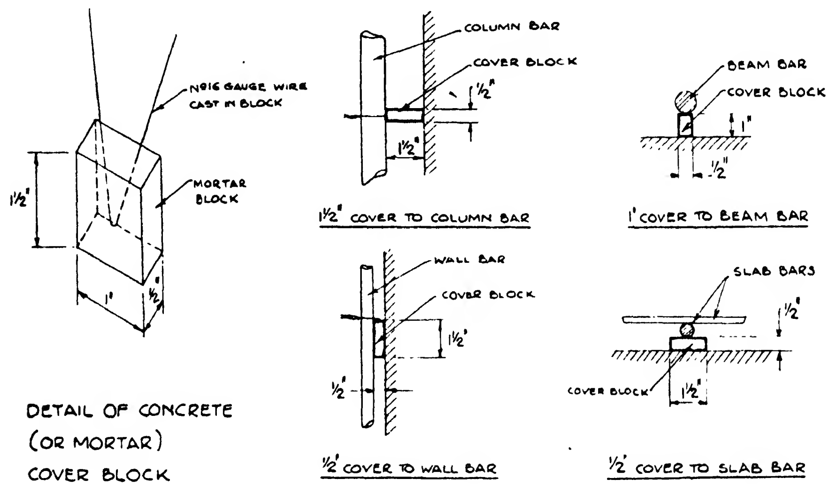


FIG. 95.—CONCRETE COVER BLOCKS.

When blocking out beam and column reinforcement it is preferable to insert the block under or at the side of the main bars and not against the binders, as the latter are apt to spring and reduce the cover intended for the main bars. For slabs laid on the ground the cover is frequently maintained by placing timber laths under the assembled mesh of bars. The laths, which must be withdrawn as concreting proceeds, are in cases of reinforcement laid on the ground preferable to the isolated support offered by blocks owing to the uneven surface of the ground. Where the ground surface is blinded with a lean concrete layer, or where slab reinforcement is laid on shuttering, separate concrete cover-blocks placed at intervals under bar intersections are sufficient. This is illustrated in *Fig.* 96, where the slab shuttering is lined with an insulating layer of cork. This illustration also shows horizontal electricity conduits buried in the concrete slab beneath the reinforcement. If laid as shown there is no objection to the bars being in contact with the conduits. It is not good practice to allow the conduits to be in contact with the shuttering, and a thin layer of concrete should be interposed between the conduit and the slab soffit. This may lead to an excessive cover being provided on the bottom bars. For this reason when large-diameter conduits are installed they are better placed above the bottom layer of steel,

Fig. 96.—Cover Blocks for Slab Reinforcement.

the latter being given the minimum specified cover. These remarks also apply to heating coils that are sometimes buried in concrete floor slabs where panel heating is installed.

Welding.

As the London Building Act permits welding, this method of joining reinforcement bars will be more common in cases where the alternative of overlapping bars leads to difficulties of design and construction. In general the positions of the welded joints would be specified by the engineer, and the main task of those on the site is to see that welding processes and equipment of proved reliability are used and that the recommendations of the appropriate British Standard Specification and the London Building Act are followed. For butt welding of round bars electric arc welding is the most common and economical method.

Before welding commences the ends of the bars must be rigidly held in the correct position relative to each other, the ends being shaped to render the deposition of the weld metal in full contact with the ends practicable, to ensure access for the electrode, and to enable the welder to see the work clearly. Owing to the restriction in most beams, bridge girders, etc., of being able to work from the top only, the most usual form of end is a single **V** or **U**. Before depositing the weld metal in such joints a small plate as shown in *Fig.* 99 and about $\frac{1}{16}$ in. thick is sometimes spot-welded on the underside of the joint and acts as a seating upon which the weld can be built. For large bars a single **V** necessitates a large amount of weld metal, therefore a double-**V** joint, which does not require the use of a plate and effects a saving in metal and time, is more suitable. If the bars being connected can be rotated, as in the case of long straight pile bars, etc., full advantage of a double joint can be obtained, but if access to the work is only available from the top, a single **V** should be used. It is usually necessary in such cases, when a plate is not provided, to form an additional thickness of weld metal on the lower half of the joint, otherwise a point of weakness occurs at the lowest point of the joint and a reduced working stress has to be adopted. A butt weld should be made about 10 per cent. thicker at the centre than the thickness of the bars being welded. If a smooth joint is necessary this additional thickness should be first formed and then dressed off.

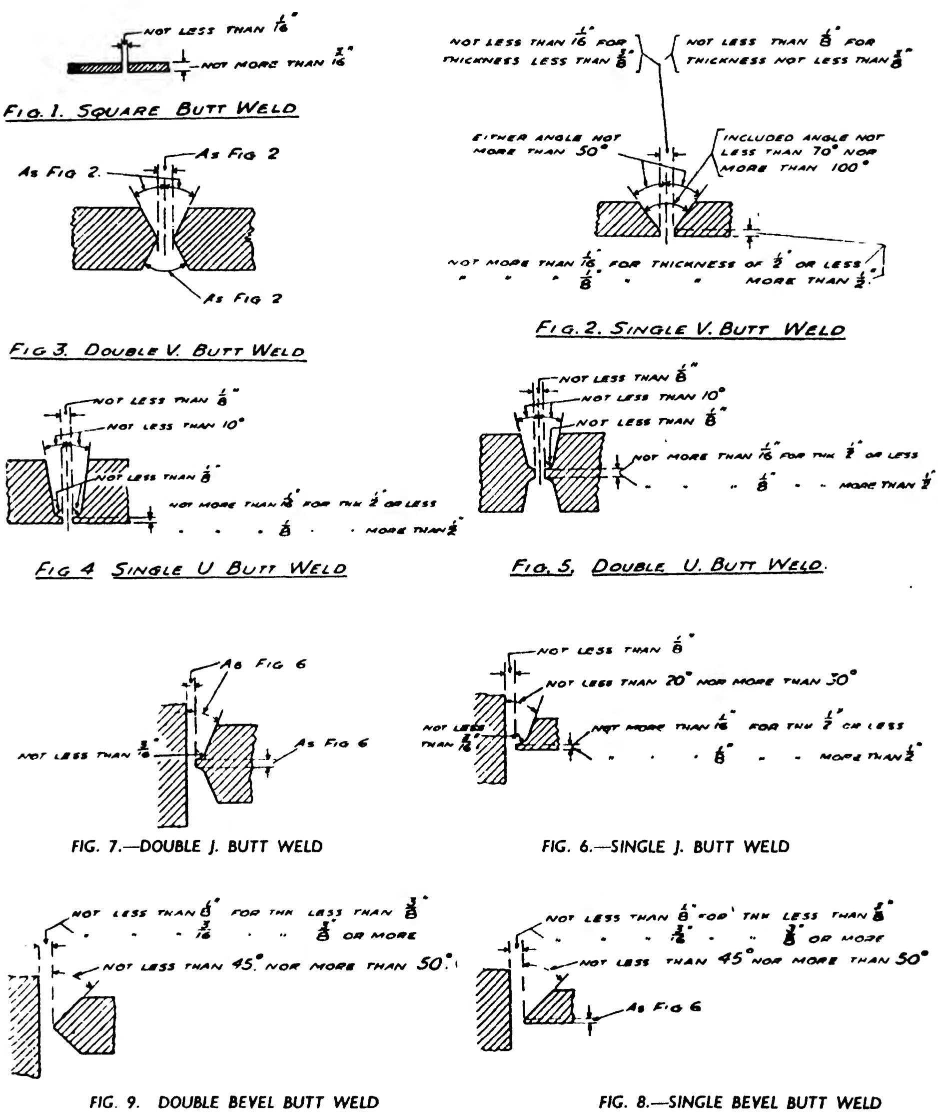


FIG. 97.—FORMS OF WELDED JOINTS.

The forms of joint recommended by the London County Council are illustrated in *Fig.* 97, and in tests authorised by the Council it was found that for bars larger than 1½-in. in diameter a double-**V** or a double-**U** butt weld is preferable to a single joint, while for bars 1⅛ in. to 1½ in. in diameter either a single or double joint is suitable. A single-V butt weld is preferable for bars 1 in. or less in diameter. When bars of unequal diameter are connected by butt welds, a single bevel butt weld appears to be most suitable.

A square butt weld should not be used if the thickness of bars being jointed exceeds $\frac{3}{16}$-in., and J-butt welds and bevel butt welds should be avoided where possible as their use entails a reduction in the working stresses. The minimum gaps that should separate the ends of the bars being welded are shown in *Fig.* 97, together with other dimensions.

The ends of the bars are shaped by shearing, clipping, machining, or by flame, and the process is best performed before erecting the bars in position. Machine cutting is preferable to cutting by flame, and the latter should only be resorted to when machine cutting is impracticable. The joint and a length of

Fig. 98.—Welding Reinforcement.

bar at least $\frac{1}{2}$ in. from the joint should be chipped and brushed clean of loose metal, heavy rust, dirt, grease, paint, etc., before commencing to deposit the weld metal. If the faces of the joint are irregular, they should be dressed by chipping, grinding, or filing.

An illustration of the process of applying the weld metal is given in *Fig.* 98, where a joint in bars fixed in position in a bridge girder is being formed. During the process the bars are held rigidly in position either by wiring to the beam stirrups or by attachment in a jig or clamp. A No. 8 gauge electrode is most suitable for reinforcement bars up to 2-in. diameter, and the joint is built up by depositing the weld metal in successive runs. It is essential that the operator and the supervision should be reliable and competent. The weld should be uniform and continuous, the surfaces having an even contour and regular finish. If cavities are evident, or if the edges of the weld metal fall over the top of the bars which are being welded, the weld should be cut out and re-welded. After making each run all slag should be removed by light hammering and wire brushing.

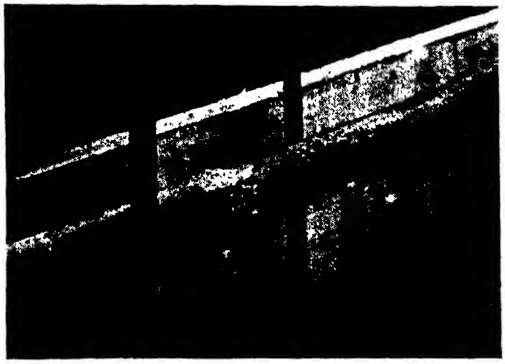


FIG. 99.—BUTT-WELDED JOINT.

A completed single-**V** welded butt joint is illustrated in *Fig.* 99. The time taken to prepare and complete a single joint of this type varies from 20 to 45 minutes, depending on the diameter of the bar which will also affect the amount of metal to be deposited.

SECTION IV.—COST OF REINFORCEMENT.

Cost of Material.

The cost of reinforcing material is principally a function of the diameter, small diameter bars being dearer per ton than large diameter bars. The basic price is usually that for $\frac{5}{8}$ in. diameter bars, and all larger sizes in lengths between 5 ft. and 40 ft. are supplied at this rate, while, within the same length limits, for bars less than $\frac{5}{8}$ in. diameter the charge is increased by about 5*s*. for each successive $\frac{1}{16}$ in. less than $\frac{5}{8}$ in. Bars cut to length are slightly more expensive than stock lengths. The cutting tolerance claimed by the mill is 1 in. under or over the specified length. When a minimum length is specified the tolerance is 2 in. over and nothing under. When a maximum length is specified the tolerance is 2 in. under and nothing over. Bars supplied directly from stock are more expensive than similar bars delivered from the mills. Short bars cost more than normal lengths, the increment over the normal price for a given diameter usually being 5*s*. per ton for lengths from 4 ft. to 5 ft., 10*s*. for lengths from 3 ft. to 4 ft., and 15*s*. for 2 ft. to 3 ft.

The saving in the cost of bars supplied in long lengths may be offset by costs for cutting on the site and waste ends, or by the extra cost of carriage per ton if the quantity required is not sufficient to secure the most favourable rates. Bars exceeding 40 ft. in length are subject to an extra charge over the basic rate, the increment being of the order of 10*s*. a ton for bars from 40 to 45 ft. in length, 15*s*. a ton from 45 to 50 ft., and then 15*s*. plus 1*s*. 6*d*. a foot above 50 ft. Subject to this extra charge bars can be obtained up to 75 ft. long, although only infrequent cases of long precast piles, long-span beams, or arches warrant such abnormal lengths.

If long bars are dispensed with where otherwise they could be used it is necessary to overlap the shorter bars to obtain sufficient bond. These laps increase the weight of steel required, and if the ends of the bars are hooked additional labour is involved. These extras offset to a certain extent the extra cost of long bars, and for any given case it is advisable to compare the cost of short lapped bars and overlength bars without laps, using in the calculations the known material and labour rates. Generally it will be found that beyond 50 ft. it is cheaper to use lapped bars. The question of sufficient tonnage to warrant special

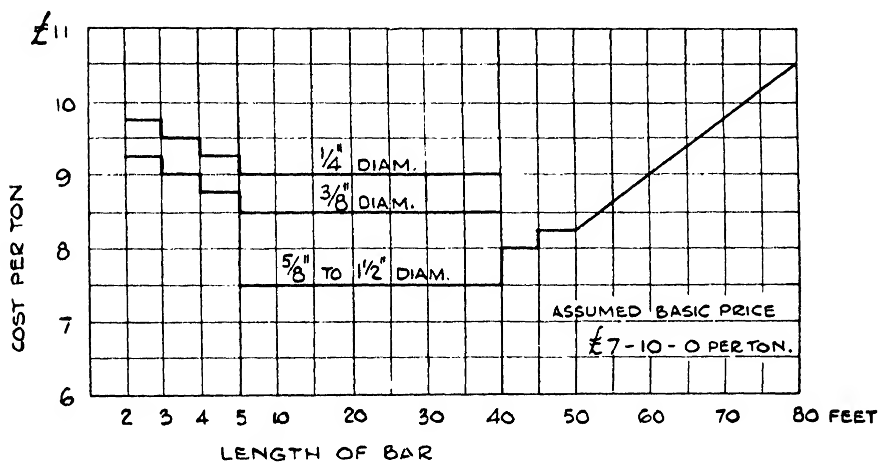


FIG. 100.—COST OF REINFORCEMENT BARS.

transport facilities for long bars, and the handling of very long bars on the site, must also be considered.

Long bars can also be replaced by short bars by the use of welded connections or by providing turnbuckles. Neither of these methods is cheaper than lapping two bars together, but they are sometimes necessary when restricted space in either the width or depth of the member concerned does not permit the additional clearances required by lapped bars. Butt-welded connections are allowed by some authorities, but the incidence of such connections is limited, usually to one at any cross section, with a restriction on the distance apart of welds in adjacent bars. Screwed turnbuckle connections are expensive, as in addition to the provision of the turnbuckle and the operation of threading the ends of the bars there is the added cost of forging and jumping the end of the bar so that the area at the bottom of the threads shall not be less than the normal area of the bar.

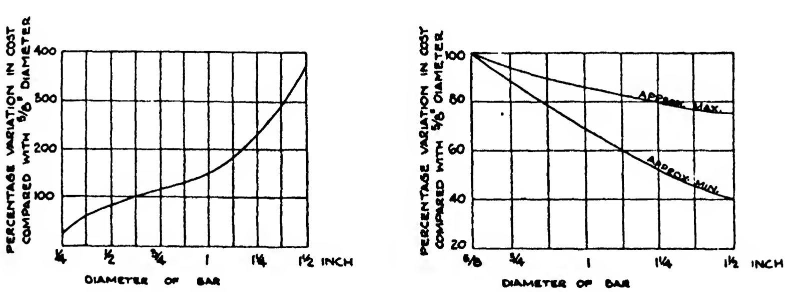
To summarize the foregoing observations on the effect of diameter and length on the net cost of reinforcing bars (material only) the curves on *Fig.* 100

have been prepared. These show the variation in the cost per ton delivered of bars from ¼ in. to 1½ in. diameter and from 2 ft. to 80 ft. long, assuming that the basic price is £7 10s. per ton. Additions or deductions to this rate would, with the usual rates for extras, accordingly increase or reduce the rates shown by like amounts.

In considering weights of reinforcement, the "rolling margin" of rolled mild steel bars must be taken into account. For bars ½ in. and upwards the difference between calculated and actual weights may be 2½ per cent., the latter weight being the larger. For small bars the rolling margin may be up to 5 per cent. That is to say, for buying purposes, a calculated weight of 100 tons of large bars may weigh 102½ tons, and the latter weight would be charged for. Some merchants reserve a rolling margin of ± 2½ per cent. on the calculated weights taken over the whole order, or plus 5 per cent. when a minimum weight is specified, or minus 5 per cent. when a maximum weight is specified. For sizes under ¼ in. the tolerance is ± 5 per cent.

Bending and Fixing Costs.

The cost of forming a single bend is greater for large than for small diameter bars, and curve (i), *Fig.* 101, indicates the approximate variation taking ⅝-in. bars as the basis. This basic cost will be determined by the type of machine employed, the efficiency of handling arrangements, and the tonnage to be dealt with. It is therefore of little value to give actual figures; one penny per bend



(i) Cost of Forming Bends and Hooks.

(ii) Bending Costs per Ton for Large Diameter Bars.

FIG. 101.—BENDING COSTS.

is often considered an average, but wide variations from this figure are possible. The formation of a semi-circular hook may cost almost twice as much as bending through 90 deg. or less.

Although the cost per operation increases with increase in diameter, the bending costs per ton of reinforcement in normal work decreases with an increase in the diameter, and the curves (ii) (*Fig.* 101) indicate the limits within which such relative costs may ordinarily lie for large bars. The number of bends on each bar and the length of the bars will obviously affect the actual cost on any job. Thus pile reinforcement that may only involve forming a single crank in long bars of large diameter might cost only a few shillings per ton, whereas the cost of bending doubly-cranked bars of large diameter might exceed £1 per ton.

If unloading, stacking, cutting, bending, hoisting, assembling, and fixing with tying wire are included, together with machine charges and profit, the overall labour and plant cost per ton of reinforcement does not vary appreciably for bars of ⅝-in. diameter and upwards in ordinary building work, and for the purposes of priced bills of quantities the reinforcement is usually more conveniently divided into items for ⅝ in. bars and over and for each diameter less than ⅝ in. The variation in cost for bars below ⅝ in. may be similar to that shown in *Fig.* 102. The basic labour and plant cost for ⅝-in. bars might be anything from £1 15*s*. a ton for simple reinforcement as in piles to £3 15*s*. a ton for building work, and even more for tanks, bunkers, and other structures involving more intricate bending and placing.

Combined Cost.

When the costs of material and labour are combined the trend of the variation in overall cost of reinforcement is similar to that shown in *Fig.* 102, the higher cost of the small work being because the small diameter bars are more

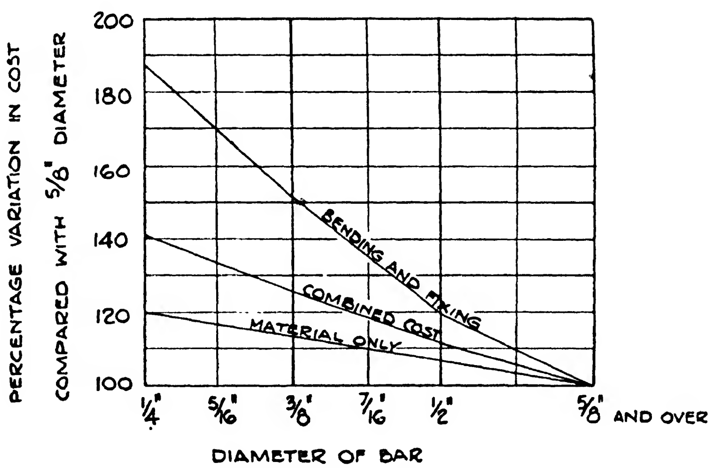


FIG. 102.—REINFORCEMENT COSTS.

costly to buy and involve a greater labour charge than large diameter bars. Taking ⅝-in. and ⅜-in. bars in normal lengths as examples, the overall costs may be calculated as follows:

		⅝ in. £	s.	d.	⅜ in. £	s.	d.
Cost of material delivered	per ton	7	10	0	8	10	0
10 per cent. profit on material	,,		15	0		17	0
Labour charges, including unloading, bending, fixing, tying wire, general charges, and profit on these items	,,	3	15	0	5	12	6
Totals	per ton	£12	0	0	£14	19	6

In closely estimating reinforcement costs, however, such factors as extreme length of bars, average length of bars, number of hooks and bends per ton of steel, amount of repetition bending, accessibility for assembling and hoisting, and the tonnage to be dealt with must be considered in addition to the diameters of the bars concerned since they can influence the overall cost to such an extent that this may be anything from £9 to £20 a ton when the basic price is £7 10s. per ton. The supply of reinforcing bars from stock and the inclusion of design costs (as is sometimes the case) in the reinforcement rates will also influence the latter appreciably.

CHAPTER IV

SHUTTERING

SECTION I.—PRINCIPLES OF CONCRETE SHUTTERING.

Construction of Shuttering.

THE problems of shuttering and of constructional procedure are interwoven to such an extent that one cannot be considered apart from the other. In this chapter, therefore, consideration is given to the principles underlying the design and construction of shuttering, the application of these principles to particular structures being dealt with in later chapters.

The engineer's requirements are usually expressed in the specification, a concise clause embodying these reading as follows:

" All shuttering is to be substantially and rigidly constructed and efficiently propped and braced to prevent deformation due to the deposition of the concrete or due to other loads to which it may be reasonably subjected, and to prevent distortion due to effects of the weather or otherwise. Boarded shuttering must be treated to obtain a fair surface and be tight against leakage of cement grout. The shuttering must be constructed to facilitate easing and removal of the various parts in the recognised sequence without jarring the concrete, and no shuttering or strutting must be struck until the concrete has attained a satisfactory strength."

During the development of the practice of reinforced concrete, standard methods of treating the more common problems associated with shuttering have been produced to attain the desired rigidity and tightness, the production of fair surfaces, and ease of removal.

Of the total cost of a reinforced concrete structure the cost of the shuttering and temporary staging may be anything up to 30 per cent. and may be considerably in excess of this proportion in bridge work and high-level roofs where extensive centering is required. It is therefore obvious that any economies made in shuttering can affect overall costs appreciably and such economies are most readily made in the following directions.

(i) By making calculations to determine adequate sizes for the various components of the moulds and supports. This avoids both waste of material and the use of undersized boards and props that due to distortion or collapse may lead to expensive replacement.

(ii) By constructing the shuttering in such a way as to involve a minimum of cutting and the least damage to the timber in order to allow of re-use and to ensure a good salvage value at the completion of each stage of the work.

(iii) By arranging for as many uses of one set of moulds as possible. The designing engineer can considerably assist in this direction by reducing variations of sections to a minimum, by avoiding superfluous breaks, and by providing simple profiles that, even at the expense of a little more concrete, may lead to economical shuttering.

The quality of the shuttering considerably affects the quality of the finished concrete surface. The opinion of most practical men is that when a fair surface is required it is cheaper in the long run to use shuttering well made by skilled joiners working on wrought material, rather than indifferently constructed shuttering formed from rough timbers which necessitates much making-good.

Experience has established for normal construction suitable sizes for shutter boards together with sizes and spacings for their supports in order that the shuttering shall be strong enough to carry the loads imposed upon it and restrict the deflections within limits that will avoid deformed surfaces in the concrete work. A certain amount of deflection is unavoidable, and a limit of $\frac{1}{16}$ in. is sometimes specified although a maximum of $\frac{1}{8}$ in. is a practicable limit. To ensure that long-span centering shall not sag unduly an upward camber is usually provided, and this may amount to $\frac{1}{2}$ in. or 1 in. in a 20-ft. span depending on the size of the beam.

Where special calculations are not prepared, the following nominal sizes are common:

Boards for slabs: 1 in. or $1\frac{1}{4}$ in.; for heavy slabs, 2 in.

Joists supporting slab boards: 3 in. by 2 in. and upwards to 9 in. by 3 in., depending upon the span, load, and spacing of joists.

Bearers supporting slab joists: 3 in. by 2 in. or 6 in. by 1 in. and upwards, depending mainly upon the limiting bearing pressures and moments.

Beam sides: 1-in. boards.

Beam bottoms: 1-in. or 2-in. boards depending on the method of construction and the spacing of the props.

Props to beams and slabs: 4 in. by 3 in., 4 in. by 4 in., and similar sizes.

Column boxes: 1-in., $1\frac{1}{4}$-in. or $1\frac{1}{2}$-in. boards with 3-in. by 2-in., 4-in. by 2-in. or 6-in. by 4-in. yokes.

Walls: $1\frac{1}{4}$-in. to 2-in. boards, depending upon the spacing of the supports and the pressure to be resisted. For small panels 1-in. boards are used.

Sizes smaller than these may require an uneconomical amount of bracing. Where possible it is preferable to compute suitable sizes for the various parts of the shuttering in order to avoid excessively light or unnecessarily strong construction.

Data relating to the design of shuttering for floors, columns, and walls are given on *Table* XI.

Materials.

Shuttering is constructed in timber or steel, or these materials in combination. In this country the timber commonly employed is yellow pine, red wood, fir, spruce, and similar soft woods. It should be free from loose knots and well seasoned, especially if it is liable to be exposed to the weather for any length of time after it is made up. Seasoned timber should be well wetted before placing the concrete in order that the wood shall not absorb moisture from the wet concrete. For this reason unseasoned timber has been recommended for shuttering that is to be filled immediately after it is made up, but such timber cannot be repeatedly used without costly remaking and trueing up. Newly-felled timber should be left in air to season for at least six weeks before use. Unsound or very soft timbers should not be used, as distortion at bolted connections may occur. The lengths in which timber suitable for shuttering is supplied commercially vary from 10 ft. to 24 ft. with 15 ft. lengths predominating.

Boards for exposed concrete surfaces should have shot edges, and planed faces in contact with the concrete; they should be thicknessed to eliminate irregularities at board junctions. Where concrete surfaces are not exposed in the completed work, as in foundation work, undressed timber can be used with a saving in cost over wrought boards. Adjacent boards for exposed faces should be firmly in contact to render all joints tight to prevent leakage of cement grout and mortar; this is important, as it is very necessary that all the cement should remain in the concrete. Boarding that is continuously in contact with water or wet concrete, as in moving shutters, may, however, spring if the joints are so tight as not to allow for slight swelling. Any faulty joints in boarding should be caulked or the interior of the moulds should be lined.

Tongued-and-grooved boarding makes a thoroughly tight joint and one that will produce a minimum amount of blemishes due to bad joint marks. Such boarding is more expensive than plain boards, and since thinner boarding is generally used more stiffeners are required. The tongues being easily broken, remaking dismantled shutters is a doubtful economy. Tongued-and-grooved boarding is used to its best advantage when wall panels can be made up so that many re-uses are possible without alteration or repair.

Linings.

Linings for shuttering to produce smooth surfaces may be waterproofed building paper, grainless wood, plywood, or similar materials. If paper is used, the shutters should be made with fairly close joints. The paper is fixed by lightly tacking with flat-headed nails, either from the inside of the shutter or by bending the paper over the edge and nailing from the outside. If a durable material is obtained repeated uses are possible. Suitable waterproofed paper can be obtained in rolls 100 yd. long by 50 in. wide. For simple shuttering, the use of sawn boarding with paper linings in place of unlined wrought boards may be worth consideration. When using steel rammers or tampers in paper-lined shutters care must be taken not to tear the paper. Where only small areas have to be dealt with, for example, at an awkward connection of inclined beams at a column head, a paper lining on the shuttering reduces the amount of work required in shaping the timbers. Paper cement bags have been successfully used for this purpose.

When plywood or grainless-wood linings are employed it is not always necessary to use close-boarded shutters, it being sufficient to place the boards near enough together to prevent local deformation of the lining. The spacing which is therefore determined by the head of concrete and the thickness of the lining is shown in the illustrations of typical examples in *Figs.* 103, 104, and 105. Owing to the high cost of thicker plywoods and their relative weakness

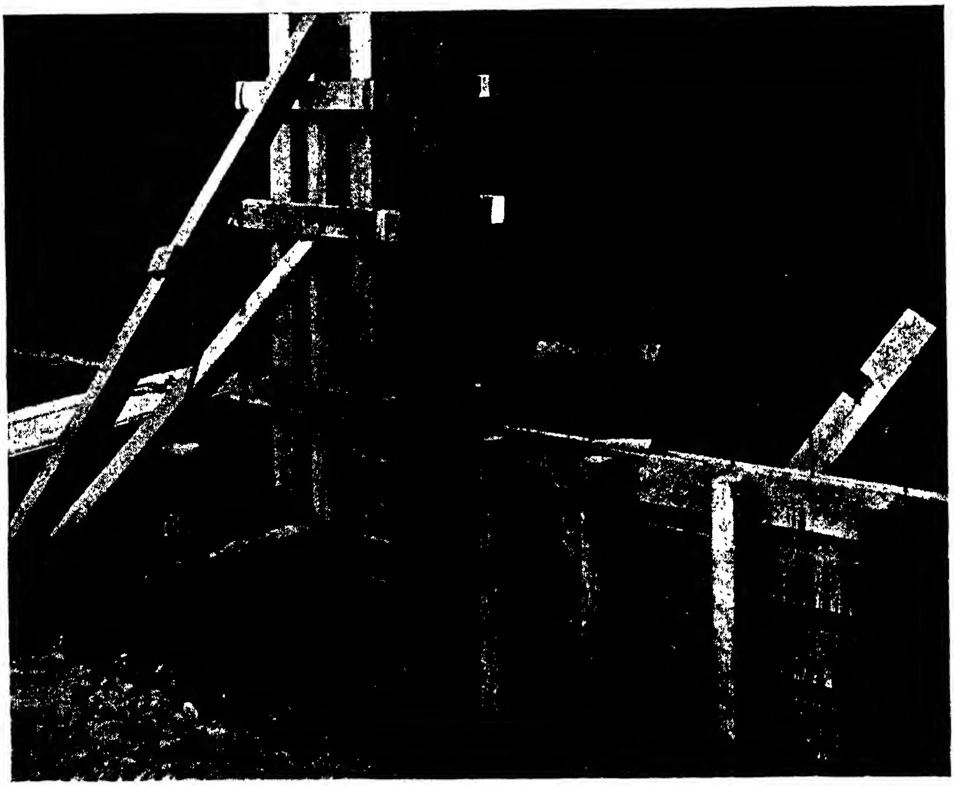


FIG. 103.—WALL SHUTTERING WITH GRAINLESS WOOD LINING.

compared with 1-in. plain boards, it is better to use a thin plywood with closer-spaced supporting boards rather than attempt to economise in the latter at the expense of the lining. The timbering for the wall shuttering in *Fig.* 103 consists of 6-in. by 1-in. boards with a clear gap of 2½-in. between adjacent boards. The verticals are 4-in. by 3-in. timbers spaced at 16-in. centres. The lagging to the curved haunches of the flooring in *Fig.* 104 is 3-in. by 2-in. with about 2½-in. gaps, in this case the lining consisting of the sides of old tea chests. The panels of shuttering illustrated in *Fig.* 105 are 2 ft. 3 in. high and consist of three 7-in. boards connected with 4-in. by 2-in. soldiers spaced at 2-ft. centres.

FIG. 104.—PLYWOOD LINING TO CURVED SHUTTERING.

FIG. 105.—SHUTTER LINING.

Plywood and similar linings are of particular value when shuttering plain curved surfaces since, as shown in *Fig.* 104, flexible sheets readily conform to the required radius. Shutters lined with plywood or grainless wood can be used many times without repairing the lining, the number of uses depending on the quality of the lining. Further consideration of linings for obtaining fair finishes is given in Chapter VIII.

Steel Shutters.

The use of steel shutters is warranted if a sufficient number of uses is obtainable. Some contractors devise and construct their own steel shutters for special contracts, but proprietary makes are available in sufficient variety of sizes to meet

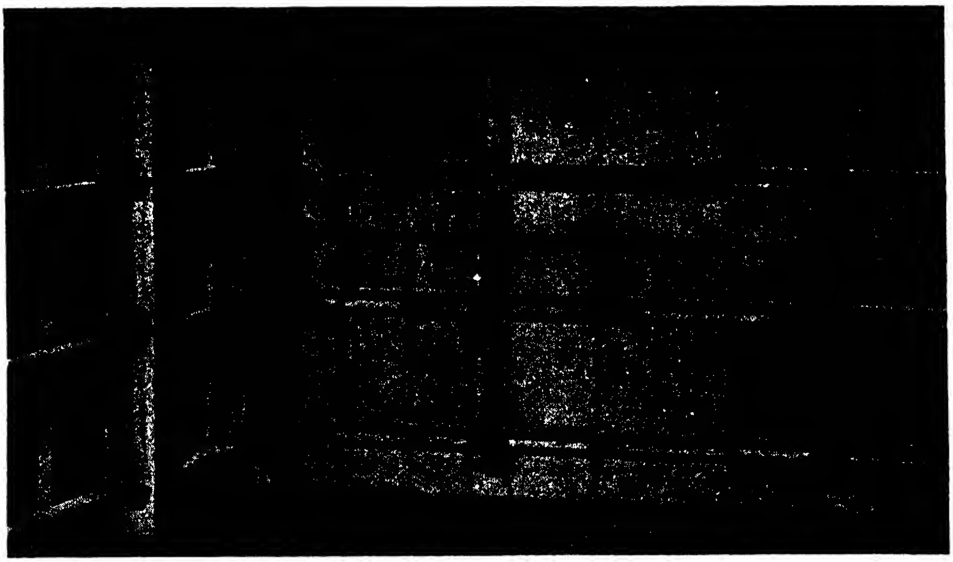


FIG. 106.—STEEL WALL SHUTTERS.

most normal structural problems. Typical examples are illustrated in *Figs.* 106 and 107.

Steel shuttering usually consists of thin plates stiffened along the edges by small steel angles, an assembly of plates being held together by two or more clamps or bolts along each edge and kept in alignment by horizontal or vertical liners of steel or timber. The usual size of wall panel is 2 ft. square, but sizes down to less than 1 ft. are obtainable as standard and in some cases plates up to 2 ft. by 4 ft. can be obtained. The panels are supplied straight or curved. For circular work of 20 ft. radius and more straight panels are suitable (*Fig.* 107). The provision of sliding plates, shaped closure pieces, and similar items enables steel shutters to be adapted to tapered walls, circular or straight on plan. A detailed discussion is given in Chapter X of the use of steel shutters for tank construction.

Steel moulds are particularly attractive for such details as the splayed heads of columns when a large number of identical units is involved, and for culverts, flumes, tunnels and retaining walls, where a constant section is maintained for a considerable length. The first cost of steel shutters is higher than that of timber for given work but, owing to their greater durability, ultimate economy results if a large number of uses can be obtained. Some cost data are given later.

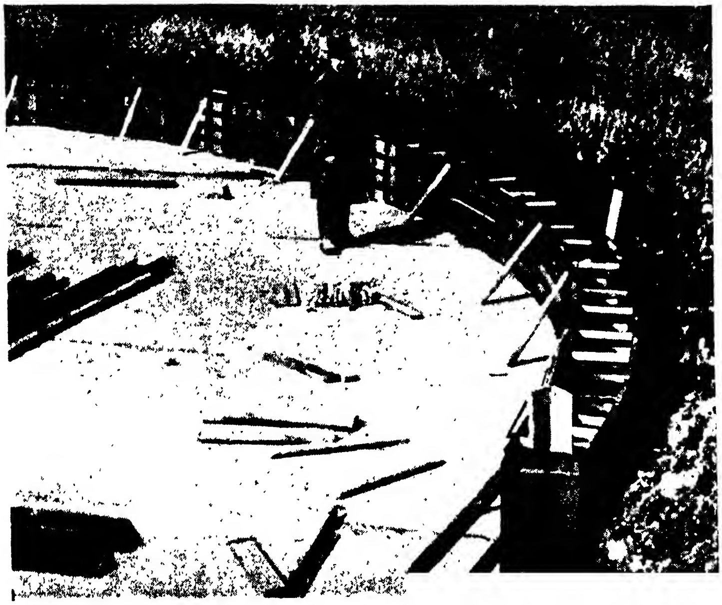


FIG. 107—STEEL SHUTTERING.

Loads and Pressures.

Centering for floors and similar horizontal construction is subject mainly to vertical loading arising from the weight of the concrete, reinforcement, and any superimposed loads due to workmen, barrows, stores, and other incidental loading. An allowance of 100 lb. per square foot is usually ample for superimposed loads in heavy work, and for light floors and roofs an allowance of 75 lb. per square foot or less is sufficient. The weight of the centering is included in these loads. The dead weight of the concrete and reinforcement can be taken as 144 lb. per cubic foot, although this may vary, being in some cases as high as 165 lb. per cubic foot.

The principal loading on vertical shutters for walls, columns, and similar work is due to the side pressure of the wet concrete, and the magnitude of this pressure depends upon the characteristics of the concrete, especially the consistency, the amount of ramming, the temperature, the period between mixing and placing, and the depth of the deposit. Possible values of this pressure under various heads of wet concrete of normal consistency and cement content (that is, a 1 : 2 : 4 mix and 6-in. slump) are given on *Table* XI. These pressures will be increased if the temperature is below normal (when hardening is retarded) and for increase in richness of mix, and will be decreased for dry mixes and for work in narrow widths or where the reinforcement is particularly closely spaced. Drier mixes give lower pressures than more fluid mixes, but the extra tamping required with dry mixes may result in lateral pressures in excess of the static fluid pressure. Although drier mixes are associated with concrete placed with vibration, the action of the vibrators, especially of the internal type, is so to liquefy the concrete that full hydrostatic pressures may be experienced. This would result in a pressure of at least 155 lb. per square foot per foot depth of unset concrete and the extra forces to which the moulds are subjected when vibrators are used necessitate a general strengthening of the shuttering. If the shutters are being continuously filled, the horizontal pressure near the bottom increases as the head of wet concrete increases up to a maximum beyond which a decrease back to zero takes place as the lower layers of concrete harden. A comprehensive expression for assessing the maximum pressure that seems to be in reasonable agreement with common practice in designing shuttering for continuous placing is $P = KR^2 + C$, where P is the maximum pressure in lb. per square foot, R is the rate of continuous vertical placing in feet per hour, K is a factor that varies with the temperature, and C is a factor depending upon the mix. Suitable values of K and C are as follows:

Temperature (deg. F.)	40	45	50	55	60	65	70	75	80
Value of K	70	60	50	40	35	29	25	22	20

Mix	1 : 3 : 6	1 : 2 : 4	1 : $1\frac{2}{3}$. $3\frac{1}{3}$	1 : $1\frac{1}{2}$: 3	1 : 1 : 2
Value of C	0	70	100	120	170

Consistency is not taken into account in the expression, it being assumed that the extra pressure of fluid mixes offsets the pressure due to the tamping of drier mixes.

Applying this expression to the case of a 1 : 2 : 4 mix, a temperature of 65 deg. F., and a rate of placing of 2 ft. vertically per hour, without vibration, the maximum pressure is $P = (29 \times 2^2) + 70 = 186$ lb. per square foot. If the concrete were placed to a depth of 3 ft. almost at once, the maximum pressure would be $3 \times 140 = 420$ lb. per square foot (see data on *Table* XI) which is considerably greater than the pressure obtained by gradually filling the shuttering and calls for more substantial construction.

Design Data.

Table XI gives the essential data for the design of shuttering elements, including the loads and pressures. The working stresses specified provide a sufficient margin of safety against reasonable accidental overload and assume that the more common woods such as fir, spruce, and yellow pine are used planed. The relative values for other timbers are given on p. 26. The working stress of 1,200 lb. per square inch in compression or tension due to bending assumes that the stress acts parallel to the grain, while the shear stress of 200 lb. per square inch is satisfactory if the shear force acts across the grain. For direct compression along the grain, as in posts or props, 1,000 lb. per square inch is taken as a basis, but this only applies to " short columns " and should be reduced to the values tabulated for members having a high ratio of length to least breadth. The safe compressive stress of 400 lb. per square inch across the grain controls the design of the end bearings of joists and similar members and timber sole-plates under posts. If the direct stress in a post or strut exceeds this figure a metal or hardwood bearing-plate should be inserted between the post and its bearing.

Although the modulus of elasticity of timbers varies considerably, a value of $1{\cdot}2 \times 10^6$ is satisfactory for deflection calculations relating to shuttering. The calculated deflection, as explained before, should not exceed $\frac{1}{8}$-in., or in special cases $\frac{1}{16}$ in.

The units given on *Table* XI and the subsequent calculations are inches for sections of boards, joists, posts, etc., and feet for such dimensions as spans of boards and joists, heights of posts, etc. The unit of weight is pounds throughout.

Deflection is the principal factor in the design of boarding for sheeting the soffits of slabs or the faces of walls, and the tabulated spacing of the supports limits the deflection to $\frac{1}{16}$ in. Wrought boarding is assumed, but if thicknesses other than the tabulated net values are used the maximum spacing of the supporting joists can be calculated from the formula given if the boards are continuous over three or more spans. If $\frac{1}{8}$-in. deflection is permissible the tabulated or calculated spacings can be increased by 20 per cent. Where one length of board only spans over two spans or a single span, the spacing of the supports should be reduced to seven-eighths of the spacing determined from the table. The load on the boards may be either a vertical load as in the case of floor or roof slabs or a horizontal pressure in the case of wall or column sheeting.

The section of joists supporting boarding will be decided principally by the bending strength of the member, and *Table* XI gives formulæ and constants for the ready calculation of suitable sizes, or alternatively of suitable spacings. The tabulated factors and expressions assume the joist spans over a single span or two spans. Where joists are continuous over three or more spans, the spacing of the joists can be increased by 25 per cent. if the deflection of the boarding carried by the joists is satisfactory. The deflection of the joist and the shear stress should be checked when a section suitable to resist bending has been selected. If the end of the joist is reduced in section to fit on to bearers, the reduced section should be used for checking the shear stress. The width of bearers supporting the ends of joists usually depends upon the crushing strength of the timber. The reaction on the bearer should be calculated and sufficient area provided to limit the bearing stress to 400 lb. per square inch. The maximum shear force and bending moment on the bearers should be calculated and the proportions of the bearer determined in accordance with the tabulated expressions.

The direct load on posts determines the section of such members, safe stresses and limiting conditions being given on *Table* XI.

Examples of Design.

The following examples illustrate the essential considerations in preparing calculations for shuttering and show the application of the data given on *Table* XI.

Floor Panel.—Determine the sizes for a panel spanning 10 ft., the concrete slab being 5 in. thick and the deflection to be limited to $\frac{1}{16}$ in. The arrangement of the boards and supporting joists would be as shown in *Fig.* 108.

SLAB SHEETING.—Load: Concrete 60 lb. per square foot
Superimposed: 75 ,, ,, ,, ,,

Total (w) = 135 ,, ,, ,, ,,

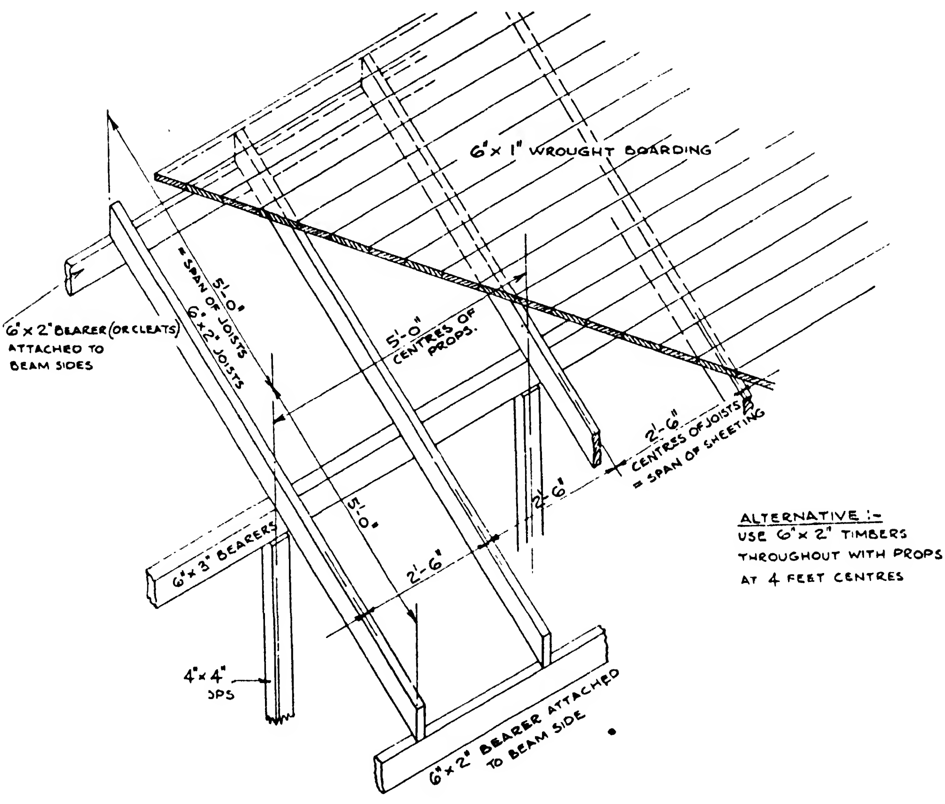


FIG. 108.—EXAMPLE OF FLOOR SHUTTERING DESIGN.

Using 1-in. boards (thicknessed to $\frac{7}{8}$ in.), from *Table* XI the maximum spacing of the joists is 2 ft. 4 in. clear. Alternatively, using $1\frac{1}{4}$-in. boards (thicknessed to $1\frac{1}{8}$ in.), the maximum spacing of the joists by calculation is

$$D = 8{\cdot}6\sqrt[4]{\frac{1{\cdot}125^3}{135}} = 2 \text{ ft. } 9 \text{ in. clear.}$$

JOISTS.—Assuming 1-in. boards and 10 ft. span of joist assumed to be 8 in. deep and 3 in. wide, the minimum breadth of joist is

$$b = \frac{(2 \text{ ft. } 4 \text{ in.} + 3 \text{ in.})\ 135}{85} = 4{\cdot}1 \text{ in.}$$

Thus an 8-in. by 4-in. joist would be required.

If this is considered too large, the alternative of introducing an intermediate bearer may be considered, reducing the span of the joist to 5 ft.; with an assumed depth of 6 in., 1-in. boards, and $D_1 = 2$ ft. 6 in.,

$$b = \frac{2 \cdot 5 \times 135}{192} = 1 \cdot 76 \text{ in.}$$

Thus 6-in. by 2-in. unwrought joists spaced at 2-ft. 6-in. centres and carrying 1-in. wrought boarding would be satisfactory.

Checking the deflection of the joist,

$$\frac{1 \cdot 0 \times 135 \times 2 \cdot 5 \times 5^4}{10{,}000 \times 2 \times 6^3} = 0 \cdot 05 \text{ in.}$$

which is within the allowable limit of $\frac{1}{16}$ in.

Checking the shear stress on the joist,

$$\frac{3 \times 135 \times 2 \cdot 5 \times 5}{4 \times 2 \times 6} = 105 \text{ lb. per square in.}$$

which is within the permissible stress of 200 lb. per square inch.

BEARERS.—The reaction on the intermediate bearer is $135 \times 2 \cdot 5 \times 5 = 1{,}690$ lb. Assuming the props under the bearer are placed at 5-ft. centres, forming the bearer into a beam continuous over more than two spans and carrying a point load of 1,690 lb. in the centre of each span and immediately over each support, the maximum bending moment is, say,

$$M = 0 \cdot 8 \times \frac{1{,}690 \times 5 \times 12}{4} = 20{,}150 \text{ in.-lb.}$$

Therefore $\frac{M}{200} = 101$ = minimum value of th^2.

The value of th^2 for a 6-in. $\times$ 3-in. timber is $3 \times 6^2 = 108$, which size will therefore be suitable.

Checking the minimum width of the bearer for crushing at support of joist,

$$R = 1{,}690 \text{ lb.}; \ t = \frac{1{,}690}{400 \times 2} = 2 \cdot 11 \text{ in.}$$

A width of 3 in. is provided.

The maximum shear force $F = \frac{1{,}690}{2} = 845$ lb.;

Checking the minimum depth of the bearer for shear,

$$h = \frac{1 \cdot 5 \times 845}{200 \times 3} = 2 \cdot 11 \text{ in.}$$

A depth of 6 in. is provided.

PROPS.—Assuming 4-in. by 4-in. props, $gh_1 = 4 \times 4 = 16$ square inches. The load on each prop is the reaction from two joists $= 2 \times 1{,}690 = 3{,}380$ lb. If the clear height of the prop is 12 ft., $\frac{H}{g} = \frac{12}{4} = 3$, for which ratio we can take the compressive stress as $c = 550$ lb. per square inch (*Table* XI).

Checking the compression in the prop, the minimum area required is

$$\frac{P}{c} = \frac{3{,}380}{550} = 6{\cdot}15 \text{ in.}^2;$$

the area provided is therefore ample.

Checking the crushing of the supported timber, the minimum area required is given by

$$\frac{P}{400} = \frac{3{,}380}{400} = 8{\cdot}45 \text{ in.}^2,$$

against 16 square inches provided.

The foregoing calculations give the arrangement shown in *Fig.* 108. In addition to the boarding this arrangement gives three different sizes of timber;

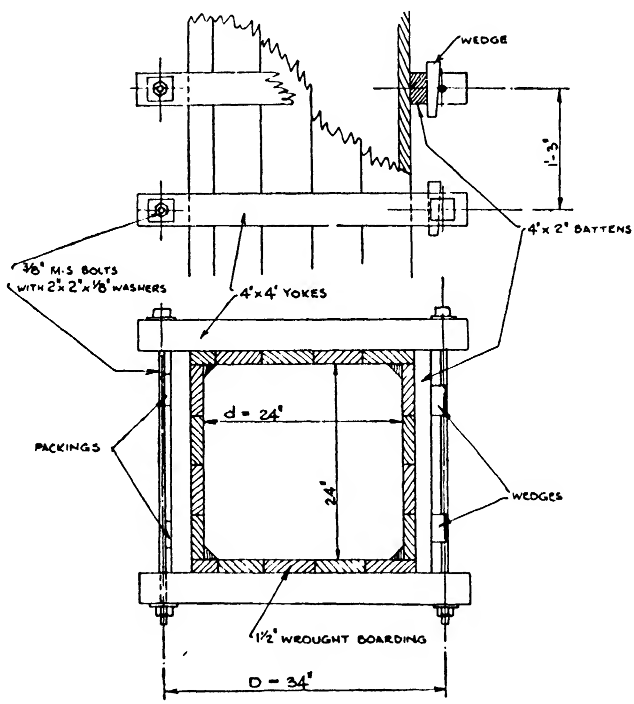


FIG. 109.—EXAMPLE OF COLUMN BOX DESIGN.

if such varying sizes are not available it would be practicable to use 6-in. by 2-in. sections throughout for joists, bearers, and props if the latter were spaced at 4-ft. centres.

Columns.—Find the spacing of the 4-in. by 4-in. yokes required for the 24-in. square column shown in *Fig.* 109, the rate of continuous filling being 4 ft. per hour, assuming 1 : 2 : 4 concrete, and a mean temperature of 50 deg. F. From the formula given previously the estimated maximum pressure from the wet concrete would be $P = (50 \times 4^2) + 70 = 870$ lb. per square foot. It can be considered that this pressure is constant over a height comparable with the spacing of the yokes. The distance between the bolt centres in any one principal yoke is approximately 34 in.; the maximum bending moment in the yoke for every foot of spacing is given by

$$M = \frac{Pd}{48}\left(D - \frac{d}{2}\right) \text{ in.-lb.}$$

The dimensions D and d are in inches and denote the lengths given in *Fig.* 109.

Therefore $$M = \frac{870 \times 24}{48}\left(34 - \frac{24}{2}\right) = 9{,}600 \text{ in.-lb.}$$

The moment of resistance of the yoke is equal to the modulus of the section multiplied by the working stress, which is

$$\frac{4 \times 4^2}{6} \times 1{,}200 = 12{,}800 \text{ in.-lb.}$$

$$\text{The spacing of the yokes} = \frac{\text{moment of resistance of a single yoke}}{\text{bending moment per foot of spacing}}$$

$$= \frac{12{,}800}{9{,}600} = 1{\cdot}23 \text{ ft., say, 1 ft. 3 in. centre to centre.}$$

$$\text{The calculated deflection of the principal yokes} = \frac{\text{spacing} \times PdD^3}{720EI} \text{ (approx.)}$$

$$= \frac{1{\cdot}25 \times 870 \times 24 \times 34^3}{720 \times 1{,}200{,}000 \times \frac{4 \times 4^3}{12}} = 0{\cdot}0512 \text{ in. which is less than } \tfrac{1}{16} \text{ in., and is}$$

permissible.

If $1\frac{1}{2}$-in. wrought boarding is used the maximum spacing of the yokes to limit the deflection of the boards will be calculated from the formula on *Table* XI.

$$\text{Spacing} \not> 8{\cdot}6 \sqrt[4]{\frac{1{\cdot}375^3}{870}} = 2 \text{ ft.}$$

The deflection of the boards will not therefore control the spacing of the principal yokes, which can be 1 ft. 3 in. throughout the height of the box except over the top 7 ft. $\left\{\frac{870}{120} \text{ approximately}\right\}$ where the spacing can be increased in increments of 3 in. up to 2 ft.

The maximum load on each bolt will be

$$T = \frac{Pd \times \text{spacing (ft.)}}{24} = \frac{870 \times 24 \times 1{\cdot}25}{24} = 1{,}090 \text{ lb.}$$

Adopting a working stress of 16,000 lb. per square inch, which will allow a margin for additional bending stresses due to wedging off the secondary yokes, the area of bolt required $= \dfrac{1{,}090}{16{,}000} = 0{\cdot}068$ sq. in. This would be given by the area at the bottom of the threads of a $\frac{3}{8}$-in. diameter bolt. The area of the washer required is $\frac{1{,}090 \text{ lb.}}{400 \text{ lb. per square inch}} = 2{\cdot}73$ sq. in. This is amply provided for by a 2-in. square washer allowing for a $\frac{7}{16}$-in. bolt hole; the washer should be $\frac{1}{8}$ in. thick. The secondary yokes can be 4 in. by 2 in. fixed with the 2-in. dimension outstanding, as the bending moments and deflections can be limited by the position of the wedges shown in *Fig.* 109.

If the concrete was placed in the column box in 3-ft. lifts with sufficient intervals between each lift to enable the concrete to set, the maximum pressure to which the shutters would be subjected would be about 420 lb. per square foot (3 ft. $\times$ 140 = 420, see *Table* XI). In this case the yoke spacing could be increased up to, say, 2 ft. 6 in. without causing undue deflection in the boards or overstressing in the yokes or bolts.

Cost.

The following particulars and quantities may be used in estimating the cost of timber shuttering.

One square yard of shuttering requires $1\frac{1}{2}$ to 3 cu. ft. of timber, depending on the amount of propping and the class of work but without allowing for repeated use. Foundations and floor slabs require about $1\frac{1}{2}$ cu. ft. per square yard; columns from $1\frac{3}{4}$ to $2\frac{3}{4}$ cu. ft.; walls and hollow floors, about 2 cu. ft.; beams and flat slabs, about 3 cu. ft.; stairs and architectural features, up to $4\frac{1}{2}$ cu. ft.

The price of boarding is usually quoted per square and the price of other timbers per standard. A square = 100 ft. super. A Petrograd standard = 165 cu. ft. = 1,980 square feet of 1-in. boarding. An allowance of 5 to 10 per cent. should be made for sawing and cutting to waste.

Common terms applicable to timber are: Battens, 7 in. wide by 2 in. to 3 in. thick; deals, 9 in. wide by 2 in. to 3 in. thick (120 deals = one hundred); planks, 11 in. wide by 2 in. to 3 in. thick.

Shuttering for octagonal columns costs approximately one-third more per square foot than for square columns. Shuttering for curved (cylindrical) surfaces costs about one-third more than for plane surfaces. Shuttering for spherical (or domed) surfaces costs about twice as much as for plane surfaces.

The average value of timber after being made up into shutters is from one-quarter to one-third of the original value after a single use. Carpenters working on old shuttering require about a penny an hour more than on new material.

Well-designed and stoutly-constructed column, beam, and wall shutters can generally be used four times without requiring excessive repairing, and six to eight times before the cost of repairing equals the cost of remaking. Beam bottom boards and props can, however, be used many more times than this without replacement. As a rule fillet pieces for forming chamfers can only be used once as they are extensively damaged when ripped off the boards.

It is impracticable to give the relation between the cost of material and labour for shuttering, as so much depends upon the amount of re-uses that can be obtained from a given consignment of timber and to what extent the wood is cut. The cost of nails, bolts, wire ties, or other connections may amount to 5 to 8 per cent. of the cost of the timber.

The extent to which economies can be made by the repeated use of timber shuttering is illustrated by the following example in which the costs are reduced to 1 sq. yd. of concrete surface covered. Six uses of timber can seldom be effected.

Material :—

	s.	d.	
1¼-in. wrought boarding at 30s. per square	2	9	per square yard
Props, cleats, joists, etc., say 1 cu. ft. at £20 per standard	2	5	,, ,, ,,
Add 10 per cent. for cutting and waste.	0	6	,, ,, ,,
Bolts, nails, etc., say	0	4	,, ,, ,,
Total	6	0	,, ,, ,,

	Single use		Two uses		Four uses assuming no repairs		Six uses with minimum repair	
	s.	d.	s.	d.	s.	d.	s.	d.
Labour making	1	6		9		4½		3
,, erecting		9		6		6		6
,, striking		6		6		6		6
,, cleaning, oiling, and repairing for re-use (if necessary)	nil			4		4		6
Total labour	2	9	2	1	1	8½	1	9
Material	6	0	3	0	1	6	1	0
33⅓ per cent. on labour for plant, small tools, insurance, mould oil, overhead expenses, and profit		11		8		6		7
15 per cent. on material for overhead expenses and profit		11		5		3		2
Gross cost per use	10	7	6	2	4	0½	3	6
Less salvage value of material, say 20 per cent. of first cost	1	2		7		3½	nil	
Net cost per square yard per use . . .	9	5	5	5	4	4	3	6

As a comparison, consider the cost of using steel shuttering in place of timber. The cost and quantities would depend on the type of equipment and whether it was hired, purchased, specially manufactured, or already in the contractor's stock.

The labour cost of the complete sequence of operations involving oiling, erecting, removal, cleaning and stacking ready for re-use can usually be taken at half the cost of the corresponding labour on timber. Whereas the net labour on timber shuttering may be 3*d*. to 5*d*. per square foot, the net labour on steel shuttering may be from 1½*d*. to 2*d*. per square foot for straight work and up to 3*d*. for curved work. The method of estimating adopted in the following example is applicable to most cases if the appropriate data are substituted for those assumed. Consider that the steel plates, stiffened with edge angles, complete with bolts, clamps or other connections, and including for vertical and horizontal aligning angles, weigh 14 lb. per square foot of concrete surface shuttered and can be obtained new for £24 per ton. The cost per square yard will be

$\frac{£24 \times 20}{2,240} \times 14 \times 9 = 27s.$ per square yard, or say 31*s.* per square yard including overhead costs and profit. For various numbers of uses the cost per square yard would be

	Single use		Six uses		20 uses		40 uses	
	s.	*d.*	*s.*	*d.*	*s.*	*d.*	*s.*	*d.*
Steel shutters	31	0	5	2	1	7		9
Labour erecting, dismantling, cleaning, oiling and preparing for re-use	1	0	1	6	1	6	1	6
$33\frac{1}{3}$ per cent. on cost of labour as above . .		4		6		6		6
Total	32	4	7	2	3	7	2	9

These rates require further additions to allow for the cost of returning the shutters to the contractor's yard, repairs and replacements before re-using on other contracts, and for deductions due to availability of the shutters for a considerable number of uses on future work. Any comparison that can be made with the data available for given conditions usually indicates that with upwards of 20 uses steel shutters for plain walls and floors may be cheaper than timber. For curved work, although the first cost of steel shutters may be much higher than for plane work, the advantage may be more marked. Even for small numbers of uses the salvage or re-sale value is an item worth taking into account.

SECTION II.—DETAILS OF SHUTTERING.

Fixings for Shuttering.

The methods of fixing various pieces of shuttering together include the use of nails, screws, bolts, wire ties, and clamps. Bolts and wire ties are the most common, the former usually being preferable except in the case of impermeable construction. Wire ties must be drawn taut without exhibiting spring and should not be made of brittle material. For this reason galvanised iron wire or steel wire are not favoured and black annealed iron wire, No. 9 to No. 16 gauge, is recommended. The ties are left in the concrete, the projecting ends being clipped off after removing the shutters. If the ends are not covered by subsequent finishing materials, objectionable rust stains may result. Methods of using wire ties in wall forms are shown in *Fig.* 110.

Wire ties to take tensions in excess of those usually associated with wall construction can be made by twisting double binding wire together with a loop at each end as in *Fig.* 111. This gives four strands in the tie portion and two strands forming each loop.

A typical connection made with bolts is shown in *Fig.* 112. Mild steel bolts should be $\frac{3}{8}$ in. to $\frac{3}{4}$ in. in diameter, the smaller sizes being usually sufficient for bolts in direct tension as in column and wall shutters while the larger sizes are used for bolts subject principally to transverse loading. The threads should be well greased and any adhering concrete spillings cleaned from them as often as practicable. Washers, 2 or 3 in. square, should always be provided, and the diameter of holes through the timber should not be more than $\frac{1}{16}$ in. greater than the diameter of the bolt. Holes should not be formed less than 6 in. from the end of the timber if the latter is subject to transverse loading.

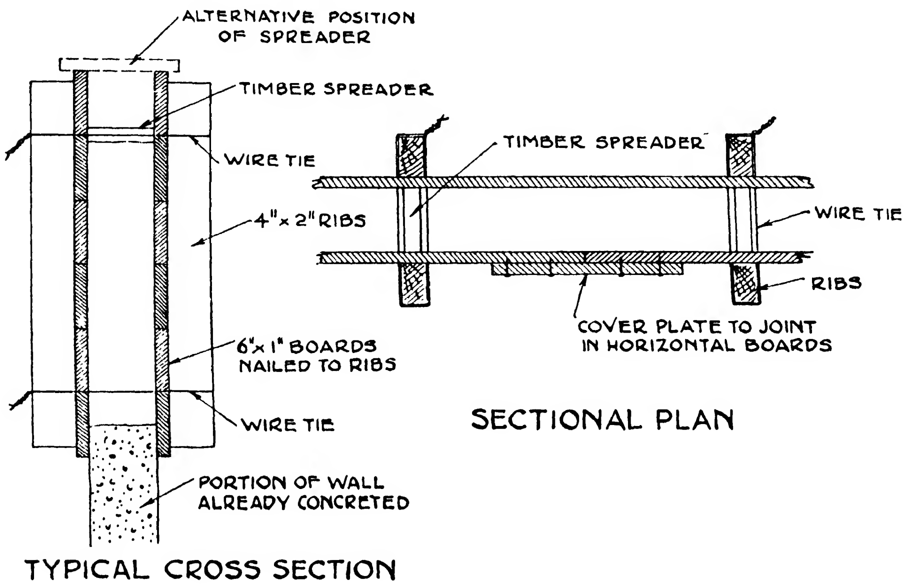


FIG. 110.—WALL SHUTTERING WITH WIRE TIES.

In calculating and specifying the quantities of bolts each size and length should be given separately and the quantity expressed in pounds, the weight including nuts and washers complete. An allowance of 1 lb. for each three or four pairs of washers should be sufficient. In order that they may be used in any type of work, bolts should be ordered in generous overall lengths with

FIG. 111.—WIRE TIE.

ample threaded lengths. Excess lengths can be readily taken up by packing as shown in *Fig.* 112.

Removal of bolts passing through set concrete may be made easier by well greasing the bolt or by giving the bolt half a turn while the concrete is only partly set. Cardboard tubes passing through the walls are a simple means of preventing the concrete adhering to the fixing bolts. As the bolts are passed through the tubes they do not come in contact with the concrete, and it is therefore unnecessary to grease them or take other precautions to ensure easy removal.

For impermeable construction it may be necessary to remove the tubes and this may be successfully done by brace and bit.

When bolts are used in watertight work the stopping of the holes after withdrawal of the bolts must be done in such a way as to render the holes impermeable, such as by plugging the holes with cement mortar after well flushing them out with water. If the wall is rendered on the inner face the risk of leakage through badly-stopped bolt holes is minimised.

For easy removal, bolted and wedged connections are preferable to nailed joints, but the latter can be easily dismantled if double-headed nails are used or, less expensively, if the nails are not fully driven home, the heads being left sufficiently proud to enable a prising tool to be used. Common wire nails 2½ in. and 4 in. long are suitable, the quantity required varying considerably with the type of shuttering and the amount of making-up required. An allowance of 1 lb. per square yard is an outside figure.

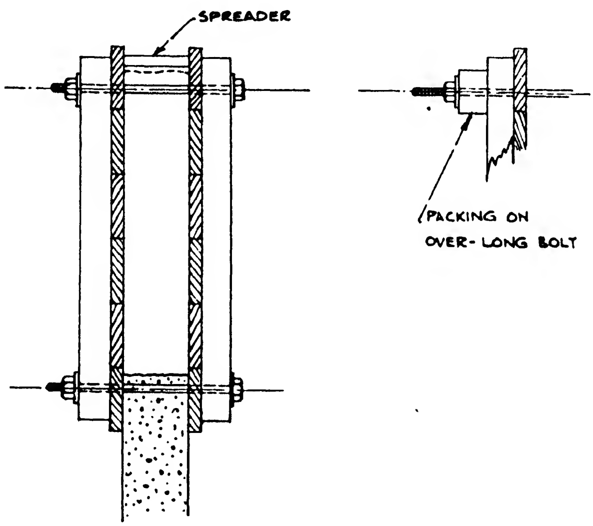


FIG. 112.—BOLTED CONNECTIONS.

As regards security, ability to be removed without jarring, and repeated use, screws no doubt offer advantages compared with nails, but it is doubtful in normal work whether these advantages offset the extra cost of providing screws or the extra labour involved. Screwed connections may sometimes be preferable for important work where there is no need to dismantle the connection each time the shuttering is struck. When external vibrators are attached to the shuttering, prolonged vibration may cause nails to draw, and the boarding should be attached to the battens by countersunk screws.

Metal clamps, similar to those illustrated in *Fig.* 113 are generally of proprietary makes and are mainly used for column shutters. If numerous uses over a number of jobs can be assured they are economical as they are readily applied, adjustable, and easily removed without jarring. Some of the more expensive types of shutter clamps and centering supports may be hired if the size of the job does not warrant purchasing outright.

FIG. 113.—STEEL CLAMPS FOR COLUMN BOXES.

Chamfers and Fillets.

Most engineers specify the provision of chamfers at external corners of beams and columns and sometimes require fillets at internal corners. The formation of these angle features is often looked upon by the contractor as an unnecessary expense, but on columns especially, the small additional net cost—say about one penny per lineal foot—of providing fillet strips in the moulds to produce chamfers is often more than offset by the saving in making good damage to corners. Chamfers on external corners are usually 1 in. by 1 in. and are readily formed by inserting a fillet strip in the forms. These strips can be purchased ready cut, or if a power-driven saw is available on the site they can be made from 1-in. boarding by cutting along a series of diagonal lines as indicated in *Fig.* 114.

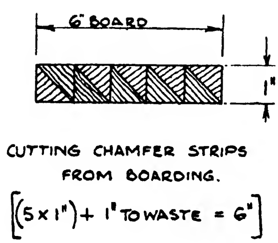


FIG. 114.—CUTTING CHAMFER STRIPS.

From a board 6 in. wide, ten lengths of 1-in. fillet strip can be cut, the remaining inch being cut to waste.

Fillets or splays on internal corners are less readily made. If not larger than about 2 in. by 2 in. they can be made by cutting or planing the corner

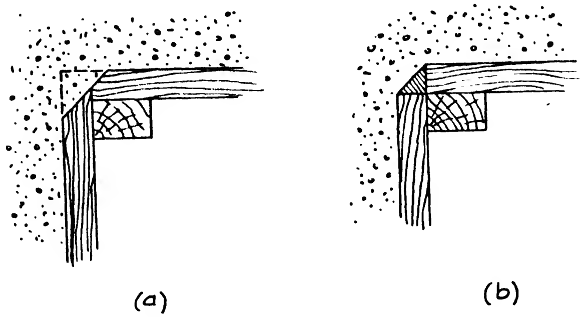


FIG. 115.—FORMATION OF FILLETS.

formed by adjoining boards, as shown in *Fig.* 115(*a*), or by introducing a length of fillet strip as in *Fig.* 115(*b*) shaping the ends of boards is avoided. For the construction of large fillets in tank and bunker work an additional board is intro-

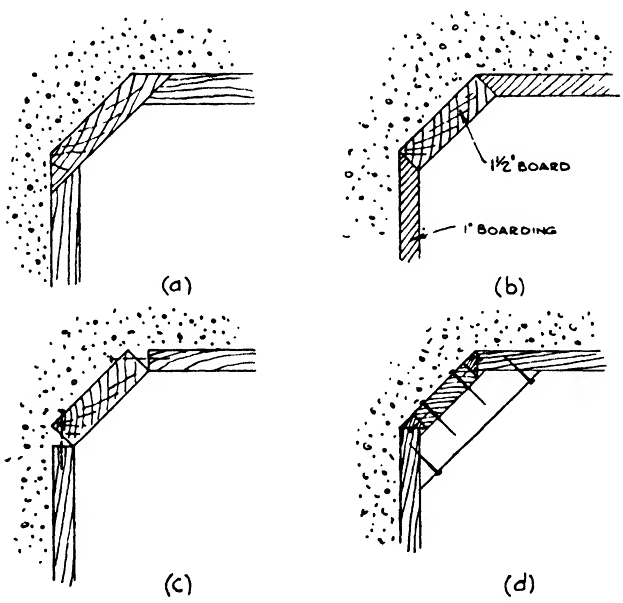


FIG. 116.—FORMATION OF VERTICAL SPLAYS.

duced, one or other of the methods illustrated in *Fig.* 116 being adaptable for vertical splays. That shown at (*a*) is probably the most expensive as it requires careful shaping of the edges of all three boards. In method (*b*) the face of the splay is formed by an unshaped board with the ends of the wall sheeting only splayed. If this splaying is neglected two triangular concrete fins will be formed as shown at (*c*) and these will have to be cut away if a fair surface is to be left. By filling the spaces by fillet strips attached along the edges of the splay board, as at (*d*), the labour of making-good is avoided. It is an obvious advantage to the contractor if the size of the splay is such that the splay-board in methods (*b*), (*c*) or (*d*) can be of standard width without being cut.

Horizontal splays and fillets between the walls and bottoms of tanks introduce complications in the shuttering. To be of any structural value the concrete fillet must be monolithic with the wall, the addition of the fillet subsequently to the construction of the bottom and walls being prohibited. This prohibition would not apply in the case of designs where the fillet is formed in the rendering only and not in the structural concrete. If the construction joint between the floor

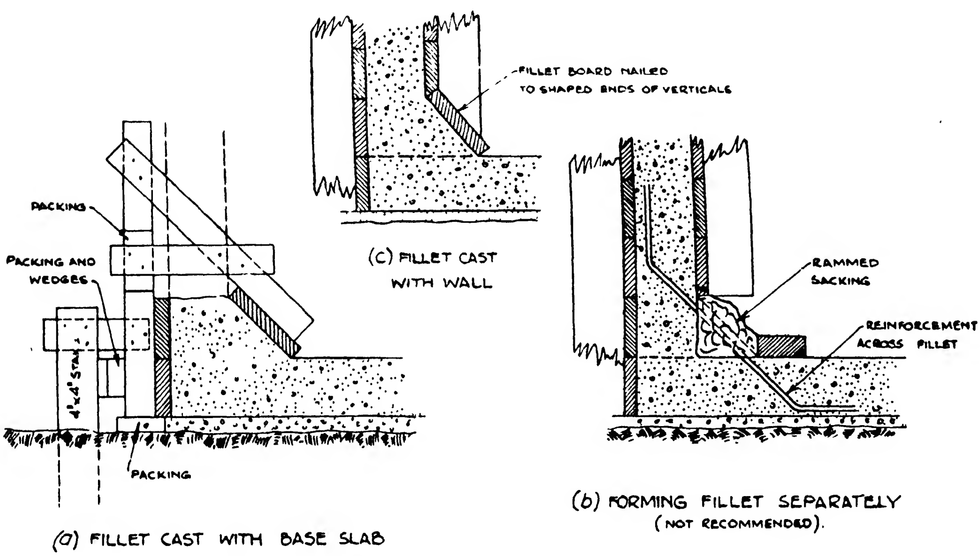


Fig. 117.—Shuttering for Horizontal Splays.

and wall is specified to be at the base of the fillet, the shuttering for the latter is included with that for the first lift of wall sheeting. In *Fig.* 117(*c*) is shown a simple method of attaching the fillet board to the shaped lower ends of the vertical battens, thus enabling the fillet to be cast monolithic with the stem of the wall. If the construction joint is at the top of the fillet the arrangement indicated in *Fig.* 117(*a*) allows the fillet and corresponding section of the wall to be concreted at the same time as the bottom slab.

If reinforcement is provided in the fillet the latter must be constructed either with the wall or with the bottom slab. The method illustrated in *Fig.* 117(*b*) is sometimes adopted to avoid forming special timbering for the fillet, although it cannot be claimed to be good practice; the lowest board of the wall shutters is omitted, sacks being stuffed in the spaces between the bars to prevent the escape of the concrete, and when the wall shutters and sacking are removed the fillet is made up in cement mortar.

Inclined Work.

The upper face of inclined concrete work requires shuttering if the slope is greater than 20 to 25 deg., that is, exceeds a slope of 1 in $2\frac{1}{2}$. If the concrete is very plastic, shuttering may be required on both sides at smaller angles. On the other hand, if a "dry" concrete is used it may be possible to place it at an angle of 45 deg. without shuttering the upper face. For thin sloping slabs the

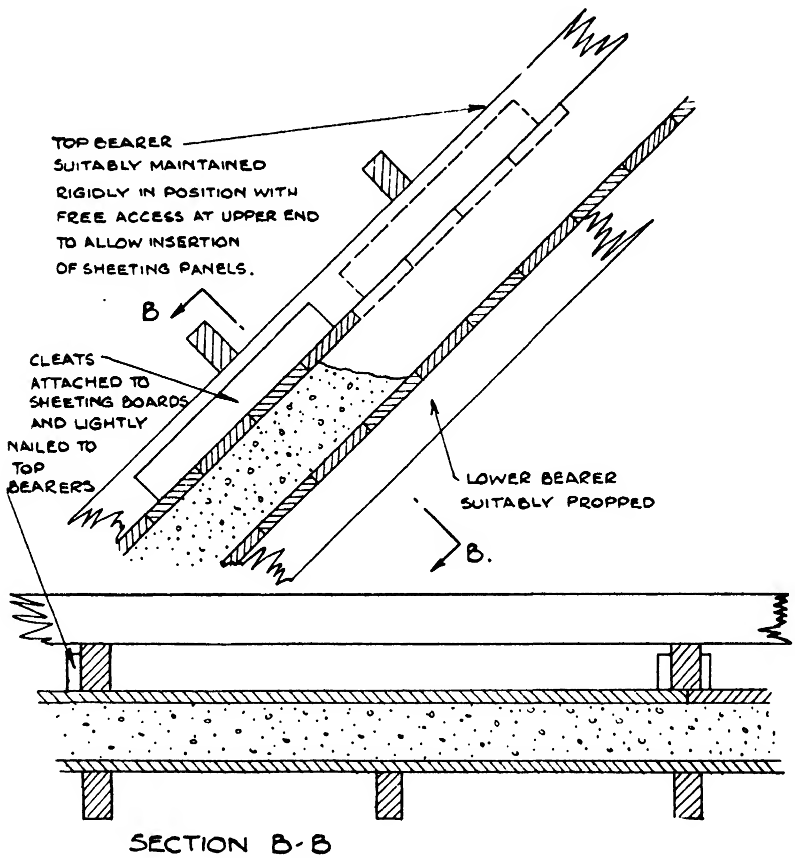


Fig. 118.—Inclined Slabs.

whole of the underside sheeting is first erected and the reinforcement fixed. The longitudinal bearers for supporting the top sheeting are then placed accurately in position and the boards inserted one or two at a time as concreting proceeds as indicated in *Fig.* 118. It is necessary only to attach the boards to the bearers sufficiently to prevent the latter dropping out of position, and this may be readily done by providing one or more cleats (as indicated) secured to the boards and lightly nailed to the bearers. When the concrete is in position the upward pressure normal to the top boards will prevent displacement.

A similar method can be adopted for inclined beams as in *Fig.* 119, in which case the top boards, to resist the upward pressure of the contained concrete are simply nailed to the top edges of the side boards.

Column footings of the splayed-top type require shuttering on the top faces, and this must be weighted to prevent it "floating." The counterweight may

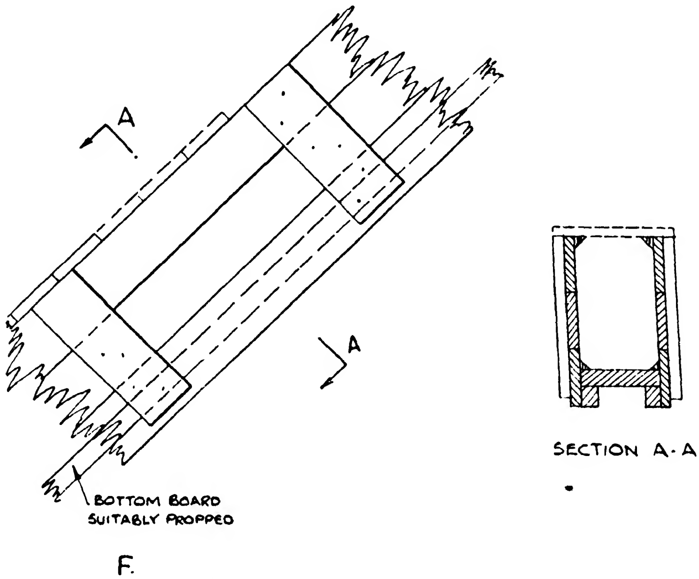


FIG. 119.—INCLINED BEAMS.

be conveniently provided by bags of cement, bricks, or similar available material. The weight required can be approximately calculated in accordance with the following method. Consider a footing 5 ft. square as indicated in *Fig.* 120. The counterweight required must balance the total upward pressure when the mould is filled with wet concrete. The upward fluid pressure from the concrete would not exceed, say, 140 lb. per square foot per foot of head, and since the maximum head operating on the inclined form is 1 ft. the pressure distribution on each face of the mould is that indicated in the diagram. The total pressure on one face is equal to the volume of the pressure diagram, that is

$$\frac{1{\cdot}5 \times 1{\cdot}75 \times 140}{2} + \frac{2 \times 1{\cdot}75 \times 1{\cdot}75 \times 140}{3} = 184 + 286 = 470 \text{ lb.}$$

The total pressure on four faces is therefore $4 \times 470 = 1{,}880$ lb.

A general formula applicable to square bases can be expressed thus: Counterweight required $= 93t(D - d)\left(\frac{d}{2} + D\right)$ (lb.). The dimensions t, D and d (in feet) are indicated in *Fig.* 120. The weight of the top shutters and sides to which they are attached would be approximately $3\frac{1}{2}$ sq. yd. at 75 lb. per square yard, or 263 lb. The additional weight required would therefore be $1{,}880 - 263 = 1{,}617$ lb. This could be provided by eight 2-cwt. sacks of cement, or cross timbers, with their ends attached to stakes driven into the ground, could be fixed to the inclined shutters. An approximate method of computing the number and depth of the stakes is to assume an average frictional resistance to withdrawal of not more than $\frac{1}{2}$ cwt. per square foot of surface in

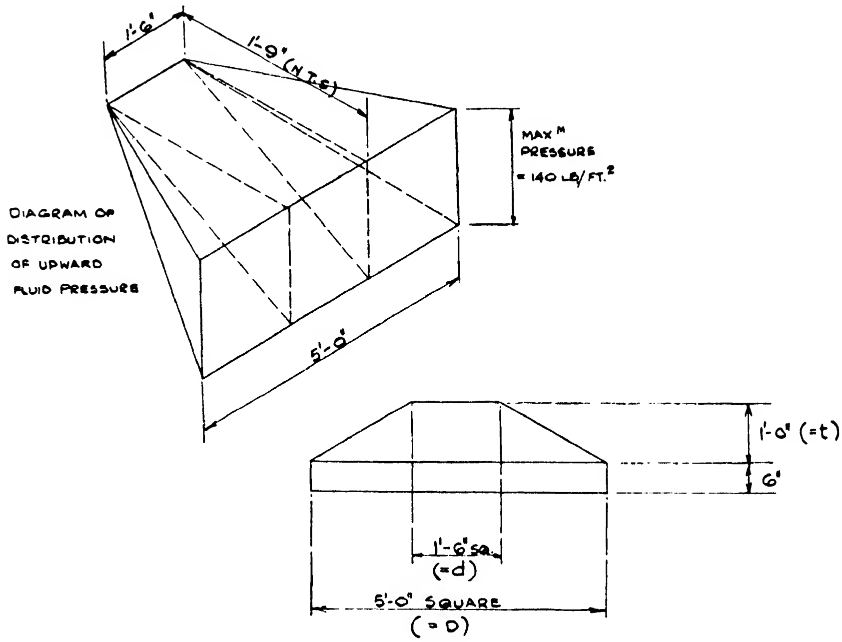


FIG. 120.—PRESSURES ON COLUMN FOOTINGS.

contact with the earth. If the ground is dry the hold on the stakes will be negligible unless they are driven deeply. To provide a resistance of 1,617 lb. the total area of embedded stake is $\frac{1,617}{56} = 29$ sq. ft. Assuming 4-in by 4-in. stakes, two to each side of the footing (that is, eight in all), the minimum depth of penetration of each stake should be $\frac{29}{4 \times \frac{1}{3} \times 8}$ = say, 3 ft. The need for much of these anti-floating precautions can be avoided if a dry mix is used.

Resistance to the outward and upward forces of concrete contained in the shuttering for doubly-sheeted inclined slabs must generally be provided by securing the ends of the principal bearers. In the case of hopper bottoms, any interior face can be strutted horizontally from that opposite and the weight of the inverted pyramid of timber can be augmented if necessary to prevent "floating."

Coatings for Shutters.

To prevent the concrete sticking to the shuttering and thus simplify its removal and minimise cleaning preparatory to re-assembly, the faces of the shutters in contact with the concrete should be painted with a suitable coating of oil or other material. A good mould coating should not stain the concrete or injure it by chemical action and should not be absorbed by timber. It should be slow drying and should not become flaky, nor be removed by rain or by the water used for cleaning the shutters. It should not be adversely affected by the heat of the sun or by low temperature. It must be insoluble in water and be able to withstand several fillings of concrete, while for economy the material should have a large covering capacity. When large areas of shuttering are to be dealt with it is advisable, before obtaining large quantities of a given oil, to try one or two samples in some positions such as basement walls where failure to give the best results is of no consequence.

Although proprietary mould oils are most frequently used, other materials are suitable. Crude linseed oil is successful for both timber and steel shuttering. Creosote, although it is otherwise satisfactory, discolours the face of the concrete work. Inert whitewash is also good, but it leaves a flaky adherent on the concrete that must be removed if a good appearance is required. A suitable material can be made by cutting a quantity of yellow soap into shreds, mixing with water, boiling, and stirring until it attains the consistency of paint. Another suitable coating can be prepared by mixing one part of raw linseed oil in three parts of paraffin oil, although such a mixture may leave a stain on the concrete surface.

The quantity of mould oil required depends upon the nature of the timber surface and the characteristics of the brand being used. An allowance of one gallon to 20 sq. yd. of shuttering is a reasonable quantity for estimating purposes, and a reasonable cost to include material and the labour of applying it is 3*d.* per square yard.

If cement retarding liquids are used the use of mould oil is unnecessary, and if waterproofed paper linings are provided additional coatings are unnecessary to prevent the concrete adhering

Shuttering coatings are applied by a brush before the reinforcement is fixed so as to avoid the mould oil dropping on the bars.

SECTION III.—STRIKING SHUTTERING.

Normal Times of Striking.

Most specifications place the responsibility of the removal time of shutters on the contractor. It is necessary to guard against premature removal, at the same time economising in timber by releasing the shutters as soon as the concrete has attained sufficient strength to be self-supporting or requires a minimum of external support shutters.

Generally each case has to be considered on its merits, allowing for the hardening characteristics of the cement, the temperature since depositing the concrete, the loads to which the member will be subjected, and the ability of the supporting members to take the weight of the part from which the shutters are to be struck. For normal temperatures (not below 45 deg. F.) the following times are commonly accepted as safe, but the precautionary notes that follow should not be overlooked.

SLABS.—For spans not exceeding 10 ft., soffit boards can be removed (if occasional props are provided) in seven days using normal-hardening Portland cement or in three to seven days if rapid-hardening Portland cement is used. For longer spans the shutters (or at least substantial propping) should be left in place for two to three weeks. The number of props required is one for about each 10 ft. of span. With rapid-hardening Portland cement the props might be removed in five to fourteen days. For spans in excess of 20 ft. additional time as for long span beams should be allowed.

BEAMS.—It is often convenient to strike beam sides at the same time as the slab sheeting, but if practicable the former may be removed at three days (two days by using rapid-hardening Portland cement). The bottom boards should be left in position at least as long as the slab sheeting, and it is important that props should be retained in position for at least three weeks (not less than eight days with rapid-hardening Portland cement). Beyond this period a single prop should be maintained under each span for, say, two weeks (or a further eight days with rapid-hardening Portland cement). For long spans these times should be increased by half a day for each foot of span in excess of 20 feet.

COLUMNS.—The sides of columns may be removed at the same time as the beam bottoms since the latter are supported on the column boxes. The minimum time for removal of boards from column faces is three days (two days with rapid-hardening Portland cement).

WALLS.—Wall shutters can usually be removed at the end of three days, but under favourable conditions 24 hours is justifiable. The early stripping of walls not subject immediately to horizontal or vertical loads other than their own weight is advocated, as satisfactory curing operations can only be undertaken when the concrete is exposed. Where moving shutters are used these times are often reduced to less than twelve hours without injury to the concrete.

MASS CONCRETE.—The shuttering to the faces of mass concrete walls or the sides of mass concrete foundation blocks can be removed in two days in the case of thick sections, but for thin vertical sections not less than three days should elapse.

The times given are minima and generally should be increased for such conditions as long-span construction as described above, heavy sections, frosty weather, or prevailing low temperatures. Before removing the bulk of the shuttering for any member a part of the encased concrete should be exposed in order to inspect the hardness attained. Members such as slabs and beams and load-supporting members such as columns, should be sufficiently hard to "ring" when struck with a hammer. The hardening of test cubes cast at the time the concrete was placed in the member will often give some indication of the hardness attained in the work if curing conditions are comparable. The use of rapid-hardening cement greatly increases the ability to release shuttering at an early date, but special care must be taken if temperatures below 50 deg. F. have occurred, when the striking time must be increased. Some specifications allow shutters to be struck at two-thirds the normal time if rapid-hardening Portland cement is used, but if temperatures around freezing-point have been experienced the times should be the same as for normal-hardening Portland cements. With high-alumina cements striking can usually proceed after 24 hours. Shuttering encasing vibrated concrete may be removed earlier than in the case of hand-tamped concrete of a similar mix, owing to the higher early strength associated with concrete that has been placed by an efficient vibration process.

Considerable increase in time may be necessary in the case of structures in which the dead load constitutes a large proportion of the total design load, and in such cases a minimum of 28 days should be imposed unless the engineer estimates that the concrete will have attained sufficient strength (allowing an adequate factor of safety) earlier than this age. If low temperatures have not occurred during the hardening period the following approximate times can be taken as minima for various ratios of dead to total load:

$$\text{Ratio } \frac{\text{dead load}}{\text{total load}} = 1 \qquad 28 \text{ days.}$$
$$\tfrac{1}{2} \qquad 21 \text{ ,,}$$
$$\tfrac{1}{3} \qquad 14 \text{ ,,}$$

If a structure is likely to be subjected during subsequent constructional operations to loads in excess of the design load, provision must be made to carry this extra load either by leaving the shuttering in place or by providing props that bear on members that are sufficiently strong to support the augmented load. The propping must be left in position until the risk of abnormal loading is removed.

Some forms of construction need special consideration. The precautions necessary when striking arch centering are discussed in Chapter IX. The shuttering and supports of suspended work such as hopper bottoms should not be removed until the beams and walls from which they are suspended are sufficiently mature to carry the load safely. The engineer will usually indicate when these operations can be carried out.

Effect of Temperature.

The variation in the setting and hardening times of concrete with change of temperature must be considered when assessing safe times for striking shuttering. With normal-hardening Portland cement, hardening is delayed by low temperatures and accelerated by high temperatures. The effect is less marked with aluminous cements and may even be reversed, but the effect of low temperatures is more serious with rapid-hardening Portland cement. It is difficult to lay down rules by which to determine suitable allowances for variations from normal temperatures, as experimental results show marked differences and are sometimes inconsistent.

The process of hardening is practically suspended when concrete is frozen and it appears that after thawing the rate of hardening is not so great as if the freezing period had not intervened. Alternate freezing and thawing are more harmful than continued freezing, and the longer freezing can be postponed the less the effect upon the rate of hardening. Wet mixes are more susceptible to harmful effects of frost than are plastic or dry mixes. It is common to specify that the time of striking shall be extended beyond the normal time by a period equal to the duration of the frost. The accuracy of this practice has, however, been seriously questioned, and it is recommended that the striking time be extended by a period equal to twice the time during which the concrete is subjected to temperatures of 32 deg. F. or below for plastic mixes and even longer for wet mixes.

All specified striking times when concrete has been frozen must depend on examination of the concrete indicating that the material has hardened. Steps should be taken (say by gently heating a small exposed face) to ensure that frozen concrete is not mistaken for hardened concrete. If, as described in the next chapter, provision has been made to prevent freezing, normal times may be adopted.

If high temperatures prevail during the hardening period and measures are taken to prevent rapid drying out, the striking time can be decreased to the following percentages of the normal time.

Mean prevailing temperature (deg. F.)	50	60	70	80
Percentage of normal striking time	100	65	55	50

A mean prevailing temperature of 40 deg. F. may require 50 per cent. increase in the time allowed for 50 deg. F.

Removal of Shuttering.

The procedure adopted in removing shuttering depends upon its construction. The conventional methods of erecting shuttering usually allow shuttering to slab soffits and beam sides to be removed first, then beam bottoms, and finally column boxes. The beam props (and, for large spans, the slab props) should be replaced for the full periods already given. If beam props are provided close to the columns the shutters can be struck from the latter before the beam and slab shutters as the beam bottoms are sufficiently supported without reliance on the cleats on the column boxes.

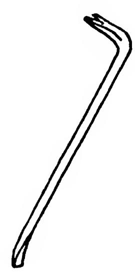


Fig. 121.—Nailbar.

All shuttering, and especially the principal supports, should be gradually eased in order to prevent the load being suddenly transferred to the concrete. For this reason wedged or similar supports are best, and bolted connections are preferable to nails. Properly constructed shutters should require little effort to remove them, although swelling of the timbers may cause portions of the boarding to jam. The only tool that should be necessary, in addition to a spanner, is a small crowbar or a nailbar similar to that shown in *Fig.* 121. This tool is easily forged out of a piece of ½-in. or ¾-in. diameter bar 1 ft. to 3 ft. long, and is useful either for withdrawing nails (the heads of which have been on account of the noise, and if an electrically-operated machine is not available the concrete is mixed by hand.

When only small quantities of concrete have to be mixed the extra care necessary to produce a good hand-mixed concrete can usually be exercised, but for large quantities, hand-mixed concrete should contain 10 per cent. more cement than the same class of concrete mixed by machine due to the inferior results of hand mixing compared with machine mixing.

The mixing should be carried out on a level, clean, and non-absorbent platform giving a clear space of at least 8 or 10 ft. The top of the stage is preferably covered with sheet metal or similar covering that will not flake or chip

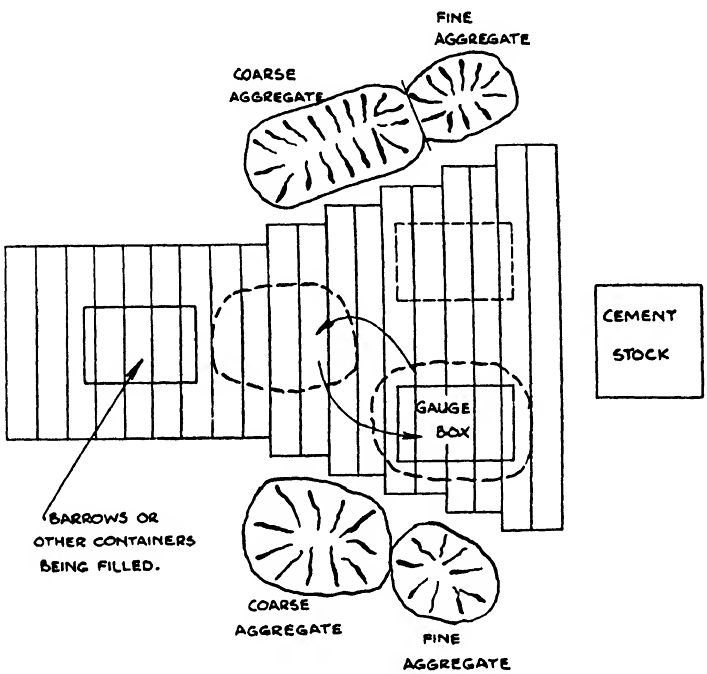


FIG. 127.—HAND-MIXING PLATFORM.

under the action of the shovels. If the concrete is mixed directly on the timber it is advisable to use tongued-and-grooved boarding to prevent leakage of mortar.

The aggregate and cement should be stored adjacent to the stage, and a suitable arrangement is shown in *Fig.* 127. The widened end allows the gauge-box to be filled ready for another batch while the preceding batch is being mixed, thus ensuring continuity of supply. For reinforced concrete work the cement and fine aggregate should be first mixed together in a dry state until of uniform colour and the coarse aggregate and water added subsequently.

Another method is to use a bottomless batch box fitted with handles and of a size which allows the coarse aggregate to be filled up to a predetermined height, and the fine aggregate to fill the remaining space. The box should be placed sufficiently far from the edge of the stage so that when it is lifted and the aggregate released the spreading pile does not run over the edge of the stage. After the box has been removed the cement is spread over the heap of aggregate.

The following order of placing the materials on the mixing platform is recommended : Coarse aggregate in bottom layer, fine aggregate spread uniformly in the second layer, and the cement, also spread uniformly, on top. In mixing, the material should be taken from the heap so that each shovelful contains some of each material. A suitable method of mixing is to shovel the materials over to an adjacent part of the stage and then back again to the original position, giving the shovel a turn so that the materials are spread out. The whole mass should be turned over two or three times while dry, and again two or three times while the water is being added until the mass is of uniform colour and consistency. The water should be applied through a rose fitted to a hose or to a watering can, the latter being preferable since it allows more precise measurement of the quantity of water. The latter should be added evenly and while the materials are being turned over. Care should be taken to pour the water on to the centre of the mass and thus prevent the water flowing off the stage and carrying cement with it.

Machine Mixing.

The essential feature of a concrete mixer is a revolving drum fitted with blades to stir the materials. The drum rotates about a horizontal or an inclined axis, the former being usual in ½-cu. yd. and larger capacity machines and the latter in smaller capacity mixers, typical examples of the latter being shown in *Figs.* 128 and 129. The power for revolving the drum and for lifting the materials skip and tilting the drum is provided by a petrol or other type of oil engine or by an electric motor. Belt-driven machines operated by steam engines are sometimes seen on big installations, and at the other extreme are hand-operated mixers of small capacity. For general work, however, a self-contained petrol-driven mixer is the most serviceable. The power required for a 1-cu. yd. mixer may be 15 to 20 b.h.p., while the power required for smaller machines varies from 20 b.h.p. for self-propelled ¾-cu. yd. machines to 3 b.h.p. for ¼-cu. yd. mixers. The power required will be affected if the engine or motor has to operate a hoist or other plant.

The blades may be either of the solid-plate type or open-lath type, but experience seems to show that whereas there is not much to choose between the two types for wet or plastic concretes, the solid-plate type is better for very dry mixes.

The method of charging the drum depends on the size of the machine. For small mixers measured amounts of the dry materials are fed separately directly into the drum. For medium-capacity mixers the materials are placed in a charging skip from which they are simultaneously tipped into the drum, the skip being elevated by power. Large-capacity mixers may have a fixed overhead hopper in which the materials are placed and released through a gate into the drum.

Small mixers of the inclined-drum type are discharged by tilting the drum and pouring the contents into barrows, skips, or other receptacles. In the fixed-drum type a chute is inserted to collect and lead out the mixed concrete. In all types the drum usually rotates throughout the process of charging and discharging.

Before commencing a mixing shift the engine should be inspected ; oil, cooling water, and petrol (or other fuel) replenished ; and the engine allowed to run a few minutes to warm up. The drum of the mixer, together with all barrows, carts, and other containers, should be flushed out with water and any adhering pieces of concrete removed after every shift.

After all the materials have been put in the drum, mixing should continue for not less than a minute or until the mixture is of uniform colour. The rate of rotation is specified by some engineers to be such that the periphery of the drum maintains a speed of about 200 ft. a minute, while others specify a speed of 15 to 20 revolutions per minute and the provision of automatic counters to enable the speed to be registered. *Table* XVIII gives the equivalent number of revolutions per minute for drums of different diameters to give a peripheral speed of 200 ft. per minute.

TABLE XVIII.

SPEED OF MIXER DRUMS.

Diameter of drum (ft.)	3	4	5	6	7
Revolutions per minute to give peripheral speed of 200 ft. per minute	21	16	13	11	10

The strength of concrete is affected by the duration of the mixing process. Up to one minute the gain in strength is considerable, and mixing up to 1½ minutes also effects an improvement. There seems to be little advantage in mixing beyond two minutes. A practical period is not less than 1½ minutes between charging all the dry materials into the drum and discharging all the concrete. Some machines have a device for automatically locking the discharge lever until the batch has been in the rotating drum for the required period. If the number of revolutions made by the drum each minute is known the time can be estimated by counting the turns.

Some specifications require the materials to be well mixed in a dry state before adding water, but this is impracticable with some arrangements of large batching and mixing plants. With modern mixing machines the wet mixing is so efficient that the preliminary dry mixing process is superfluous except in the case of very stiff mixes such as are sometimes used in road work.

Using a single mixer for both dry and wet mixing is seldom successful. When preliminary dry mixing is demanded it can be either by hand or in a separate mixer.

If a powdered admixture in the concrete is specified, the material should be mixed dry with the cement and preferably in a separate machine before mixing with the aggregate and water. Some admixtures are mixed with the water which is added during mixing the concrete.

When the mixing plant is to be closed down for more than an hour the drum should be cleaned of all adhering concrete before the latter hardens. This can be done by putting two or three shovelsful of coarse aggregate and a couple of pails of water in the drum and allowing the latter to rotate for a minute or two, subsequently discharging the contents clear of the concreting position. Throughout the mixing period the mixer should be similarly cleaned at intervals. The drum of the mixer should be inverted at night or during any period when it is not working to prevent the accumulation of rain-water and to assist in draining out the water used for cleaning. Machines with fixed drums should be protected from in-driving rain and working parts should be covered by a tarpaulin.

Output of Concrete Mixers.

Concrete mixers are sometimes rated as $\frac{1}{4}$, $\frac{1}{2}$ or 1-cu. yd., but more accurately as 10/7, 14/10 and so on, the first number denoting the capacity of the drum in cubic feet of dry unmixed materials and the second number indicating in cubic feet the approximate amount of wet concrete obtainable in each batch.

Although the output per batch depends essentially on the capacity of the mixer, the output per hour of a mixing installation under working conditions is obviously equally dependent on the duration of the mixing process, the supply of materials to the mixer, and the speed at which the charging and discharging operations are performed. It is the speed of disposal of the concrete that usually limits the output of the mixer more than any other factor. Thus although a batch of any normal size can be mixed in less than three minutes the rate of disposal is seldom less than ten minutes per batch except on contracts specially organised for large outputs. The mixer outputs given in catalogues usually assume that the concrete can be disposed of as fast as it is mixed. Thus the specified output of a 10/7 mixer may be given as 60 to 80 cu. yd. per 8-hour day, which is practically one batch every 2 minutes. In practice the output that can be disposed of may be 25 per cent. or less of this maximum. In making piles, the daily output of a single mixer seldom exceeds 10 cu. yd. and may be as little as half that amount per day. For buildings like office blocks outputs of 20 to 40 cu. yd. per day are common, and there is no gain in mixing batches larger than $\frac{1}{2}$ cu. yd. unless the whole of the distribution arrangements are duplicated.

The maximum daily output of a mixer can be calculated as follows. Assume a 10/7 mixer, the output of which can be transported to the shuttering immediately it is discharged from the drum. The total time, working to the nearest quarter of a minute, occupied in mixing one batch would normally be :

	Minutes.
Raising skip and discharging into drum	$\frac{1}{4}$
Mixing in drum, adding water, etc. (During this time the skip is lowered and being reloaded)	1
Discharging contents of drum into wheelbarrows and completing loading of skip	1
Total	$2\frac{1}{4}$

$$\text{Number of batches in one working hour} = \frac{60}{2\frac{1}{4}} = 27.$$

In a day of eight working hours there would be in each period of four hours at least half an hour occupied in flushing out the drum, chutes, barrows, etc., before beginning work ; in filling the skip for the first batch ; in preparing, attending to, and starting the engine ; in periodically cleaning out the drum during and at the end of the working period ; and for clearing up at the end of the day.

This time might be longer if it is necessary to move the mixer about the site or adjust the position of a chuting plant. This leaves seven hours effective mixing time, producing $7 \times 27 = 189$ batches a day. Each batch may represent about 6 cu. ft. of wet concrete, giving $\frac{6 \times 189}{27} = 42$ cu. yd. of concrete a day. Delays may occur that may reduce this figure. Mechanical breakdowns can be minimised by giving careful attention to the engine and moving parts ; the employment of a good mechanic for this purpose pays for itself.

Other common causes of reduction of output include the unreadiness or inability of the concreting or distributing gangs to take the concrete as fast as it can be supplied to them, and failure of material supplies to the skips. The operations of measuring the materials and loading the skip need careful organisation to maintain an output of one batch every $2\frac{1}{4}$ minutes. Intermittent mixing, as happens where carpenters and steelworkers have to get clear of the working position before concreting can be resumed, considerably lowers the average output of the mixer. Small water towers, bunkers, and chimneys are examples of such intermittent working. As an example of bridge construction, on one well-organised contract the daily output of a 10/7 mixer was 32 cu. yd., or less than 80 per cent. of the output as estimated in the previous paragraph, while on another the daily output of a 7/5 mixer was 40 cu. yd., or equivalent to nearly 220 batches in eight working hours.

On a large building contract using a $\frac{1}{2}$-cu. yd. mixer and a chuting tower, an average rate of 20 cu. yd. an hour was maintained throughout the working day, which is equivalent to one and a half minutes for each batch. On another building contract with a two-minute mixing time enforced, a 10/7 mixer gave 50 cu. yd. in eleven hours, representing one batch every three minutes, thus allowing one minute for discharging and charging the drum.

Maintenance of maximum output requires close consideration of the number of men in attendance on the mixer, and this involves consideration of the amount of material to be handled. One cubic yard of 1 : 2 : 4 mix involves the measurement and loading of $1\frac{1}{3}$ tons of sand and stone and 4 to 5 cwt. of cement. This represents more than one man-hour per cubic yard of concrete and with a peak output of 7 to 8 cu. yd. an hour a gang of at least eight men, excluding the mixer operator and his assistant, would be required. Such a calculation emphasises the advantage of mechanically handling materials up to the mixer whenever large quantities at a high output rate are required.

For a small mixer (for example a 10/7 machine) with each batch of mixed concrete embodying one hundredweight of cement, an economical squad would be three men feeding materials into the skip, one man operating the mixer, and four men distributing the concrete either by four barrows or by two concrete carts with a couple of men to each.

Defective Concrete.

Concrete that has visibly commenced to set before it is placed should not be used. The operation of placing will disturb the setting action and may prevent completion of the hardening process. The mixing, distributing, and placing operations must be completed before setting commences, and concrete should be placed in its final position and free from disturbances within half an hour of adding the water. " Retempering," that is, breaking up and remixing with water, concrete that has reached its initial set should not be allowed, and is prohibited by most regulations. Partly-set concrete should be removed from the vicinity of mixing and placing in order to avoid the risk of it being incorporated in the work. If the quantity is small the loss is negligible, but if a large quantity is involved a use for it may be found in bulk mass concrete work. In this case the concrete should be broken into pieces a short while after the final set has taken place, and in order to allow it to harden fully, it should not be used for a period of two or three weeks. The pieces of hard concrete can then be used as plums or as large aggregate in work of secondary importance.

To avoid premature setting, as may happen when a period in excess of normal setting time has to elapse before the concrete can be placed, the addition of about 3 per cent. of hydrated calcium sulphate is sometimes made. This method of retarding the setting, however, should not be used except in unimportant work.

During the course of charging and discharging the mixer materials may be spilled on the staging or the ground. These should be cleared away, and not put back into the mixer as foreign matter may be taken up with them.

If a mix has been made too wet the practice of throwing shovelsful of cement, sand, or coarse aggregate into the drum to stiffen the mix should not be allowed. The wet material should be discharged into a container and remixed half at a time with fresh materials in the correct proportions to give the desired consistency. Care should be taken to ensure that the second half does not stand too long before being remixed and placed.

Cost of Mixed Concrete.

The estimation of the cost of concrete can be best done in two stages, (i) the cost of the materials in stock on the site, and (ii) the cost of transporting to the mixer, mixing, and discharging ready for distribution.

A precise estimate of the cost of materials in a cubic yard of concrete involves the consideration of numerous factors. First, of course, is the quoted price for the supply of these materials. Cement is usually quoted per ton delivered at the site either in paper bags, jute sacks, or other containers. If it is in jute sacks a charge for the hire of the sacks is made; usually 1*s*. 6*d*. is charged for each sack and this is refunded upon return of the sacks. A labour cost is involved in collecting, cleaning, and bundling the sacks in addition to the return carriage. An allowance should be made to cover damaged or lost sacks, and a note should be made of any bags delivered on the site in a damaged state. Paper bags are not returnable. Unloading cement may cost from 6*d*. to 1*s*. per ton, depending on the distance from the wagons to the store and the method adopted.

Aggregates are quoted by the ton or by the cubic yard. If by the ton, it is necessary to know how many cubic feet of the material make a ton as this may vary from 20 to 24 cu. ft. If the price does not include delivery to the site it is necessary to add transport charges. An addition of 5 to 10 per cent. should be added to the freight charges. If the materials are delivered by road the unloading charges are very small as the lorry or other vehicle can usually dump the material where it is wanted on the site, a sleeper road being laid where necessary. If it is delivered by rail or barge the material has to be unloaded unless special wagons are used. A fair average allowance for unloading wagons or filling crane skips by hand is two men per ton per hour; this will allow for supervision, trimming the heaps, and waiting time. If a grab is used there will be, in addition to the crane driver, two or three labourers clearing up spillings, emptying corners of the wagons, and trimming the heaps.

The following examples illustrate the method of compiling detailed estimates of the cost of concrete materials as deposited on the site and of mixed concrete.

To estimate the net cost of materials required for a cubic yard of 1 : 3 : 3 concrete if the prices of materials are:

Crushed stone.—2*s.* 6*d.* per ton (24 cu. ft.) at quarry, 30 miles by rail from site.

Sand.—8*s.* 6*d.* per ton (23 cu. ft.) delivered to site by lorry.

Cement.—40*s.* per ton delivered in sacks (11 per ton); 1*s.* 6*d.* hire charge on each sack.

The quantities per cubic yard of concrete would be based on *Table* XV for 1 : 2 : 4 mix (page 185).

For the proportions specified the volume of fine aggregate equals the volume of coarse aggregate.

$$= \left(\frac{11 + 22}{4 + 2}\right)3 = \qquad 16{\cdot}5 \text{ cu. ft.}$$

Add 5 per cent. for crushed stone	=	17·3 ,, ,,
Add 3 per cent. for waste, loss, etc.	=	say 18·0 ,, ,,

Weight of coarse aggregate per cubic yard of concrete

$$= \frac{18{\cdot}0}{24} \qquad = 0{\cdot}75 \text{ ton.}$$

$$\text{Weight of fine aggregate} = \frac{18{\cdot}0}{23} \text{ say} \qquad = 0{\cdot}80 \text{ ton.}$$

Weight of cement per cubic yard of concrete

	=	485 lb.
Add 5 per cent. for waste, grout, etc.	=	510 ,,
	=	0·23 ton.

The unit cost of each material would be computed as follows:

		s.	d.	s.	d.	
Coarse aggregate.—At quarry				2	6	per ton.
Freight: 20 miles at 2·15*d.*	=	3	7			
10 ,, ,, 1·05*d.*	=		11			
Terminal charges	=		10			
Use of railway wagons	=		10			
				6	2	,,
5 per cent. addition on freight	=				4	,,
Unloading by hand from railway wagons	=			3	0	,,
	Total =			12	0	,,

		s.	d.	
Fine aggregate.—Supply, including unloading		8	6	per ton.
Labour waiting on lorry, trimming heap, etc.			3	,,
	Total =	8	9	,,

Cement.—Delivered at site		40	0	,,
Cleaning, packing, and returning sacks		2	6	,,
Making good loss of sacks (1 per ton)		1	6	,,
Unloading from railway wagon to store		1	0	,,
	Total =	45	0	,,

Summary.—

Coarse aggregate,	0·75 ton at 12*s.*	=	9	0	per cu. yd. of concrete.
Fine aggregate,	0·80 ton at 8*s.* 9*d.*	=	7	0	,, ,, ,,
Cement,	0·23 ton at 45*s.*	=	10	4	,, ,, ,,
	Total =		26	4	,, ,, ,,

The cost of labour on the site included in this sum is about 3*s.* per cubic yard, and to this should be added, say, 5 per cent. for workmen's insurance and 5 per cent. for consumable stores and small tools. The materials for one cubic yard of concrete would then cost, say, 27*s.* 0*d.* in stock on the site.

If the quantity of material were considerable it would be worth while unloading the coarse aggregate by crane from the railway wagons and discharging it either on to a stock heap or into storage bins. An example of the cost of this operation is given in Chapter II.

For the purposes of the present example we will consider a building contract involving 1,600 cu. yd. of concrete, the contract time being nine months. Employing a single 10/7 mixer the principal charges would be as follows: Assuming that there are twenty full working days for each of eight of the nine months, the average output required would be 10 cu. yd. per day, but more than three times this amount might be required when the mixing gang and placing equipment are working at full capacity.

When working continuously the machine would produce an average of, say, twenty batches throughout an eight-hour working day. Each batch would be based on a 1-cwt. bag of cement and would give about 6 cu. ft. of concrete. Thus in one working day the output would be

$$\frac{20 \times 8 \times 6}{27} = 35 \text{ cu. yd.}$$

The subsequent calculations are based on an output of 30 cu. yd. a day.

Labour.—The labour required to maintain an average rate of one batch every three minutes, assuming the materials are stored in heaps close to the mixer, is

2 men measuring coarse aggregate and loading skip.
2 men measuring fine aggregate, loading skip, and assisting with coarse aggregate when necessary.
1 man fetching cement (in 2-cwt. sacks) from nearby store, opening bag, measuring, and discharging into skip.
1 man waiting on mechanic, fetching petrol, clearing-up around mixer, and helping with cement when necessary.

	s.	d.	
Total: 6 labourers at 1s. 1½d. an hour	54	0	per day.
1 mechanic on mixer, water control, etc., at 1s. 4d. per hour	10	8	,,
50 per cent. of concrete foreman's time (remaining 50 per cent. allocated to distribution and placing)	8	0	,,
	72	8	,,
Add 5 per cent. on net labour for small tools and consumable stores, and 5 per cent. for workmen's insurance = 10 per cent. of 72s. 8d.	7	4	,,
Total labour cost	80	0	,,

Hence labour cost $= \frac{80}{30} =$ 2s. 8d. per cu. yd.

PLANT.—The cost of mixers varies from say £15 for small hand-operated machines up to £120 for a 7/5 petrol-driven machine and upwards of £200 for 10/7 and larger machines. The power consumption depends on the rating of the engine; as a guide, a 3-h.p. engine driving a 7/5 mixer would consume about one gallon of petrol for each 20 cu. yd. of concrete mixed. Thus the cost of petrol and oil per cubic yard would be 2d. to 3d. The cost of the mixer would be assessed as follows:

	£	s.
Depreciation on mixer, costing £200, at 25 per cent. per annum, for 9 months (allowing for repairs)	37	10
Carriage to and from site	10	0
Erection and dismantling, including water connection services	12	10
Total	60	0

Hence plant cost $= \frac{£60}{1,600}$ cu. yd. = 9d. per cu. yd.

Summary.—	s.	d.	
Labour	2	8	per cu. yd.
Plant		9	,, ,,
Fuel, say		3	,, ,,
	3	8	,, ,,
10 per cent. for contingencies		4	,, ,,
Total	4	0	,, ,,

These costs allow for delivery into barrows, hoist skips, or other form of distributing plant, but exclude profit, general overhead charges, etc. Water is included in general charges. On this basis the plant charges will within limits be practically the same, whatever the daily output, whereas the labour charges depend primarily upon output. If instead of an average of one batch every three minutes, which might mean one batch every two minutes at peak-output, an average of six minutes per batch were allowed, that is, about three minutes per batch at peak-output, the mixing gang could be reduced to four labourers and a mixer operator, thus giving a daily labour cost of 59s. The daily output would then be reduced to 18 cu. yd., with the result that the cost of mixing the concrete would be increased to 4s. 9d. per cu. yd., assuming the same plant and fuel cost per cubic yard and the same percentage for contingencies.

The saving on higher outputs is due to the wages of the mixer operator and foreman and the cost of general attendance being spread over a larger quantity of material, while the men handling materials, although more in number, can be more fully employed with greater outputs.

Combining the material and mixing costs we have the following cost of one cubic yard of concrete :

Material in stock	27s.
Cost of mixing	4s.
	31s.

From the point of view of labour costs it is usually more economical to place the mixer adjacent to the material heaps, even if these are not in a central position compared with the concreting positions. Three barrowloads of unmixed aggregates are required for two barrowloads of wet concrete. Thus, if the mixer is remote from the material stores, three longer trips from the latter to the mixer are required together with two shorter trips from the mixer to the position where the concrete is placed. If the mixer is adjacent to the materials, only two longer trips from the mixer to the placing position are required with three short trips from the material store to the mixer.

SECTION IV.—DISTRIBUTION OF CONCRETE.

Methods of Distribution.

Various methods of transporting concrete from the mixer to the shuttering are in use to suit different conditions met with in practice. The introduction of special distributing plant increases the plant cost for any particular job, but for jobs of suitable size decreases the labour costs to such an extent that an overall economy results. The usual methods of distribution are :

(*a*) Barrows, concrete carts, wagons, or skips carried on bogies running on rail-tracks, or in motor-lorries for distribution to work at approximately the same level as the mixer ;

(*b*) Elevating in barrows, carts, skips or other containers to levels above mixing level, either by hoist or crane ; from the head of the hoist or from the point of delivery by crane the concrete may be distributed as in (*a*).

(*c*) Chutes used in conjunction with hoists for work at any level relative to mixer.

(*d*) Transporters, conveyors, pumps, tremies and other special distribution plant designed to meet the requirements of particular contracts.

When the amount of concrete is very small, when it is to be placed in awkward positions, or when access to the work is restricted, the concrete may be conveyed by hand in buckets. Such a method, being tedious, slow, and resulting in waste of materials and uneconomical use of labour, is expensive, but may be warranted when the value of the concrete work is only a small percentage of the total contract. For moderate distances and medium quantities, steel wheelbarrows holding about 2 cu. ft. of concrete are more commonly used. When larger outputs are required, hand-operated two-wheeled concrete carts

(*Fig.* 130) holding 7 to 14 cu. ft. of concrete are used; the larger sizes require two men to push them. A plank road is required between the mixer and the

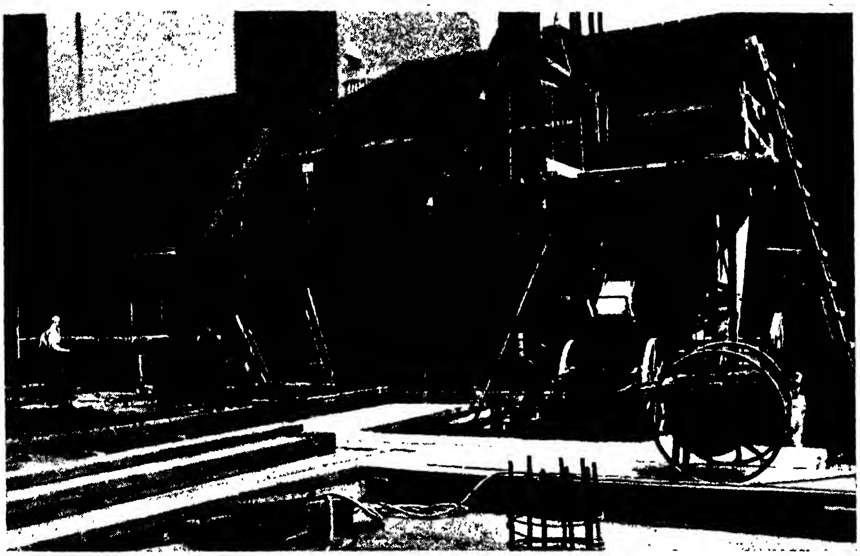


FIG. 130.—HOPPER AT HEAD OF HOIST.

place of deposit, unless the intervening surface is hard or free from irregularities. The loose plank roads shown in *Fig.* 130 are easily moved about the site as the

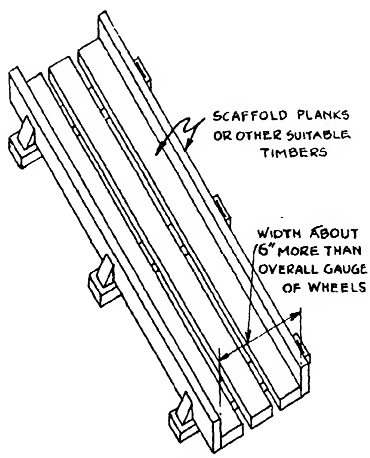


FIG. 131.—TIMBER ROAD FOR CONCRETE CARTS.

concreting position is changed. A more substantially constructed track is shown in *Fig.* 131; this is made from 9-in. by 2-in. planks, the upstanding edges acting as guides and the central plank as a walk-way. In the example illustrated in *Fig.* 132 a timber runway has been constructed around the periphery of the work from which concrete carts can discharge at any required position. Passing

Fig. 132.—Runway for Concrete Cart.

places are arranged as shown. Single-wheel barrows are run on planks resting on bearers to overcome irregularities in the ground. When the track crosses reinforcement the planks should be elevated so that the steel is not disturbed.

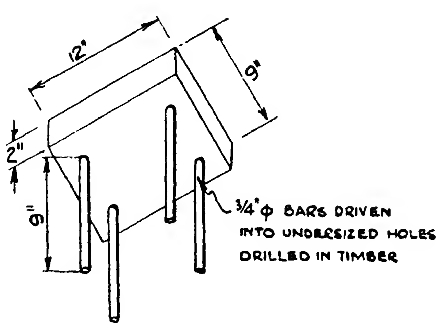


Fig. 133.—Stool for Barrow Run.

A form of stool illustrated in *Fig.* 133 is a convenient type of bearing for raising a single-plank track a few inches above the surface.

The advantage of transport by barrows and carts is that the route can be easily adjusted to lead to any part of the work, but such adjustment is not so convenient when rail-tracks are used. Transport by jubilee side-tipping skips running on narrow-gauge rail-tracks is more useful when a large volume of material is to be conveyed along a single route over longer distances than are practicable with barrows or carts. Adjustment of the position of the main route is not readily made, but the railhead is capable of some lateral movement to suit different positions of deposit. Narrow-gauge tracks can also be laid for the conveyance of bogie-platforms carrying skips of concrete. The wagons may be either propelled by hand one at a time, or drawn in trains by an oil, steam, or electric locomotive, or in special cases, as in tunnel work, by ponies. A number of the illustrations of construction work given in this volume demonstrate various adaptations of narrow-gauge track transport.

With all types of distribution by individual containers it is advisable to tip the concrete from the container on to a timber banker, similar to that shown on *Fig.* 134, from which it is shovelled into the shuttering. Only in the case of

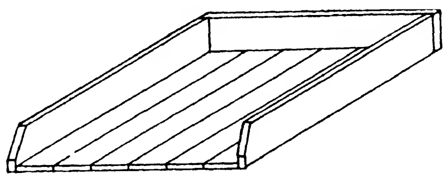


FIG. 134.—CONCRETE BANKER.

bulky work and slabs of large area should the concrete be discharged directly from the container into or on the shuttering.

Transport by motor lorries is usually associated with central-mixing plants placed some distance, even a few miles, from the site of the work. Special lorries have been designed for this work, but for short distances ordinary open steel-body lorries have been successfully used. When employing the latter the interiors should be sprayed with water and sprinkled with sand before each loading. Tarpaulins or other covers should be provided to protect the wet concrete from the sun, wind, or rain, and the time occupied on the journey should be as short as possible. An alternative to a central-mixing plant from which the mixed concrete is delivered in an open lorry, is the supply of ready-mixed concrete by truck mixers operating in conjunction with a central batching plant. Such an installation is illustrated in *Fig.* 135, where the materials are proportioned by weight. The equipment, which has a capacity of 100 cu. yd. per hour, consists of a pair of aggregate storage hoppers which are fed by a conveyor. From the hoppers the materials are discharged into weighing hoppers and, by means of special scales, the water content of the aggregates is recorded. The weight of aggregate in each batch is then adjusted accordingly. A separate hopper and scale are provided for the cement, and the whole of the materials in their correct proportions are discharged dry into the drum of a mixer mounted on the motor lorry. The latter also carries a tank which is filled with the correct volume of water for each batch, allowing for the moisture in the aggregates. In some systems, water is not added to the dry materials nor is the mixing started until the lorry is within a few minutes' run from the site, while other suppliers continuously mix the concrete throughout the journey. Upon arrival the

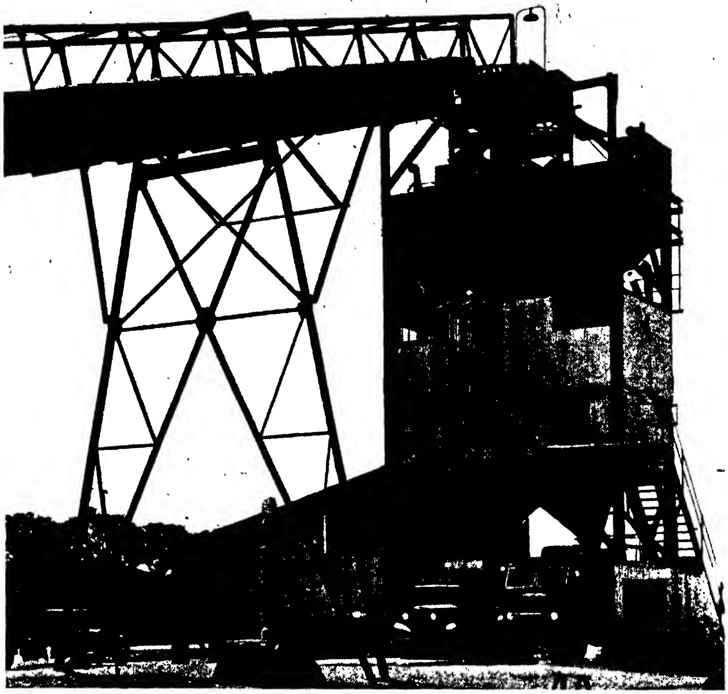


FIG. 135.—READY-MIXED CONCRETE PLANT.

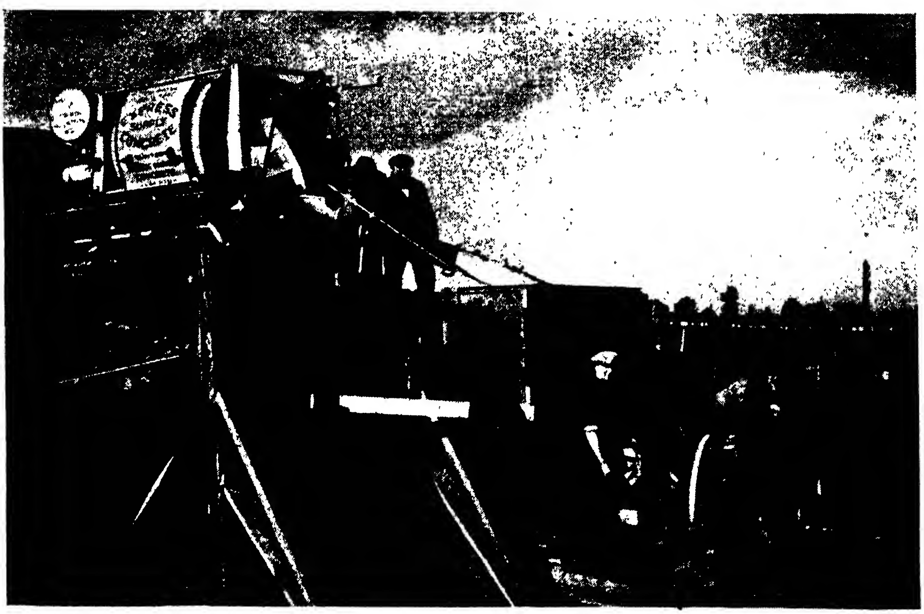


FIG. 136.—DISCHARGING READY-MIXED CONCRETE.

concrete is discharged through a chute (*Fig.* 136) into a hopper from which concrete carts for distribution are fed. This system of mixing and distribution is useful where several small contracts are going on simultaneously.

When concrete is to be placed below the general ground level, the mixer can be placed on the upper level and discharge the concrete to the lower level through a short chute of corrugated iron or timber leading to a banker, into barrows, carts or other transporters, or in the case of heavy work directly into the final position. The use of a timber chute is illustrated in *Fig.* 137, where

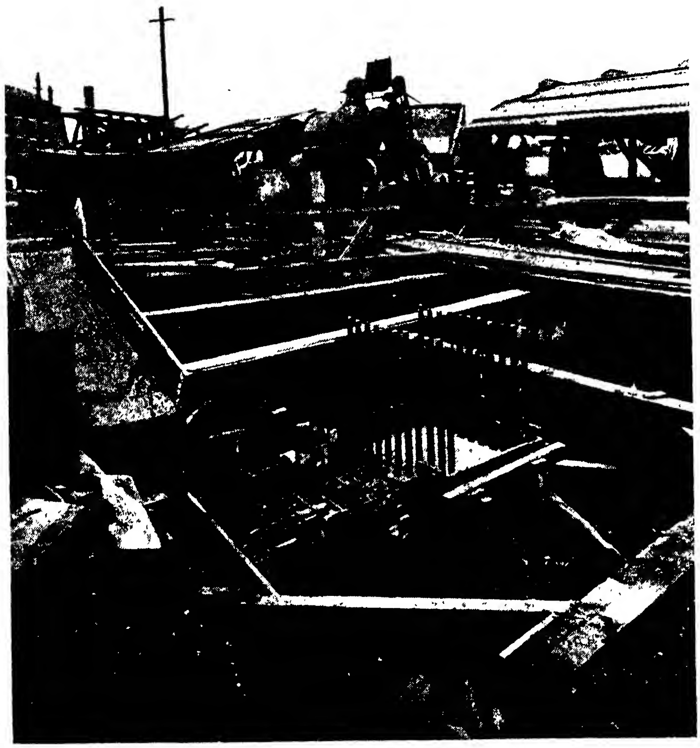


FIG. 137.—PLACING CONCRETE BELOW GROUND LEVEL.

the mixer is fixed in a position convenient for the reception of supplies of materials. The mixer feeds into a skip mounted on a jubilee track laid around the sides of the excavation. Turntables are provided at the corners to save the space otherwise required for curves and switches. The skip is run along to the point where concrete placing is proceeding and discharges down the movable inclined timber chute illustrated, directly into the shuttering, where the concrete is tamped into position around the reinforcement. In the example in *Fig.* 138 the mixer can be brought right up to the work and discharge down a wooden chute directly into the shuttering for the abutments of a small bridge.

If the final position of the concrete cannot be directly reached conveniently from the sides of the excavation, the mixer may be placed on the edge of the

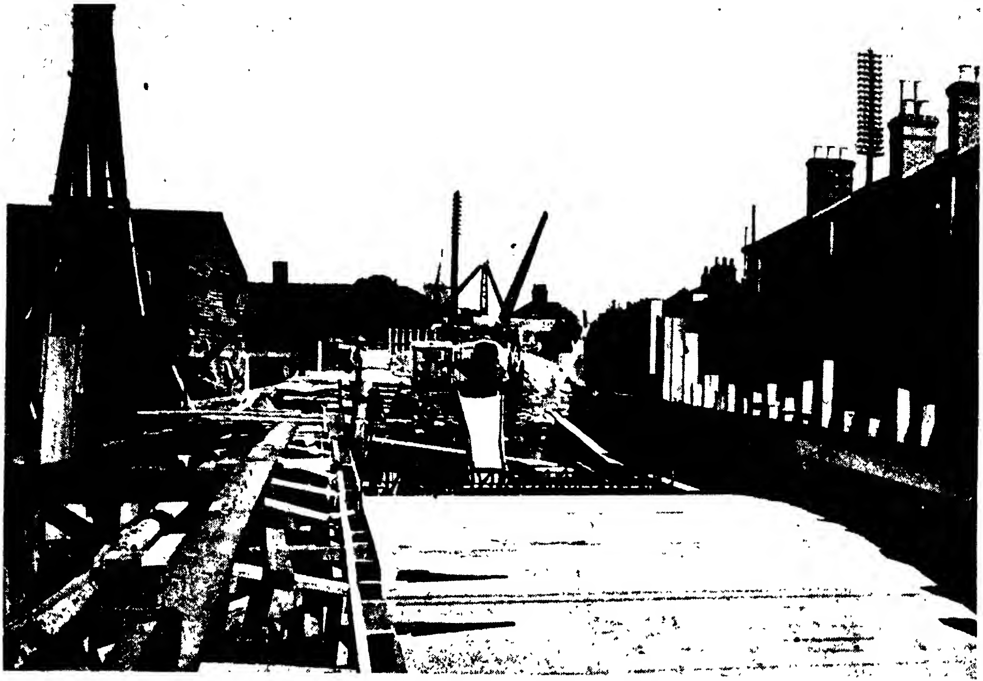


FIG. 138.—DIRECT DISCHARGE FROM MIXER.

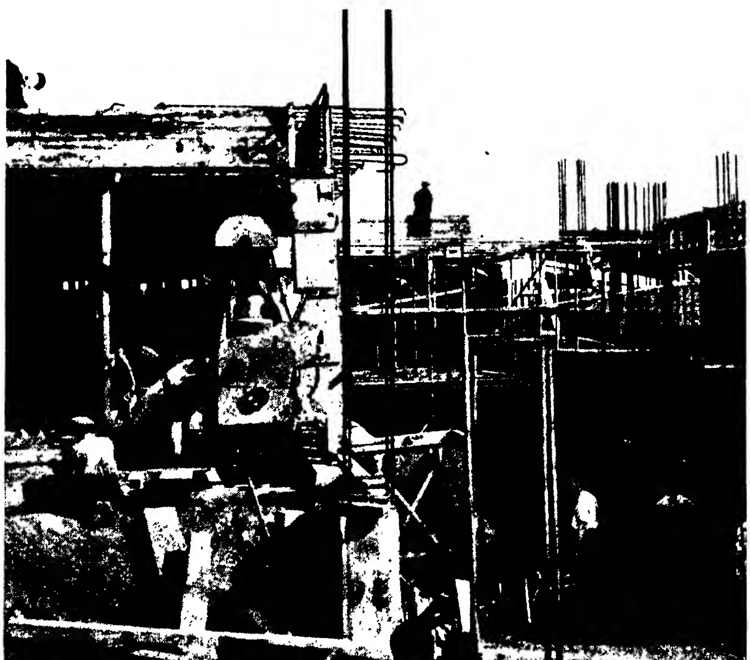


FIG. 139.—FEEDING BARROW FROM HOPPER.

excavation and discharge down a chute into barrows or into a hopper from which the barrows can be fed, as in *Fig.* 139. Alternatively the mixer may, as in *Fig.* 140, feed directly into jubilee wagons running on tracks laid on the bottom of the excavation. When the amount of concrete to be placed below the general ground level is considerable it is convenient to arrange a central mixing plant adjacent to the material store. Such an arrangement is shown in *Fig.* 141 where three ½-cu. yd. mixers are installed on the bottom of the excavation and feed

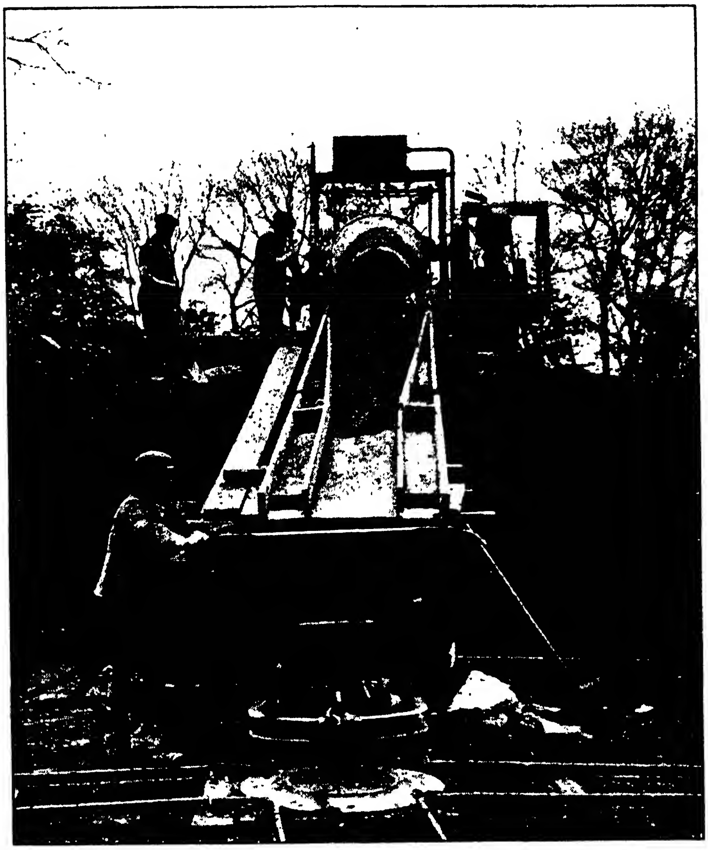


FIG. 140.—LOADING JUBILEE WAGONS.

into jubilee wagons for distribution. With the material heaps located as shown, lorries can discharge direct from the road level.

On bridge and marine work, and occasionally on building contracts, cranes are used to handle skips of concrete. In building work the crane may be erected primarily for handling other materials and its use as a distributor of concrete is only secondary. In construction in or over water a crane is perhaps the most convenient means of transporting concrete from mixers on shore to working positions off-shore. This convenience is particularly marked when concrete is to be lowered within cofferdams for bridge piers, or when concrete is to be deposited under water. Skips used with cranes are generally of the tilting-bucket type when large masses of concrete are to be placed, of the bottom-opening

FIG. 141.—MIXERS INSTALLED AT LOW LEVEL.

type when placing under water or within narrow cofferdams, and of the side-gate type when depositing on bankers or directly into shuttering. A tipping or tilting-bucket type skip, as shown in *Fig.* 142, is constructed from mild steel

FIG. 142.—TIPPING SKIP.

plates, the trunnions, hoop, bales and catches being of cast iron. The trunnions are so arranged that when the catch is released with the skip full, the latter tips and rights itself when empty. In the double-catch type illustrated, the two arms work independently, thus allowing control of the direction of tip. Skips of this class are made in nominal capacities from 1 cu. yd. down to $\frac{1}{8}$ cu. yd., but when heaped with dry material they contain more than the rated capacity. When filled with wet concrete, the amount of material is slightly below the rated capacity so as to allow freeboard to prevent spilling during movement. A bottom-opening skip is illustrated in *Fig.* 143, where a large capacity skip is shown depositing concrete directly into a heavily reinforced beam.

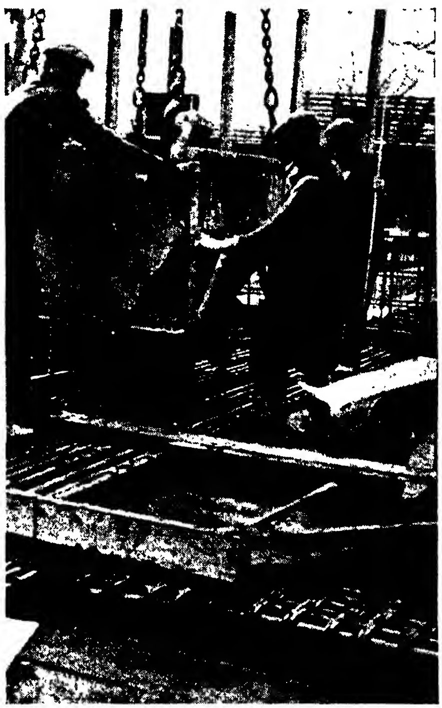


FIG. 143.—BOTTOM OPENING SKIP.

Distribution of concrete by hoists, chuting plants, and pumps, detailed consideration of which is given in subsequent paragraphs, is suitable for placing concrete below, at, or above ground level. Other appliances that are less commonly used but have been installed on large contracts include transporters, cableways, and conveyors.

Transporters have been employed on such works as the floors of transit sheds, warehouses, wharves, roads, and casting yards. They consist essentially of an elevated steel girder spanning the width of the work, the supporting frames being mounted on bogies running on rails laid parallel to the length of the work. The transporter, upon which the concrete mixer is carried, travels up and down the work as necessary and a traversing crab conveys skips of concrete to the required positions. An example of this class of mixing-distributing installation is illustrated in *Fig.* 144, where a transporter is shown operating in a pile yard.

Cableways, which have been successfully used for distributing skips of concrete over dams and viaduct constructions, and which are only suitable for such major works, require to be specially designed for the contract in view.

Horizontal and inclined belt conveyors, singly or in series, are only suitable to carry " dry " concretes having slumps of one to two inches. Open-air conveyors are susceptible to weather variations, high winds and high temperatures

FIG. 144.—CONCRETE DISTRIBUTOR IN PILE YARD: MIXER ON LEFT OF GANTRY.

causing evaporation of the moisture and rain increasing the water content. To maintain conveyors working continuously in all weathers, they are therefore often provided with light sheet-metal covers. A usual width of belt is 24 in., and at speeds of 250 to 300 ft. per minute such a belt can handle about 40 cu. yd. per hour. The surface of the belt is usually of rubber, with a four-ply or five-ply backing. A conveyor installation might cost from 10*s*. to 15*s*. per foot run complete, the power required to operate a 24-in. horizontal concrete conveyor 60 ft. long being 15 to 20 horse-power. At the discharge end of the belt where a metal scraper is attached to ensure that all the concrete is removed the concrete is tipped into a hopper to counteract any tendencies to segregate.

Hoists for Concrete.

The commonest type of concrete hoists are vertical, the simplest appliance being a chain sling suspended from a pulley and operated by a power winch at ground level. The sling is attached to the barrow or cart containing the concrete and, as shown in *Fig.* 145, the container is lifted bodily to the working level.

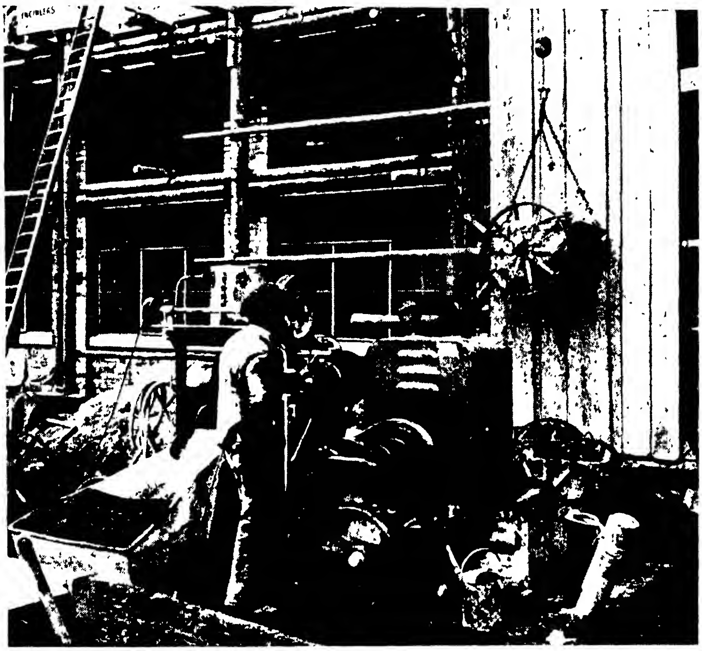


FIG. 145.—SLING HOIST FOR CONCRETE CARTS.

Alternatively the concrete can be contained in skips and lifted in a similar way. A simple form of pulley frame for hoisting concrete either in skips, buckets, barrows, or carts is illustrated in *Fig.* 146. The frame consists of a pair of 9-in. by 3-in. timbers set on edge, packed out and bolted together at the extremities. Two pulleys are provided, the rope from the hoist passing over one pulley and across the top of the second from which the load is suspended. The bearings for the pulleys consist of metal rings cut from suitable piping and let into the timber frame, the axles being loose-fitting in the bearings. Since the friction of roughly made bearings of this kind is appreciable they should be well lubricated by packing with grease. When in service the pulley frame is lashed to the top of the scaffolding or to a specially-constructed timber tower. The sizes of ropes, the diameter of pulleys, and the power and running speeds of winches suitable for hoisting concrete or similar work are discussed in Chapter I.

Barrows and carts can also be elevated by platform hoists operating on vertical steel guides. In this type the containers are wheeled directly on to the platform and when elevated to the required level are wheeled off. Platform hoists may have either single or double platforms ; in the latter case, one platform descends while the other is being elevated. In the smaller sizes the platform may be capable of taking one barrow only, but larger sizes may take two barrows or a jubilee skip. The hoisting speeds vary from 120 to 200 ft. a minute, depending on the power of the motor or engine and on the load being lifted. For example, a ½-ton platform hoist driven by an 8-h.p. electric motor lifts its load at about 190 ft. a minute, whereas if the hoist is fitted with an 8-h.p. petrol engine, the working load is ¼ ton at 170 ft. a minute or 7 cwt. at 130 ft. a minute. Similarly a ¾-ton platform hoist fitted with a 12-h.p. electric motor operates at 200 ft. a minute, while if provided with a 9-h.p. petrol engine the working loads are ½ ton at 190 ft. a minute or ¾ ton at 125 ft. a minute. Larger platform hoists

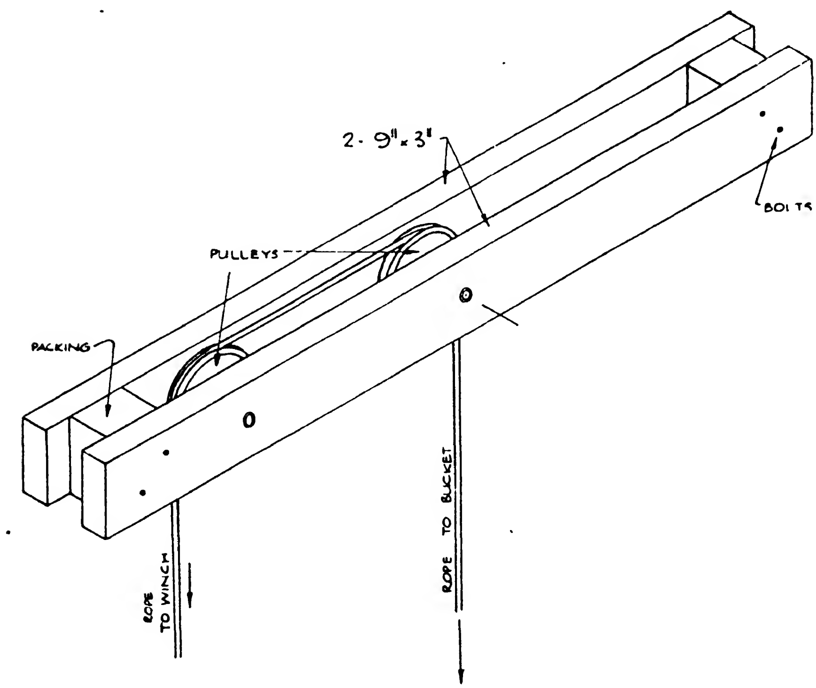


FIG. 146.—PULLEY FRAME FOR CONCRETE HOIST.

capable of lifting 1½ tons at 150 ft. a minute are obtainable. A convenient value for assessing the power of a hoist required for lifting a given amount of concrete is to consider that 1 cu. yd. of wet concrete together with the container weighs 2 tons. Thus to deal with the output of a 10/7 mixer, the batch capacity of which is about ¼ cu. yd., a hoist having a lifting capacity of not less than ½ ton would be required. It is not advisable to work the machine at full load continuously, therefore a hoist nominally rated at ¾ ton would be suitable for dealing with the complete output of a ¼ cu. yd. mixer and would allow a margin for additional loading when necessary.

Skip-hoists, instead of accommodating removable concrete containers, are fitted with a one-way or two-way side tipping skip, as shown on *Fig.* 147. The concrete is discharged from the mixer into the skip, elevated, and discharged at the working level into barrows or carts for transportation to the placing position. If a hopper fitted with a discharge gate as shown on *Fig.* 147 is placed at the head of the hoist the skip can discharge directly into this. The hopper

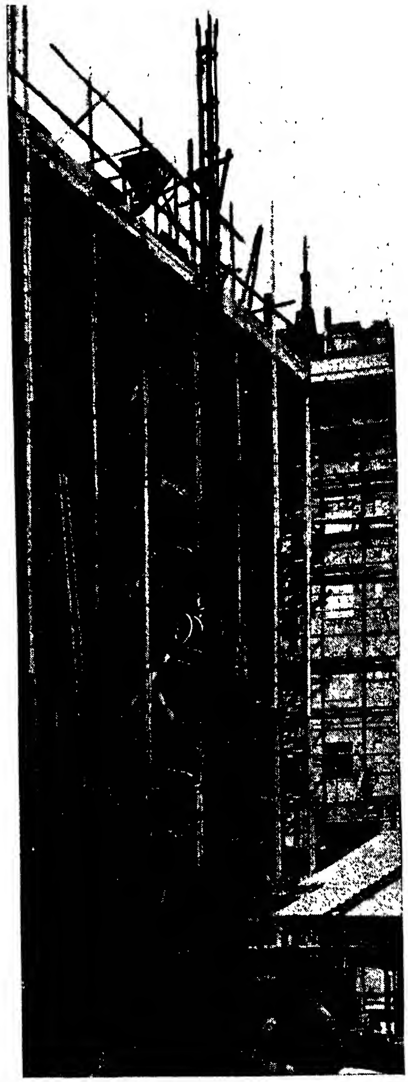


FIG. 147.—SKIP HOIST.

then acts as a storage from which the barrows or carts can be filled, thus avoiding the need for precise co-ordination of the mixing, hoisting, and transportation operations. Platform hoists are usually made so that they can readily be converted into skip hoists.

Vertical skip-hoists are often associated with distribution by inclined chutes, in which case the skip discharges into a hopper head feeding the chutes. When hoists are erected outside the boundary of the actual structure, as is usually most convenient, the skip or platform moves within a tower to which the guides are attached; in chuting plants the guide rails may be attached to a vertical guyed steel mast. Hoist towers may be constructed either in steel or timber or may be formed within the scaffolding.

A timber hoist tower (*Fig.* 148) can be conveniently made on the site. In the case illustrated the concrete is hoisted in a skip, but a similar arrangement

FIG. 148.—TIMBER HOIST TOWER.

can be used to lift wheelbarrows or carts on a platform. A platform hoist operating in a timber tower is illustrated in *Fig.* 149. The usual overall sizes of a steel wheelbarrow are 2 ft. 3 in. wide by 5 ft. 3 in. long over wheel and handles. The dimensions of the platform and the cage within which it operates should be sufficient to allow a clearance over these dimensions. As such towers are built beyond the ultimate top level of the work the head of the tower must be braced by guys extending to ground level. The quantity of material and cost of erection of a timber hoist tower 90 ft. high of similar construction to that shown in *Fig.* 148 can be assessed as follows, assuming the plan dimensions are 6 ft. by 4 ft. with horizontal braces placed at intervals of 6 ft.

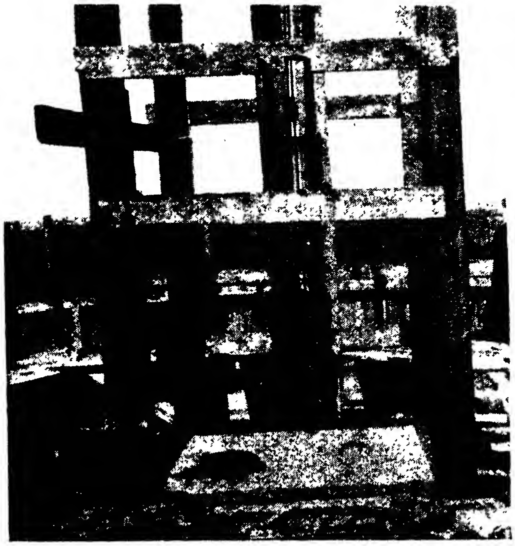


FIG. 149.—PLATFORM HOIST IN TIMBER TOWER.

The length of timber required would be:

Verticals (double timbers)	$2 \times 4 \times 90$ ft.	=	720 ft.
Horizontals	$2 \times 15 \times 7$ ft.	=	210 ,,
	$2 \times 15 \times 5$ ft.	=	150 ,,
Diagonals	29×9 ft.	=	261 ,,
	30×6 ft. 6 in.	=	195 ,,
		Total =	1,536 ft.

Allowing for waste the quantity required is 1,600 lin. ft. of 5-in. by $2\frac{1}{2}$-in. timbers,

$$= 1{,}600 \times \frac{5 \times 2\frac{1}{2}}{144} = 140 \text{ cu. ft., say, 1 standard of timber } (= 165 \text{ cu. ft.}).$$

The number of bolts required is about 350, and if $\frac{5}{8}$-in. diameter by 9 in. long, each with two washers, their weight is about $1\frac{1}{2}$ cwt. The cost would be estimated thus:

	£	s.	d.
One standard of timber at £19	19	0	0
$1\frac{1}{2}$ cwt. of bolts at 25s.	1	17	6
	20	17	6
Deduct $33\frac{1}{3}$ per cent. salvage value	6	19	0
Net material	13	18	6
Labour in erecting and dismantling 140 cu. ft. of timber, say	10	10	0
Total cost	£24	8	6

or, say, 5s. 6d. per foot height of tower.

To this must be added the cost of the skip, platform, or barrow sling, pulley, operating hoist, guides, ropes, and other attachments, the value of which is indeterminate in a general example. Much depends on whether the items concerned are in the contractor's stock or have to be purchased or made specially for the work in hand.

When construction necessitates an external scaffold no special tower is required to accommodate the concrete hoist, the guides being erected within a cage formed from the scaffolding members. An illustration using timber scaffold poles is given on *Fig.* 150, while *Fig.* 151 shows tubular steel scaffolding used as a hoist tower. If steel scaffolding is used for the whole structure the hori-

FIG. 150.—SCAFFOLD POLE HOIST TOWER.

zontal rakers are attached to adjacent vertical and horizontal scaffolding members. In the case illustrated, where no external scaffolding is provided, the rakers are attached by bolts through the completed section of the concrete walls.

When the site does not permit an external hoist, as in city buildings on confined sites, the hoist must be erected within the building and built up in sections, the topmost of which will extend a few feet above the working level. The guide-rails can be held at each floor level by struts, but the projecting portion will require to be guyed. In the absence of adjacent scaffolding, anchors for these guy ropes can be formed as each floor is constructed and arranged as in *Fig.* 152. The anchor can consist of a loop of $\frac{3}{8}$-in. diameter bar projecting a few inches above the concreted floor; when no longer required the loop can be cut off or bent down and concealed by the floor surfacing.

A vertical hoist can frequently be arranged in multi-floor buildings to operate within the space allotted for stair or lift wells, otherwise temporary openings have to be formed in the floor panels to allow a passage for the hoist. Building regulations require temporary openings to be protected by a guard rail or by a toe board. The location of these temporary openings requires careful considera-

FIG. 151.—HOIST WITHIN STEEL SCAFFOLDING.

tion; recommended dimensions relative to their size and position are given in *Fig.* 153. If openings in other positions or of other dimensions are required, due to the size of the hoist or due to irregular planning of successive floor panels, special permission should be obtained from the engineer as reconsideration of the design of the panels affected may be necessary; a reduction in the distance between the edge of the hole and the side of the beam may also affect the design of the beam if this has been computed as a tee-beam. For other conditions of continuity or of two-way spanning the position of the opening should be fixed after consultation with the engineer, and in all cases, if possible, the position of the opening should be decided before the drawings showing the slab reinforcement are prepared. If the opening measures more than 3 ft. in either direction, and if no special provision has been made for the adjustment of the reinforcement, the procedure indicated in *Fig.* 153 can be followed.

The main reinforcement should be cut alternately at the places indicated, and the two ends of each bar hooked and bent down to form a clear opening of the required dimension. Some steel fixers advocate bending the bars upwards, and it may be better to leave it to the man to decide in which way he thinks he will do least damage to the concrete when rebending the bar ; in considering this, however, it is obviously essential that the temporarily bent bars should

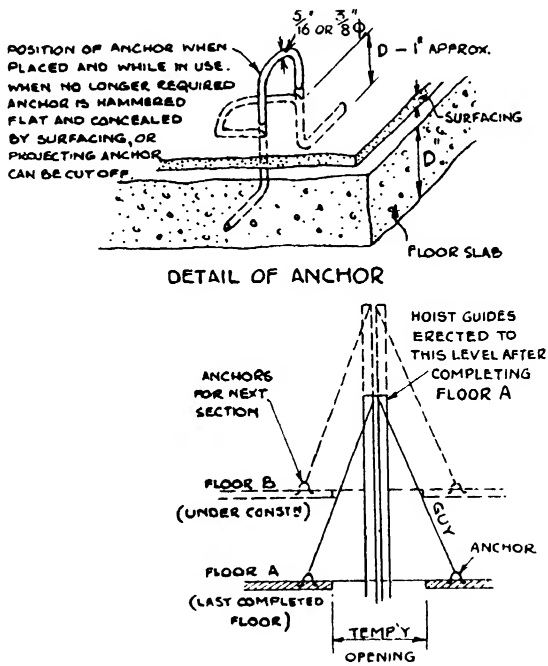


FIG. 152.—ANCHORAGE FOR HOIST GUYS.

not obstruct the access to the hoist platform at each floor level. When the hoist has been removed and the panel is to be filled in the bars should be carefully bent back. The concrete around the hole should be cut back sufficiently to remove all material damaged by rebending or otherwise. The surface should be left rough and shaped as indicated in *Fig.* 153. Additional bars of the same diameter and at the same spacing as the original reinforcement should be fixed. If the width of the opening in the direction of the span of the slab exceeds three-quarters of the span, larger bars or closer spacing may be required since the newly laid concrete may not properly be considered as being monolithic with the older material around the hole, as after a considerable lapse of time the bond may be of doubtful value or insufficient allowances for continuity may obtain. The cost of the extra reinforcement required for making good the opening and all labours involved are generally a charge against the contractors unless arrangements are made to the contrary. The shuttering for closing the opening

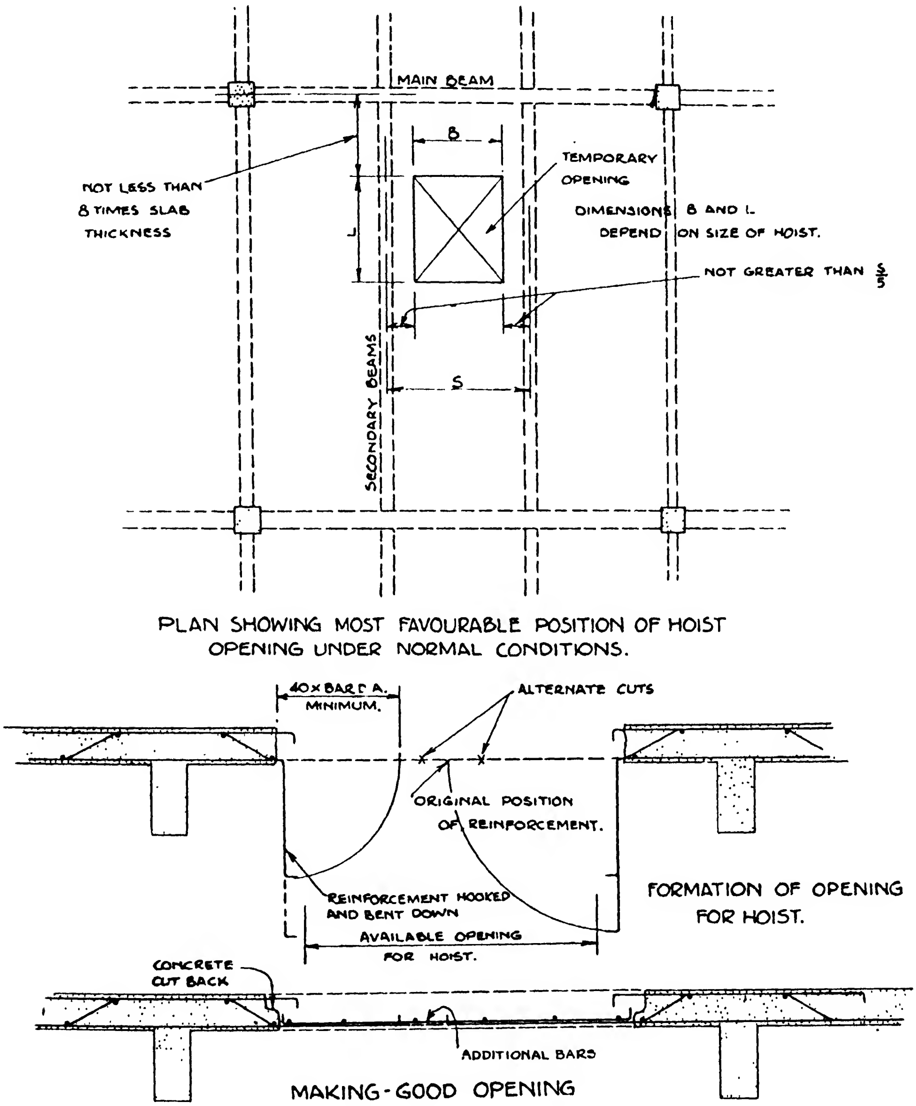


FIG. 153.—TEMPORARY OPENING FOR HOIST.

should only be erected after the cutting-away has been completed and after close inspection of the underside of the slab for defective concrete.

Inclined hoists, although not common, are useful when the concrete has to be lifted only a few feet ; they may be used in conjunction with a system of chutes.

Cranes are used in place of hoists, principally to lift skips of concrete from the discharge side of the mixer to the place where it is to be deposited in the work. As with other methods the contents of the skip are discharged on to a banker, from which they are placed into the shuttering by hand unless the work is of such a kind that direct tipping is allowable. If part of the work is outside the reach of the crane, the skips may discharge at the higher level into wagons, or may be placed on bogies for transport to the shutters.

Chuting Plant.

Distribution of concrete by a chuting installation is a convenient method when fairly large areas have to be covered, whether at ground level, as in reservoir floors, or at various elevations as in large multi-story buildings. The chutes swivel about a central hoist-tower, typical arrangements being illustrated in *Figs.* 154, 155 and 157, the latter showing a twin installation on an extensive contract. It is useful, before installing a chuting plant, to prepare a "covering plan" of the proposed equipment to ensure that with the hoist or mast in a given position the line of chutes and the end swivelling member will control the whole of the work or will reduce transport beyond the extreme range of the chutes to a minimum. In *Fig.* 156 a covering plan for a building on a corner site is illustrated.

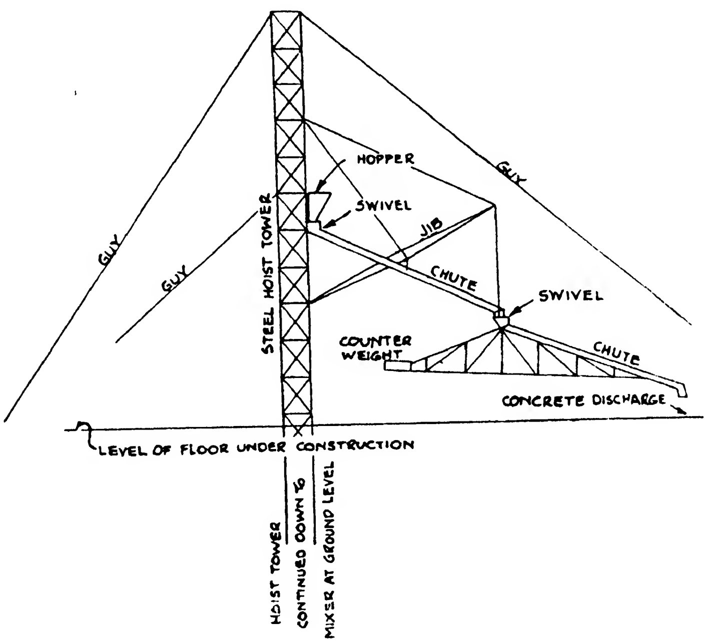


FIG. 154.—VERTICAL HOIST AND CHUTES.

The tower or mast, up which the concrete is elevated in a tilting skip, is guyed with ½-in. diameter wire ropes as for hoist towers described in a previous paragraph, the guys being adjusted by turnbuckles. The chutes are usually of 14 or 16-gauge half-round section, stiffened by angles along the upper edges

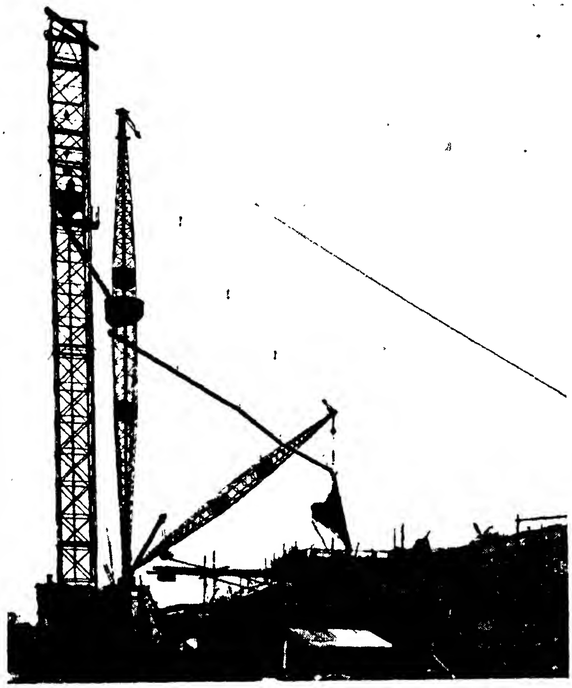


FIG. 155.—CHUTING PLANT.

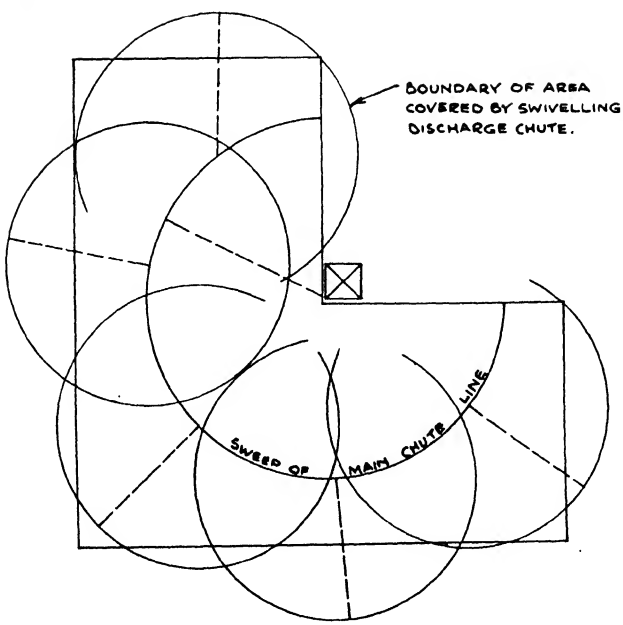


FIG. 156.—COVERING PLAN FOR CHUTING SYSTEM.

with transverse ties at intervals. The life of such chutes is about 2,000 cu. yd. of concrete. At the discharge end of each section of the chute a short vertical apron-piece is fitted. The slope of the line of the chutes should be uniform so that regular flow takes place throughout the line, and depends on the distance of the end of the line from the tower ; if the consistency of the concrete is maintained uniform, the slope increases with this distance. The slope should be such that the concrete flows without the addition of excessive water and without separation of the coarse aggregate from the mortar. A slope between one in two and one in three, that is about 30 deg., is usually suitable for flow without separation. The slope from the tower to the discharge position can be adjusted by altering the position of the receiving hopper on the tower.

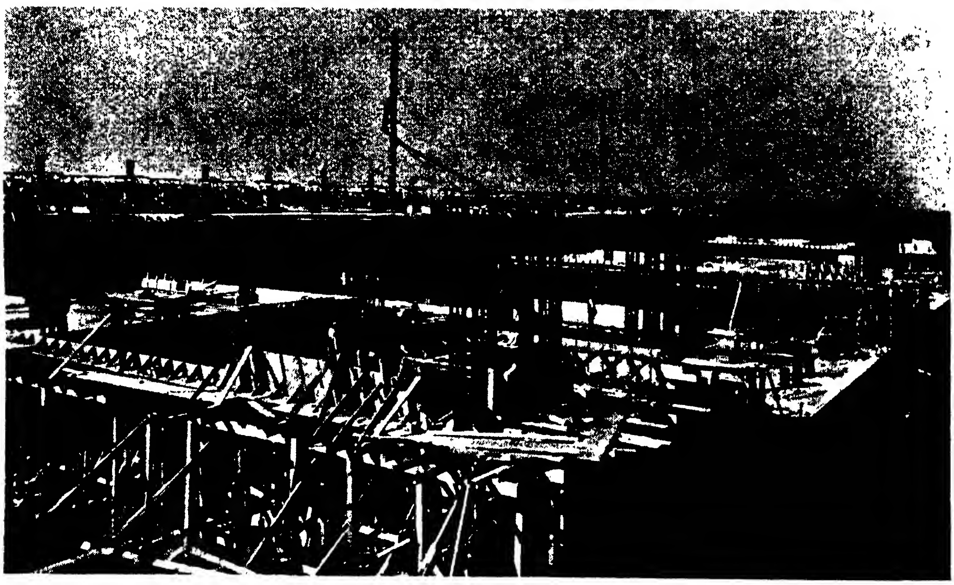


FIG. 157.—TWIN CHUTING INSTALLATION.

A sluggish flow can be speeded up along short chutes by tapping or by the aid of rods or pokers or, in the case of long lines of inaccessible chutes, by attaching light pneumatic vibrators to the chutes. When heavy aggregates are used separation may be experienced, but this tendency can be overcome by using a flatter slope and by increasing the amount of mortar in proportion to the amount of coarse aggregate. As an additional safeguard against separation the concrete should be discharged from the mouth of the chute on to a banker, from which it can be shovelled into the shuttering.

If large quantities of concrete have to be dealt with in short periods, the arrangement illustrated in *Fig.* 157 can be adopted. Here the chute, operating from masts, discharges into a transportable hopper from which barrows or concrete carts are fed for transport to the shuttering. This equipment avoids intermediate handling and frequent movement of the chuting line.

If the flow in the chute is continuous direct feeding into final positions may be permissible, except in work of narrow dimensions, but with the usual conditions of intermittent flow the concrete should always be discharged on to a banker and fed into beam, column, or wall shutters by hand. A possible exception to this double process may be made in the case of slabs. In all cases, it is clearly most economical to have the delivery end of the chute as near the concreting position as possible, but when this distance is so great that wheeling is necessary the concrete can be fed directly into the carts or barrows.

Before, after, and at intervals during a concreting shift, the chutes, auxiliary hoppers, bankers, and any other apparatus should be thoroughly washed down with water. This is easily effected at the same time as the mixer is washed out by discharging the drum-cleaning water into the skip-hoist and discharging it down the chutes. Water used for this purpose should be discharged clear of the shuttering or unhardened concrete.

Pumping Concrete.

Pumping concrete from a central mixing plant through pipe-lines to remote parts of a structure is another useful method of distribution. The plant consists essentially of a pump and a pipe line, an example of the former being shown in *Fig.* 158. A typical pipe-line is illustrated in *Fig.* 159, while the discharge end of a pipe is shown in *Figs.* 161 and 279. The concrete is fed into a hopper surmounting the pump; this process is illustrated in *Fig.* 160. Two sizes of pumps are obtainable, both of the single-acting, reciprocating piston type. One size is capable of discharging 15 to 20 cu. yd. hourly and a smaller machine will deal with up to 10 cu. yd. an hour. Under favourable conditions the larger size pump can maintain an average delivery of 23 cu. yd. per hour. The output of a pump varies slightly with the consistency of the concrete and the length of the pipe-line, but to ensure continuous discharge it is essential for the mixing plant to be of adequate capacity. The larger pump would require a mixer not smaller than 21/14 and the smaller pump not smaller than 14/10. Alternatively two mixers can be used to feed alternately through short chutes into the pump hopper. Intermittent working can be carried out, as the pump can be stopped and restarted when necessary. In all cases the pump should be placed close to the mixer, a direct feed from the latter into the hopper being arranged by placing the mixer on a platform over the pump. From the hopper the concrete falls by gravity through the inlet valve into the cylinder.

Concrete pumps are usually operated by electric power, but when this is not available it is possible to drive them by a petrol, compressed-air, or steam engine.

The pipe-line leading from the pump to the concrete placing position is composed of lengths of light drawn-steel pipes, connected with couplings enabling the laying, adjustment, and dismantling of the pipe line to be effected in a short time. The coupling action is provided by steel wedges driven between the flange on the spigot of one pipe and lugs projecting from the socket of the adjacent length. The pipes are 6-in. internal diameter for large-capacity pumps and 4½-in. diameter for the smaller machine; they are supplied in standard straight lengths of 10 ft., but 7 ft., 3 ft. 6 in., and 1 ft. 9 in. are also obtainable. Bends are made to 90, 150, and 135 deg. A 6-in. pipe will take concrete with 2½-in.

Fig. 158.—Concrete Pump.

Fig. 159.—Pipe Line from Concrete Pump.

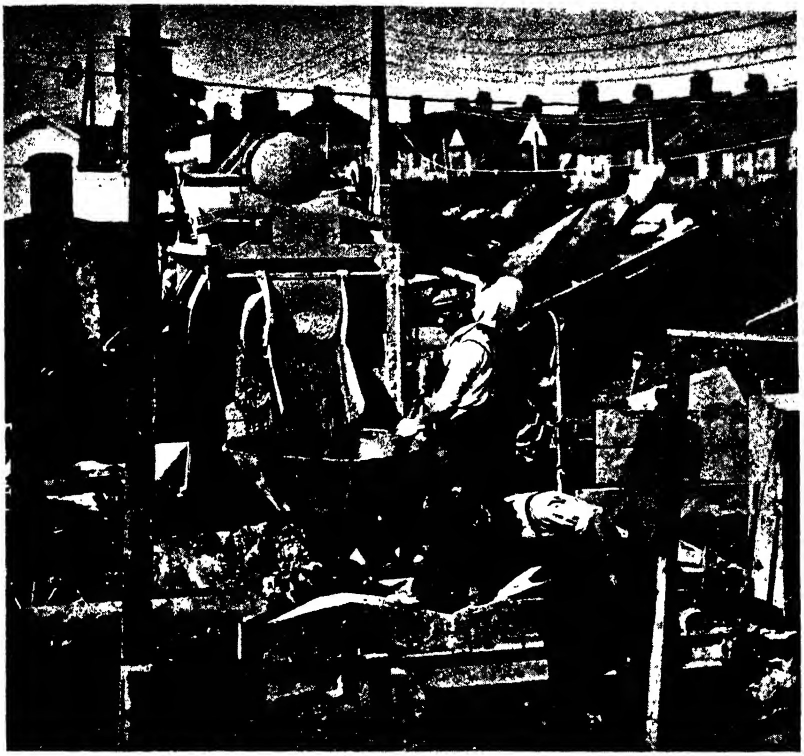


Fig. 160.—Feeding Concrete Pump.

Fig. 161.—Discharging Pumped Concrete.

aggregate; but a 5-in. pipe is suitable when the length of piping does not exceed 300 ft. and the aggregate does not exceed $1\frac{1}{2}$ in. For concretes with aggregates up to 3-in. maximum gauge, 7-in. internal diameter pipes should be used. At the close of a concreting period it is necessary completely to empty and clean out the pipe-line by compressed air.

The delivery range of pumping plants is 125 ft. vertically or 750 ft. horizontally, or combined vertical and horizontal transmissions within these limits.

The charges connected with the installation and operation of a pumping outfit are offset by saving in plant and labour due to the elimination of hoists, wheelbarrows, or chutes. The actual cost of distribution by pumping depends upon the general conditions of the job, the amount of concrete, and the cost of power. On contracts in London the cost, inclusive of all labour and running costs connected with the mixer and an electrically-operated pump, has been reduced to about 1*s*. 5*d*. a cubic yard with a large-capacity pump and 2*s*. per cubic yard with the smaller pump. Mixing and placing concrete by pumping in foundation work of about 1,000 cu. yd. may cost 2*s*. 3*d*. to 2*s*. 6*d*. per cubic yard.

The power required to operate a pump at full range and capacity is up to 35 h.p. An example of power costs in London is a halfpenny per cubic yard when placing an average of 22 cu. yd. an hour with a maximum lift of 100 ft. The labour required for this output was eight men who dealt with the cement and aggregate supplies to the mixer, operation of the latter and the pump, and tamping the concrete into position.

Further consideration is given to the overall cost of pumping installations in the succeeding section.

Choice of Distributing Plant.

The circumstances appropriate to each particular job will determine the most suitable character of the distributing plant, but the following remarks on selection of plant can be considered as a guide. In the first place consideration has to be given to the size of the job and what plant is available. It may be necessary to decide whether the job is sufficiently large to warrant buying new plant or hiring an elaborate distributing installation. This will involve estimating the rate of concreting and preparing comparative cost estimates of methods of distribution to maintain this rate. In considering manual labour the possibility of accommodating the required number of men in the available working space must be foreseen.

Such comparatively small self-contained structures as water towers and coal bunkers favour the use of a power hoist of the skip or platform type. As the concrete is not usually placed in large volumes at a time, distribution from the head of the hoist by means of wheel-barrows is most convenient. For tall buildings, multi-cell silos, or similar structures, if one or two hoists can be centrally placed so that the wheeling distance does not exceed, say, 20 yd., a similar choice of hoists feeding into carts or barrows is available. If the area of the building is larger than can be conveniently covered by wheeling, one or more chuting plants could be installed. With large areas of floors and large volumes of concrete, distribution by pump offers a suitable alternative to chuting plants within the limits of vertical and horizontal travel previously given.

In silos and similar structures where the ratio of shuttering to concrete is high, a concrete pump could only be used with advantage if continuously-moving shuttering is employed, since for economical operation of the plant associated with a 10-cu. yd. pump, a daily output of 80 to 100 cu. yd. must be maintained. This would entail the erection and dismantling of 800 to 1,000 sq. yd. of normal type shuttering each day, which might represent £80 to £100 per day in carpenters' wages, that is 40 or 50 carpenters would be continuously employed.

Owing to the absence of high lifts and due to the relatively small quantity of concrete in relation to the area of the work for reservoirs, tanks at ground level and swimming baths, barrows, carts or jubilee skips, hand-operated on small jobs and power-driven on large jobs, are suitable. If the area is particularly large, as in paving large reservoirs, chuting plants are undoubtedly economical.

Bridge construction favours the use of cranes for lifting and transporting concrete, and examples have already been given in the plant arrangement diagrams given in Chapter I.

Pile-casting yards can be economically operated with either transporter or mobile chuting plants.

Tall chimneys present special problems that are dealt with in a later chapter. For chimneys of moderate diameter, winch-operated block-and-tackle hoists operating within the cylindrical shell to elevate the concrete in small quantities to the working level are most suitable. The top pulley may be carried on an extension of the internal scaffolding or on a centrally-erected steel mast guyed to suitable points at ground level. This method is applicable to water towers having central stair-wells, elevated tanks having centrally-placed outlets, towers, and domes with lantern lights. For large-diameter chimneys or similar structures of some height and where internal dimensions permit, as in ventilation shafts and cooling towers, an internal skip-hoist would be most suitable.

In brick, masonry, or steel-frame buildings, where the concrete construction forms only a subsidiary part of the contract, special plant is seldom warranted. Concrete at ground level can be distributed by barrows directly from the mixer, and, for roof slabs and other superstructure work, barrows or concrete carts can be elevated by a chain sling working from a pulley block, or by whatever method of hoisting is adopted for other materials. In the case of steel-frame structures a crane is usually installed to deal with the steel members, and if this crane has not been dismantled before concreting operations commence it can sometimes be arranged to use it for hoisting concrete.

For such major contracts as dams or long viaducts, cableways or other specially designed distributing methods are devised. The same remark applies to tunnel work.

Chuting plants are generally most economical if 125 to 200 cu. yd. have to be placed daily. If the total quantity of concrete in a job is less than 1,000 cu. yd. it is usually cheaper to employ the minimum amount of plant and therefore the maximum amount of labour. With upwards of 2,000 cu. yd. the maximum amount of plant and minimum labour result in general in the lowest cost. Between 1,000 and 2,000 cu. yd. the most economical method depends on the plant available.

In any given case selection is often governed by the equipment the contractor possesses or can readily hire. When a free choice is presented the problem becomes one of balancing saving in labour costs against increased plant costs, not overlooking the consequent saving or loss in time. At one extreme, maximum labour and minimum plant would be represented by unloading materials and mixing concrete by hand, associated with distribution in wheelbarrows or in buckets elevated by a hand-operated pulley-block and tackle. This would only pay in exceptional cases involving a very small quantity. At the other extreme of minimum labour and maximum plant, an extensive mechanical unloading, batching, mixing and distributing equipment would be involved, and when several thousand cubic yards have to be placed economy results. Intermediate cases of contracts involving a few hundred cubic yards present the less decisive case and are more fully controlled by available resources, experience, and wage rates. In such indecisive instances estimated costs of one or two schemes should be compared. In preparing such estimates the following items should be considered :

LABOUR : Unloading materials and transferring to stock and/or to mixer, including attendance on any handling equipment. Attendance on mixer, hoist, etc. Distributing concrete to shuttering. Unloading, erecting, maintaining, dismantling, reloading, etc., all items of equipment. On-costs on net labour including supervision, time-keeping, insurance, small tools, head office and general charges and any other costs chargeable to labour, such as provision of transport, living quarters, etc.

PLANT : Depreciation (or hire charge), transport charges to and from the site, and cost of fuel and consumable stores for all equipment installed for unloading and handling materials, and for mixing, hoisting and distributing the concrete. On-costs on plant including head-office and general costs.

For any given installation some of these items will be constant irrespective of the size of the job. Others, such as unloading materials, will be proportional to the quantity of concrete involved, while charges such as depreciation and hire of equipment depend on the time occupied on the site. The following examples of estimated cost apply to two schemes, one involving a reasonable minimum amount of plant, and the other minimum labour.

EXAMPLE (i). Consider the case of contracts involving up to 1,000 cu. yd. of concrete for the execution of which the only plant used includes a 10/7 mixer, power winch, a site-made hoist tower, wheelbarrows, batch boxes, ropes, shovels and other small tools. The sequence of operations would be unloading cement by hand and tipping aggregates from lorries ; measuring aggregates ; loading skip of mixer with cement and aggregates ; mixing and discharging into skip of hoist ; hoisting and discharging into wheelbarrows ; wheeling to placing position ; discharging on to banker or direct into shuttering ; tamping.

The costs can be conveniently divided into three classes : (*a*) Constant charges irrespective of volume of concrete or contract time. (*b*) Charges proportional to quantity of concrete. (*c*) Charges proportional to time.

If the average rate of concreting remains constant irrespective of the quantity, then the time would be proportional to the volume of concrete and classes (*b*) and (*c*) could be combined. In practice, however, the average output is usually lower for small volumes and increases with larger volumes.

The constant charges (*a*) include (i) construction of batch boxes for fine and coarse aggregate (cement will be measured per cwt. bag) ; cost of keeping boxes in repair and transport charges at end of job will offset any salvage value. (ii) Carriage to and from the site of mixer, winch, materials for constructing hoist tower, etc. (iii) Erection and dismantling of mixer, winch, hoist-tower, etc.

The charges (*b*) proportional to the quantity of concrete will be (i) unloading and storing cement ; (ii) attending on aggregate lorries, trimming aggregate heaps, etc. ; (iii) measuring aggregates and loading mixer skip with cement and aggregates ; (iv) fuel and grease for mixer and hoist-winch ; (v) operation of skip at head of hoist and loading into wheelbarrows ; (vi) wheeling and depositing concrete ; (vii) tamping concrete in shuttering.

The charges (*c*) that will be directly proportional to the contract time include : (i) wages of man in charge of the mixer and winch ; (ii) foreman, timekeeper, etc. ; (iii) depreciation or hire charge of mixer and winch.

At full capacity the 10/7 mixer would produce, say, 12 batches of 6 cu. ft. of wet concrete per hour, that is $2\frac{2}{3}$ cu. yd. per hour. The average output throughout the period of mixing and placing several hundred cubic yards would not probably exceed 12 cu. yd. per eight-hour day. When working at full capacity the weight of aggregate and cement that has to be measured and fed into the skip of the mixer is about $4\frac{1}{2}$ tons. This would require four men who at other times would be available for unloading cement and attending to material lorries. At full capacity, wet concrete will arrive at the head of the hoist at the rate of 72 cu. ft. per hour, which is about 36 wheelbarrow loads. If each wheeler can do 10 journeys per hour, three or four wheelers will be required together with one man at the head of the hoist discharging concrete into the barrows. Two or three men will be engaged in dealing with the concrete when it is discharged from the barrows. When the maximum output is not being maintained, some of these seven or eight men at the head of the hoist would be employed on cleaning out the shuttering, arranging barrow runs, making adjustments to the hoist tower, attending to curing operations and similar work. Similarly the man in charge of the mixer, during non-concreting periods, would be overhauling the engine, examining winch ropes, etc., greasing, and other mechanical operations.

If the contract involves 500 cu. yd., taking 42 working days (8 weeks) to place, the approximate cost would be made up as follows :

		£
4 men serving mixer.		
8 men distributing, tamping, etc.		
12 men at 1*s*. 1*d*. an hour for 8 hours a day for 8 weeks	=	229
1 man in charge of mixer at 1*s*. 6*d*. an hour for 8 hours a day for 8 weeks	=	27
Net labour	=	256
Add 8 per cent. for insurance, small tools, etc.	=	20
One-third of foreman's wages at £6 per week	=	16
Allow for 50 per cent. of timekeeper's wages	=	8
Total labour	=	£300

	£
Fuel and grease for mixer and hoist : 500 cu. yd. at 6*d*. a cu. yd. .	13
Depreciation on £180 mixer at 25 per cent. per annum for, say, 3 months	12
Constant charges : Carriage of mixer and materials for hoist tower to and from site	15
Erect and dismantle mixer, winch, hoist tower, etc., making batch boxes, etc.	10
Total cost	£350

That is, for 500 cu. yd. the cost is $\frac{£350}{500} = 14s.$ per cubic yard.

If a similar estimate is prepared for 300 cu. yd. the cost would be 17*s.* 9*d.* per cubic yard, and for 1,000 cu. yd. it would be 13*s.* 4*d.* per cubic yard, assuming an average daily output of 10 cu. yd. in the first instance and 15 cu. yd. in the latter.

Example (ii).—Now consider a contract involving upwards of 1,000 cu. yd. of concrete in which the maximum amount of labour-saving plant is used. The equipment would include a boom-scraper feeding an elevator which in turn loads

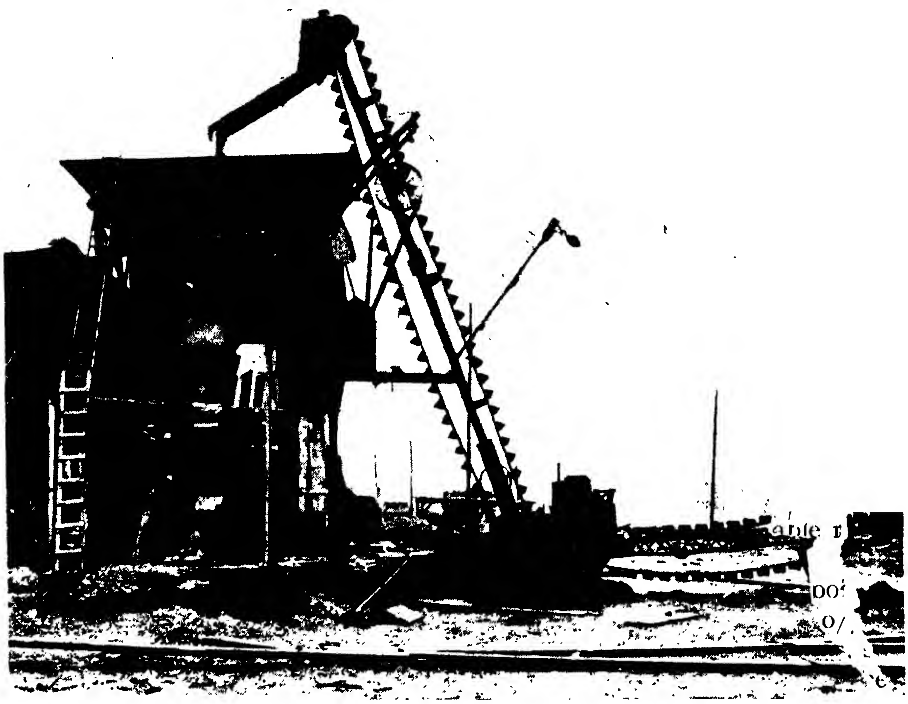


Fig. 162.—Mechanical Handling of Materials.

the aggregates into a bin provided with an automatic batch weigher. The batcher feeds directly into a 14/10 mixer, the equipment being arranged as in *Fig.* 162. It will be assumed that the concrete is distributed by pumping through 300 ft. of piping with an average vertical travel of 30 ft. At full capacity with two-minute batches of 9 cu. ft. each, the mixer would produce 10 cu. yd. an hour and since a pump can deal with this quantity, the average output, for estimating purposes, throughout the period of mixing and placing over 1,000 cu. yd. of concrete will be taken as 60 cu. yd. per eight-hour day, thus allowing for moving pipe-lines, unforeseen delays, etc. The sequence of operations would be unloading cement by hand and tipping the aggregate from lorries, adjusting the position of the boom-scraper, operation of scraper and elevator power units, operation of batching plant, mixer and pump, adjusting pipe-lines, tamping and other operations at the discharge end of the pipe-line. The labour force required would be one man attending to the boom-scraper and waiting on the mechanic, three men unloading and opening bags of cement, one mechanic, one man operating batcher, mixer, and pump, and three men tamping, adjusting pipe-lines, etc.

Thus the cost would be:

		s.	d.
8 men at 1*s.* 1*d.* an hour	=	8	8
1 mechanic at 1*s.* 8*d.* an hour	=	1	8
		10	4 per hour.

		£	
8 hours per day for one week at 10*s.* 4*d.*	=	23	per week.
Add 8 per cent. for insurance, small tools, etc.		2	,,
One-third of foreman's wages at £6 per week and part of time-keeper's wages		3	,,
		£28	,,

If the contract involves 2,000 cu. yd., the time occupied would be six weeks. Wages would be calculated on this period, but plant hire or depreciation would be based on a longer period, say 3 months, to allow for time in delivery, erecting, and dismantling. The constant plant charges would be:

Carriage of scraper, elevator, bin and batcher, mixer, pump, pipe-lines, etc., to and from the site	say	£25
Erecting and dismantling	say	£15
		£40

The approximate value of a large concrete pump with 300 ft. of piping is £1,000 and of a small pump with a similar length of piping £800 approximately. Such equipment may be hired for about £30 a month plus a charge of, say, 9*d.* a cubic yard placed. These rates only apply to horizontal work with vertical travel not exceeding 30 ft., an increase in the latter being accompanied by an increase in the yardage rate. The value of the batching plant and mixer might be £150 for a boom scraper, £210 for an elevator complete with motor, £295 for a steel material bin provided with an automatic weigh batcher and £225 for a 14/10 mixer, the total being £880, or a hire charge of about £80 per month.

Thus the total cost for 2,000 cu. yd. of concrete would be made up as follows:

		£
Labour: 6 weeks at £28	=	168
Carriage and erection of plant	=	40
Fuel and grease: 2,000 cu. yd. @ 9*d.*	=	75
Hire charge for bin, mixer, etc.: 3 months @ £80 per month	=	240
Hire charge for pumping equipment: 3 months @ £30 per month	=	90
Yardage charge for pump: 2,000 cu. yd. @ 1*s.*	=	100
Total	=	£713

That is $\frac{£713}{2,000} = 7s.\ 2d.$ per cubic yard.

If similar estimates are prepared for 1,000 cu. yd. and 5,000 cu. yd. on contract periods of 3 weeks and 2 months in the first case and 16 weeks and 5 months in second, the cost would be 8*s.* 8*d.* and 5*s.* 11*d.* per cubic yard respectively.

CHAPTER VI

CONCRETING

SECTION I.—PLACING CONCRETE.

General Principles of Placing.

THE quality of concrete depends, among other things, on the thoroughness with which the operations preliminary to placing are carried out. The shuttering should be inspected and corrected if warping or other defects have occurred. All shuttering should be washed down with a hose and the faces of existing concrete work should be treated as explained later for construction joints. In frosty weather the watering should not be done if it is likely to freeze. Sawdust, chips, nails, and other debris should be removed from the shuttering. The most convenient method is to wash out the shuttering through an opening left for this purpose, or a 2-in. square opening may be cut out of the sides of beam, wall, or column shuttering through which the water can escape and carry with it any light and loose debris. The cut-out piece can be replaced in the manner illustrated in *Fig.* 163 before concreting commences. Larger pieces of wood and other materials can be lifted out or " speared " with a pointed rod.

Reinforcement should be finally inspected, and special attention given to the cover provided. Concrete distance-pieces should be inserted between the bars and the shuttering where necessary as described in Chapter III. Wood should not be used for these distance-pieces as serious faults may develop if the wood is left in position, and since it is necessary to remove the timber blocks before the level of the concrete reaches the position of the block this may result in detrimental movement of the bars. Moreover, pieces of wood dropped into the concrete may not be recovered.

Except for slabs and large work, such as heavy foundations, the concrete should be placed into the shutters by shovels, and care should be taken that all the more liquid parts of the batch of wet concrete are taken up as well as the more easily handled stiffer portions. Where practicable the concrete should be placed directly into its final position and the practice of depositing the concrete in one place and working it along the shuttering should be discouraged.

Other than in slabs and beams where the complete thickness should be placed in a single layer, concrete should be placed in successive layers. If the concrete is very dry the layer should not exceed 6 in. thick, but the depth of each layer may be up to 3 ft. with increasing plasticity. The layers should be either parallel or perpendicular to the direction of the principal compressive stresses. Thus the layers would be horizontal in the case of columns and walls, and deep beams.

While being deposited in the shutters the concrete should be well tamped, rodded, rammed, or spaded. This operation is important if a dense concrete is to be obtained, and particular care should be paid to the ramming in the space between the bars and the shutters, as the denseness of the skin formed here

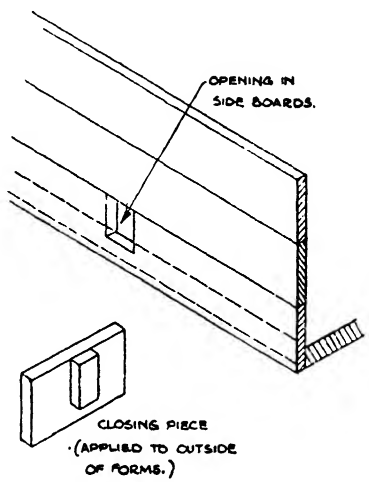


FIG. 163.—OPENING FOR CLEANING OUT SHUTTERING.

largely determines the amount of protection from corrosion afforded to the reinforcement. For ordinary beam, slab, and column construction, one man tamping to each man depositing concrete should be sufficient, but for work with thin walls, especially in tanks, bunkers, and all impermeable structures, two tampers to each placer is a good rule. The tamping should be carried out so that the concrete is worked free of air bubbles and honeycombing and so that it is well worked around the steel. For dry concrete in positions where reinforcement is very congested, as in the bottom of heavy beams, the bars should be covered with cement grout or a fairly plastic mix before placing the drier concrete in the body of the work.

Superficial "honeycombing" is due either to inefficient tamping against the face of the shuttering when a fairly dry concrete is used or to leakage of cement grout when a wet mix is employed. Defects due to the latter cause can be avoided by making the joints between the boards tight by more careful construction with planed boards, by caulking, by using tongue-and-groove boarding, or by lining the shuttering (as explained in Chapter IV) with paper or similar material. Linings, such as paper, should be kept free from creases or irregularities that may cause indentations in the finished concrete surface.

While tamping is a necessity, excessive tamping leads to a non-homogeneous concrete in which the larger aggregate with insufficient mortar tends to be driven to the centre and bottom, while a rich mixture is localised at the sides and top. The liability to overtamp is greater with wet than dry mixes. Very dry mixes need to be well compacted with a 6-in. square hand rammer or a pneumatic hammer, special care being taken at the corners and edges of the shuttering. The ramming should not be too heavy otherwise there is a tendency to " spring " the sides of the shuttering. The consolidation of moderately dry (or plastic) concretes can be assisted by tapping the outside of the shuttering. The consolidation of concrete by controlled vibration is dealt with later. Normally, ramming or tamping is effected by a steel bar, a piece of timber, or one or other of the special tools made for this purpose. A wood lath, the least dimension of which is slightly less than the minimum cover to the bars, is as good as anything and has the advantage that its distinctiveness compared with the reinforcement may prevent it being inadvertently left in the concrete.

It is necessary that the time between mixing and placing should be reduced to a minimum, otherwise the initial setting process may be interfered with. The concrete should generally be placed within twenty minutes of mixing, and should not be disturbed after one hour from the time when water was added to the cement. This period may be modified when special measures are taken as in some methods of distribution of ready-mixed concrete.

At the completion of placing a layer of concrete, any laitance appearing on the surface should be skimmed off before the concrete hardens, and should be removed in such a manner that the concrete is not robbed of cement. Neglect to remove laitance leads to defective surfaces and weak joints. Projecting reinforcement that is to be bonded into subsequent work should be reinspected during concreting, and any adjustments necessary should be made before the initial set ; it is worse than useless afterwards, as the movement of a bar leaves a cavity that cannot be adequately grouted up and destroys the bond between the bar and the concrete.

In succeeding pages the essentials requiring attention in placing concrete in the structural members of buildings, bridges, and other structures are described.

In sloping members, such as diagonal braces, hopper bottoms, inclined roof beams, etc., with shuttering on all faces, concrete should be placed in small quantities and well rammed to prevent air pockets being trapped at the bottom. The same precautions should be applied to splayed footings that are shuttered on the top as well as the sides. Where possible, foundations and footings should be concreted to their full depth without joints in one operation. Excavations should be cleared of water before depositing concrete in foundations, unless special arrangements are made as described later to allow for the concrete being placed under water. Precast members such as piles should be completed in a single operation.

In steps, the concrete should be as dry as possible to prevent " welling-up " of wet concrete in the treads of lower steps due to the head of concrete in subsequent steps. Usually not more than three or four steps can be cast in situ in one operation unless the treads of the lower steps are boarded over to prevent welling-up. If boarding is provided, difficulty arises in ensuring that the concrete penetrates well into the nosings. Reinforcement in stairs is liable to considerable distortion during concreting operations and care should be taken to ensure that the bars are in the correct position as concreting proceeds.

The principal objectives when placing concrete below the surface of water are the prevention of the loss of cement and the prevention of excessive saturation of the concrete. Precast reinforced concrete units such as piles and braces are frequently used in wharf and jetty construction to prevent the difficulties otherwise occurring in satisfactorily placing concrete in situ under water. In tidal water the pre-cast work usually extends up to a convenient level above low water where cast-in-situ work can be adopted. For the stage of the latter between tides it is necessary to construct the shuttering so that members are fully encased, in order that water at high tide shall not wash out the cement or otherwise disturb the concrete while it is hardening, and to anchor down the shuttering on which the tide rises before the boxing is filled.

Concrete breakwaters and similar massive structures in deep water are often constructed of mass concrete blocks, which are pre-cast on land or on stagings and lowered into position with cranes, usually under the guidance of divers.

The usual methods of placing concrete in situ under water are not generally adaptable to reinforced construction, and when pre-cast work cannot be employed cofferdams which can be pumped out are constructed around the work to enable the reinforcement and concrete to be placed in the dry. This method is only practicable with water of moderate depth and is much used for the erection of river bridge piers. For deeper water, compressed-air caissons are more adaptable for providing a dry working space for excavation and foundation construction, or pre-cast sectional concrete cylinders are sunk to the required depth by weighting and under-water excavation is performed by grabs operating within the cylinder. The cylinder is then filled with concrete, either after being pumped dry or while containing water.

The most common methods of placing concrete in situ under water are by means of a tremie, or by bottom-opening skips, or by bagged concrete. The last mentioned method, which is very suitable for coast-defence work constructed between tides, consists of bags (usually cement sacks) partly filled with fairly dry concrete. The bags are simply laid in position to form the wall or other structural unit, and " header and stretcher " construction is adopted to ensure bonding between the bags.

The tremie consists of a pipe from 6 in. up to 12 in. in diameter extending from a hopper head to the bed of the work as indicated diagrammatically in *Fig.* 164. The lower end of the pipe is usually open, or it may be fitted with a valve. The pipe should be clear of water before it is filled with concrete, or if it is open-ended a temporary plug of sacks can be inserted which will be forced out by the head of concrete when the pipe is filled. The pipe should be maintained filled throughout the operation of placing, and the end of the pipe should be kept immersed in the deposited concrete.

Steel skips of various designs operating in conjunction with a crane are frequently used in place of a tremie. One successful type consists of a square box capable of holding 6 cu. ft. of concrete and fitted with doors in the bottom that are kept closed by chains connected to the point of suspension. The box is lowered until its downwardly projecting sides touch bottom, the doors are released, and while they are gradually opening the concrete falls through not more than 18 in. of water on to the bottom or on to concrete laid previously. An ordinary bottom-opening circular or square skip can also be used for this purpose, but the skip should rest on the bottom before the flap is released. The operation of lowering and raising the skip of any design through the water should be carried out as slowly as possible in order to prevent eddying that may affect the concrete already placed.

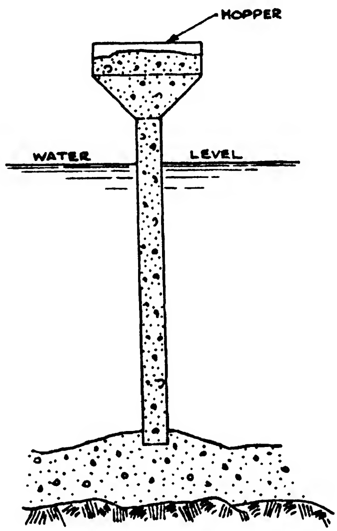


FIG. 164.—THE PRINCIPLE OF THE TREMIE FOR PLACING CONCRETE UNDER WATER.

Concreting under water by tremie or skip should proceed as rapidly and continuously as possible to prevent the formation of layers, and hollow places in the concrete surface should be avoided during placing as dirt and other detrimental material may be deposited producing weak strata or pockets in the concrete. To counteract the effect of too much water a fairly dry mix should be used; the cement content should be increased. For under-water work a mix leaner than 7 cu. ft. of cement to each cubic yard of concrete should not be used. If the temperature of the water falls below 35 deg. F. concreting should cease, and if the temperature is permanently below this some form of pre-cast work should be adopted.

Consolidation by Vibration.

Portable vibrators operated by compressed air or electric motors are also used for consolidating concrete, and are either attached to the shuttering or immersed in the concrete. *Fig.* 165 shows three electric vibrators attached to the soldiers of a panel of shuttering for a wall; a vibrator attached to channel-iron soldiers is shown in *Fig.* 166. *Fig.* 167 illustrates a vibrator attached to a piece of wood for use on the shuttering of a floor slab.

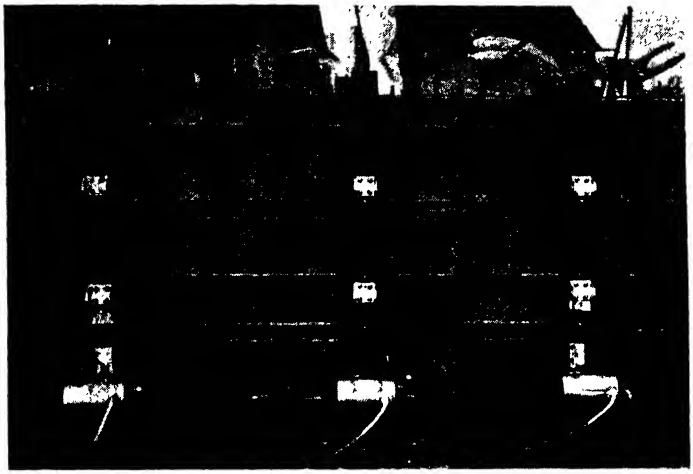


FIG. 165.—ELECTRIC VIBRATORS ATTACHED TO WALL SHUTTERING.

Where the sections of concrete members are thin and very heavily reinforced, carefully executed vibration, either alone or combined with tamping, causes the concrete to flow uniformly around the bars. Moreover, the increased fluidity due to vibration occurs even with mixes which would otherwise be re-

FIG. 166.—VIBRATORS FIXED ON SHUTTERING.

garded as too dry for reinforced concrete. In consequence, in work where a low cement-water-ratio would be necessary with hand tamping, higher strengths may be obtained by vibration due to the drier concrete that can be used and consolidated with these machines. There is sufficient evidence that the increased strength gained by vibrating the concrete is useful in increasing its resistance to the somewhat indeterminate stresses in pre-cast piles hammered into the ground.

In work consisting of slabs, singly-reinforced beams and columns, the use of vibrators saves time and labour in placing concrete but does not always result in a saving in cost due to possible extra shuttering costs, higher mixing costs if a dry concrete is used, the cost of purchasing or hiring the machines, the cost of power, and labour fixing the vibrators to the shuttering. Good surface finishes are also produced, and side shutters of beams can be stripped in eight hours to enable the board marks to be removed before the concrete becomes really hard.

Fig. 167.—Vibrating Slabs.

A slump not exceeding 2 in. is recommended if this result is to be secured, but much drier concrete can be consolidated by vibration.

With doubly-reinforced beams the results are not so satisfactory unless the horizontal intervals between the bars are more than twice the bar diameter. With closer spacing of the bars vibration is not sufficient to force the concrete through the layers of steel, and should be combined with, or replaced by, hand tamping.

In the construction of walls unsatisfactory results are caused if the fixtures holding the steel shuttering, or the nails or bolts in the timber, work loose. With timber shuttering screws are preferred to nails when the concrete is to be vibrated, and additional strutting is required to make the shuttering rigid. Tongue-and-groove sheeting $1\frac{1}{2}$ in. nominal thickness with 4-in. by 2-in. battens at 2-ft. centres is recommended for shuttering wall panels, and the horizontal walings should be bolted at intervals not exceeding 2 ft. Great care is required to make the panels true and to obtain good alignment, particularly when the panel is raised to deposit concrete in a later lift. If the joint between the shuttering and the hardened concrete at the bottom of the new lift is not tight, there is danger of an unsightly honeycombed construction joint being produced at this level. When the supervision is good, lifts of more than 3 ft. can be placed satisfactorily in walls if vibrators are used.

A satisfactory arrangement of the vibrators is obtained by placing them in a horizontal line just above the top of the previous lift. Each vibrator is brought into action as soon as there is sufficient concrete near it to absorb the vibrations. As the concrete rises in the shuttering the vibrators are raised.

Owing to the greater consolidation obtained, up to 10 per cent. more material may be required for vibrated concrete as compared with hand-tamped concrete. Care must be taken to avoid continuing the vibration too long, as this causes separation of the fine and coarse aggregates. Vibration should be stopped when the water comes to the top of the concrete. It may be mentioned that longer periods of vibration are required to consolidate concrete made with angular aggregate than in the case of rounded aggregate. In unskilled hands vibration is more dangerous than hand tamping. It is essential that the vibrators be attached firmly to the shuttering, otherwise much of the vibrating action will be lost and, in the case of electrical machines, overheating and increased power consumption may result.

In the construction of walls vibration makes it possible to strip shuttering in two hours in good weather, thus increasing the speed at which the work is carried on and counterbalancing some of the extra costs due to the use of more rigid shuttering and the greater volume of concrete required.

The cost of electric and pneumatic vibrators is controlled by the cost of running a compressor plant compared with the cost of electricity. The same efficiency of vibration is given by a 3-h.p. compressor and an electrical vibrator consuming 150 watts. On actual jobs other factors may control, such as the remoteness of the site from the electric mains or availability of free electricity, or of a cheap air supply as is sometimes the case in colliery and industrial contracts.

In internal vibration, a vibrating rod is plunged into the concrete at intervals, although in some instances, where the reinforcement permits, the vibrator can be inserted in the concrete and worked along the length of the member. The vibration effect may be felt effectively over a radius of 3 ft., hence the vibrators should be inserted at 2 ft. to 2 ft. 6 in. centres. Internal vibrators should be kept clear of the inner face of the shuttering, as the abrasion on the timber may result in blemishes on the face of the work.

If vibration is specified for floor slabs, roads, or other slabs, a machine such as that shown in *Fig.* 168 is used. This machine consolidates the concrete by vibrations produced by one or more vibrators mounted on a steel plate. An alternative method applied to floor slab construction is illustrated in *Fig.* 167, where a vibrator is attached to a 4-in. by 2-in. timber batten, the foot of the latter being applied to the centering. Usually suspended floor slabs of moderate area can be more effectively vibrated by attaching external vibrators to the underside of the shuttering.

With vibration it is essential to strengthen the shuttering considerably more than for normal placing. The denser and more liquid concretes (after vibration) will subject horizontal shuttering to higher loads, and vertical shuttering to greater hydrostatic pressures. A weight of not less than 155 lb. per cubic foot of concrete should be taken when designing shuttering for vibrated concrete.

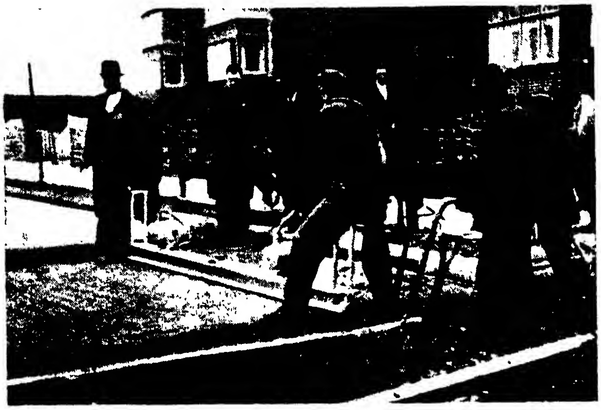


FIG. 168.—PLATE VIBRATOR FOR SLABS.

With external vibration extra strength and especially extra rigidity at connections are required. The strengthened shuttering may require 25 per cent. more stiffeners, that is battens and walings. Tight joints between the boards are essential to prevent leakage due to springing; tongue-and-groove boarding is preferable. The required increase in strength of the shuttering may lead to an additional cost of 25 to 30 per cent. of the cost of normal shuttering.

Construction Joints.

Construction joints may be as strong as unjointed concrete but they may be sources of weakness unless they are carefully made, since lack of adhesion prevents the concrete from withstanding the tensile stresses induced by shrinkage and other causes. A badly-formed joint may allow water to enter and subsequent freezing may lead to disintegration.

The first step in preparing a joint is to remove any laitance, scum, or porous concrete by hacking. If care has been taken, as already described, to remove these deposits immediately after placing, there should be no trouble in hacking away the few portions that may have been passed over. Hacking should be continued until the old face is entirely removed, and the exposed concrete should be vigorously scrubbed with a wire brush to remove all loose mortar and stone. The exposed surface of the old concrete should be thoroughly wetted and, to produce the best results, given a thin coat of neat cement grout and an application of 1 : 2 cement-sand mortar immediately before placing the new concrete. The grout and mortar application should be kept as thin as practicable, otherwise the line of the joint will be emphasised on the finished surface.

Before commencing to place the new concrete the shuttering already erected should be tightened and re-adjusted where necessary to prevent leakage between the face of the timber and the old concrete. The portion of the new concrete in contact with the prepared face of the old concrete should be plastic, that is, not too dry to enter into the interstices of the old concrete nor too wet to produce weaknesses due to excessive shrinkage.

It appears that there is little harm done in using a type of cement in the new concrete different from that in the matured concrete, but generally an improvement of bond is obtained if the matured concrete is made with normal-hardening Portland cement. The difference in bond strength of the joint is more pronounced if rapid-hardening or high-alumina cement is used. High-alumina cement concrete bonds equally well with matured high-alumina cement concrete or with matured rapid-hardening Portland cement concrete, but the bond between rapid-hardening Portland cement concrete and matured high-alumina cement concrete is considerably less than the bond between two specimens of matured rapid-hardening Portland cement concrete. High-alumina cement concrete should be at least 24 hours old before joining up with new Portland cement concrete ; before joining up new high-alumina cement concrete to normal Portland cement concrete, the latter should be seven days old and rapid-hardening Portland cement should be two days old. When both old and new concretes contain the same type of cement there is no restriction on the age of the old concrete before the joint is made, although the less time that elapses between the two pourings the better the joint is likely to be. If the old concrete is only a few hours old when the joint is made, care must be taken when hacking to avoid loosening the stones in the matrix ; rubbing vigorously with a wire brush may be sufficient treatment when the laitance has been removed. In the case of such immature concrete, also, it is sometimes sufficient to apply cement grout only to the thoroughly wetted prepared surface, although some regulations insist on the application of a 1 : 1 or a 1 : 2 mortar.

In the case of a long joint in a slab or wall, the grout should not be placed more than a few minutes in advance of the mortar or new concrete, otherwise it may dry out and produce an undesirable scaly layer between the two faces.

A 4 : 1 solution of muriatic (or hydrochloric) acid and water, or a retarding liquid, is sometimes used in place of hammering to remove the cement on the face and expose the aggregate at a construction joint ; if this treatment is adopted all traces of acid should be removed by washing before applying the grout. With retarding liquids it is necessary to use wire brushes on the surface as soon as the shuttering is struck.

As regards the profile of construction joints, a keyed joint is preferable. Inclined " feather " joints in slabs, as shown in *Fig.* 169, should be prohibited except where concreting will be continued in a few minutes. Recommended forms of keyed joints for slabs and walls are given in *Figs.* 169 and 170. As indicated, the stop-board for slabs can be slotted to allow the bars to pass through ; the slotting provides a convenient way of spacing and maintaining the spacing of the bars. In order that one slotted board may be used several times, the spacing of the bars should be constant throughout the panels affected. An example of slotted construction joint boards and the method of retaining the boards in position is illustrated on *Fig.* 171. If a fairly dry concrete is used it is not essential to slot the board as it can merely rest on the bars and the space below may be small enough to restrict the flow of concrete. Any concrete that flows through must be cleaned off the shuttering and the face of the joint before continuing the work.

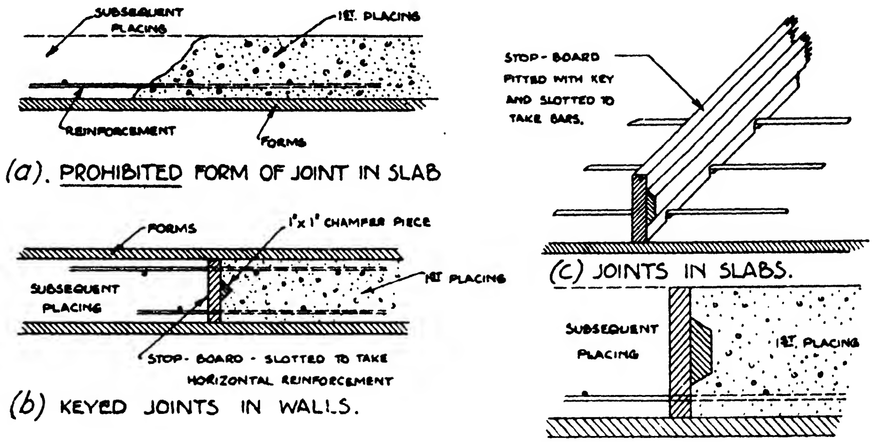


FIG. 169.—CONSTRUCTION JOINTS.

FIG. 170.—KEYED JOINTS IN SLABS.

FIG. 171.—SLOTTED STOP-BOARD.

The number of day's work, or construction, joints should be reduced to a minimum and the positions should be predetermined after careful consideration. The amount of work to be done each day should be decided and this should end at a correct position for a joint. The stoppage of work for meals should also be made at an approved position.

If the position of the construction joints is not specified the following should be adopted. Primarily the joint should be placed at a section where the shear is a minimum, and the face of the joint should be normal to the compressive forces in the member. Thus for beams the joint should be in the centre of the span, or at any rate within the middle third; if this is impossible, it should be made at the centre of the support. The face of the joint should be vertical or, in the case of sloping beams, perpendicular to the length of the beam. If the joint in a slab is made normal to the main reinforcement it should be placed at the centre of the span, but if it is parallel to the main steel it may be placed anywhere

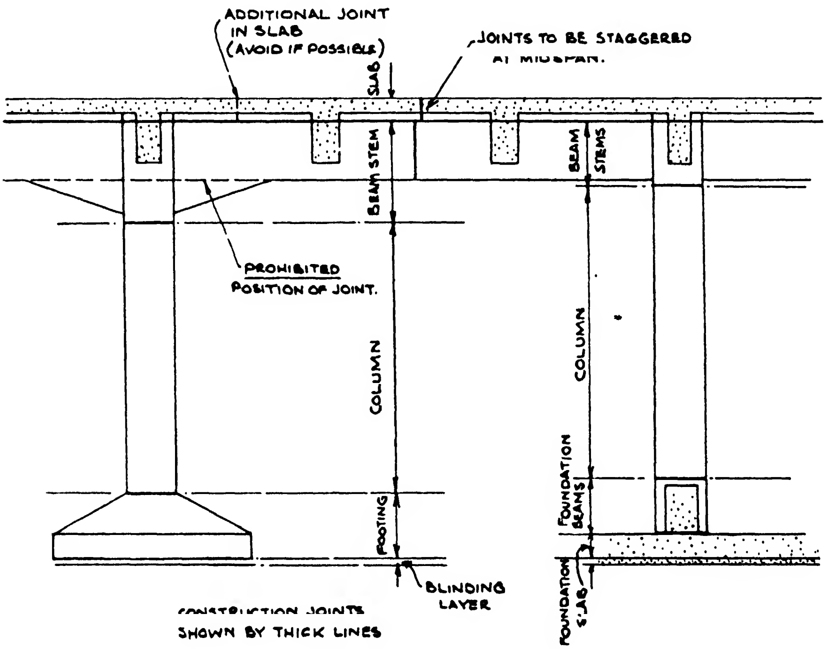


FIG. 172.—POSITIONS OF CONSTRUCTION JOINTS.

so long as it is not less than eight times the slab thickness from the edge of a parallel beam designed as a tee-beam. When concreting columns the joint should be horizontal and placed a few inches below the beam intersection. In frame-construction the joints should be at the points of contraflexure or alternatively at sections of minimum shear. The engineer should always indicate in such structures the allowable positions for joints, but if this is not done the contractor should submit for the engineer's approval a sketch indicating the positions where he proposes to make the joints. Construction joints in frames and in arches receive further consideration in subsequent chapters, together with the position of joints in impermeable construction. *Fig.* 172 shows approved positions of joints in ordinary column and beam construction.

Slabs with permanent joints, for example, long corridor slabs, ground slabs on the solid, and roads, should be free from construction joints, each concreting operation terminating at a permanent joint. If a panel bounded by permanent joints cannot be completed in the time available it is usual to make a permanent joint at the section at which work is stopped. Dry joints between panels of non-suspended slabs are more effective if coated with a film of bituminous paint or similar application before concreting the adjacent panels. In this case there is no need to leave a gap between the panels unless the slab is liable to be exposed to extremes of temperature.

Simple cantilever retaining walls should be concreted in complete sections between expansion joints, in a series of horizontal layers about 3 ft. deep, allowing about two hours to elapse between placing successive layers. If expansion joints are not provided the construction joints should be vertical and may be placed at any convenient section. The vertical joints in counterforted retaining walls should be made through the slab at a section about midway between the counterforts.

Panel walls should be concreted complete, panel by panel, in a single operation, but if this is impracticable the position of the joints in the boundary members should be located to suit the architectural lines. Beams or other large projections at the head of walls should be concreted a few hours later than the wall panel to enable the concrete in the latter to settle.

Concreting in Cold Weather.

The effect of low temperatures on wet concrete is to lower the speed at which the chemical reactions occur while the cement is setting and to produce disruptive forces due to the expansion of freezing water. Since these factors adversely affect the setting and hardening of concrete it is necessary to take steps to counteract them. If there is more than a slight frost effective precautions cannot be taken without considerable increase in costs, and this increase is greater for reinforced than for mass concrete work. Usually in this country frosts are of such a slight nature and small duration that fairly inexpensive precautions can be taken, and in the event of severe frost concreting can be suspended owing to the usually short duration of such periods. The London Building Act stipulates that during mixing, depositing, and setting the temperature of the concrete should not fall below 40 deg. F. Normally concreting can proceed without special precautions if the temperature of the air is not below 38 deg. F. on a falling thermometer or below 35 deg. F. on a rising thermometer.

When it is necessary to place concrete during frosty weather the precautions taken should ensure that the temperature of the concrete while being placed should be maintained at a minimum of 40 deg. F. (and not more than 120 deg. F.), and until the concrete has thoroughly hardened the temperature should not fall below freezing point. To maintain the temperature during placing, the water or the aggregates or both can be heated. During hardening concrete can usually be protected from slight frost by a covering of sacking or straw, but for protection against more than four or five degrees of frost it is necessary to enclose the work and keep the air temperature up by fires or by steam or hot-water pipes. Coke braziers and oil stoves are convenient and economical for this purpose. In building construction, the work can be enclosed by temporary external walls covered with tarpaulins or timber sheeting. As a rule it is only necessary to enclose the story immediately below the floor that is being concreted. The heating apparatus should be below the latter, and the top surface of the floor slab should be protected by sacking, straw, or other covering. In hard frosts heating should be maintained for five or six days after placing the concrete. Heating by fires should be controlled so as to avoid setting fire to the shuttering.

If braziers are used during the construction of slabs and are placed above the slab, there is a tendency to dry out excessively or to bake the concrete below and in the immediate vicinity of the brazier. The heat is excessive if vapour rises from the slab. If, as in the case of slabs laid on the ground, braziers can be placed along the windward edge of the slab, a current of warmed air will pass over the surface of the concrete. For this purpose braziers or fires low down on the ground are more effective than if standing on legs.

Although salt and other chemicals are used to prevent freezing, the best practice prohibits the use of these for reinforced concrete work, but for mass concrete work the addition of 1 lb. of common salt to each gallon of water is sometimes advocated. The use of rapid-hardening Portland cement or especially aluminous cement ensures higher setting and hardening temperatures; sometimes a richer mix is specified to attain the same end. No frozen aggregates or materials containing ice should be used; warming the aggregates removes the contained ice and raises the temperature of the mixed concrete.

A method of warming aggregates is to lay lengths of steel pipes 3 ft. or so in diameter on the ground and heap the aggregate over the pipes, fires being maintained within the latter. A less elaborate way of dealing with small quantities of frozen aggregates is to apply steam or hot water to the materials as they are raked off the storage pile. When using steam from a portable boiler and injecting it into the mixing water, a reducing valve is required to bring the pressure down to a low value.

The simplest and least expensive method of raising and maintaining the temperature of concrete during moderately frosty weather is to heat the mixing water. The cement and aggregates are first mixed dry and then the water is added. Frozen aggregate should not be used and the hot water should not be added to the cement alone. If the water has a temperature of not less than 150 deg. F. the temperature of the concrete will normally be over 60 deg. F.

As setting proceeds heat is generated that in moderately cold weather may maintain or even raise the temperature at which the concrete was placed. In colder weather, as much as possible of the generated heat should be conserved by insulating the shuttering and the exposed faces of the concrete. The radiation from timber shuttering is less than from steel shuttering.

Shuttering can be "blanketed" by nailing a series of boards on the outside of the battens. In exceedingly cold weather the cavity should be packed with dry straw, hay, or similar material. The surface of all freshly-placed concrete should be covered at night with waterproof paper, straw (sacks filled with straw are convenient to handle), sacking, hessian, or similar dry coverings. Any covering should be securely held down to prevent it being blown away. If the concrete is to be protected before setting, the coverings should be kept just off the surface and draughts excluded from the intervening space. The coverings should be kept in position throughout all periods of low temperature until the concrete has hardened. If periods of warm sunshine occur the coverings should be removed to expose the concrete to warmth.

Sound practice for dealing with cold weather concreting can be summarised as follows: Concrete will not suffer adversely if during the first six days after placing the temperature does not drop below 36 deg. F. For short periods of frost during which the temperature does not drop below 27 deg. F. the water or aggregates (or both) should be heated, but for long periods not below 27 deg. F. or for short periods below this temperature the water and aggregates must be heated and the work must be enclosed and the temperature maintained so that the concrete can set and harden without being disturbed by frost.

When using a fairly dry concrete, care must be taken to ensure that the water required for setting and hardening is not absorbed by excessive artificially produced heat; for this reason a slight increase in the normal water content may be advisable.

SECTION II.—CURING AND PROTECTION OF CONCRETE.

Shrinkage.

Hardening of cement in air is accompanied by progressive shrinkage. During the first month shrinkage proceeds at a maximum rate but lessens as the concrete ages; the total contraction in twelve months almost approaches the maximum. Unless a concrete member is free to contract, tensile stresses are set up in the concrete and may exceed the ultimate tensile strength of the material, with the result that cracks are formed. These cracks, although usually of no structural significance, may be objectionable from the point of view of appearance, watertightness, or their effect on finishes.

Cement hardening in water acts in the opposite manner; it increases in bulk, but upon subsequent drying the increase is lost and shrinkage occurs. Temporary saturation of hardening mortar or concrete, although not eliminating shrinkage, decreases the rate of shrinkage, and the consequent limitation of the tensile stresses, due to restrained contraction in the early days when the concrete has developed little strength, assists in preventing shrinkage cracks. A concrete which hardens in conditions that tend to minimise the initial rate of shrinkage attains a higher strength at any given age than a concrete that has hardened in air.

Shrinkage has been explained by describing set cement as material, the nature of which is that of a small proportion of solid diffused in a much larger volume of water. This material retains its water as long as no external influences tend to release the water by changing the nature of the solid or so long as evaporation of the water does not take place. A reduction in bulk, or shrinkage, clearly follows loss of water by evaporation, and just as obviously if moisture is kept in contact with the concrete during the early stages of hardening, evaporation of the contained water is eliminated or at least delayed.

The problem of shrinkage is so complex that it is at present impossible to say that certain measures will prevent contraction cracks, but experience indicates certain precautions in design and construction that will reduce the risk of cracking.

Joints can be inserted to isolate portions of the structure and predetermine the positions where cracks will form. Short gaps, 2 ft. to 4 ft. wide, can be left between, say, 30-ft. lengths of long walls, the gaps being filled in after initial shrinkage has taken place. Reinforcement can be provided to supplement the resistance of the concrete to the tension produced by restricted contraction. Since it would not be economical to resist the whole of the tension by reinforcement, the concrete should be of such a quality that the bulk of the shrinkage stresses is resisted by its own strength. To ensure this strength is the object of maturing or curing the concrete.

Curing.

The aim of curing is to maintain the immature concrete moist for a given period, usually seven days, but with high-alumina cement the concrete need only be kept thoroughly wet for twenty-four hours. Moist conditions are more essential when rapid-hardening Portland cement is used.

The general method is periodically to apply water, usually through a hose, to all exposed faces of the concrete, but by covering the faces of walls with sacks,

FIG. 173.—COCO-NUT MATTING APPLIED TO SLABS DURING CURING.

canvas, hessian, coco-nut matting, or similar materials that can be kept saturated, the moist conditions are maintained more constant. Horizontal surfaces after initial set can be kept moist by a covering of wet sand, earth, sawdust, etc., or by flooding with water an inch or more in depth (" ponding "), the water being retained by clay banks. Ponding is often adopted for roads and other ground slabs. A common procedure is to cover the concrete for twenty-four hours with wet coco-nut matting and then for twenty-one days with 3 in. of soil maintained moist. *Fig.* 173 illustrates a portion of a road slab covered with coco-nut matting during curing. In this method the matting is in direct contact with the concrete, but if a frame is built up from 4-in. by 2-in. timbers as shown in *Fig.* 174 this can be used to protect newly placed concrete from the direct rays of the sun or from drying winds.

For floor slabs and the bottoms of swimming baths, tanks, reservoirs, and similar structures, a covering of wet canvas, coco-nut matting, or sand is usually more convenient in the early stages, although if a portion of the tank walls is erected simultaneously with the bottoms the latter can be flooded. Sand may not prove successful on steeply sloping bottoms.

In frosty weather it is necessary to take steps to prevent the curing water from freezing or from attaining and maintaining a low temperature that would be detrimental to the maturing of the concrete. Under such conditions curing

Fig. 174.—Coco-nut Matting attached to Frame.

water is best dispensed with. During hot weather the maintenance of the concrete in a moist condition is imperative. Watering should be frequent and regular and should be continued throughout week-ends and holidays. Exposed surfaces, if not covered by canvas, earth, paper, or similar covers, should be protected from the direct rays of the sun by suitable shades.

Apart from the foregoing precautions, the risk of shrinkage cracks can be minimized by using as dry a concrete as practicable and by delaying the slab finishing until the concrete has begun to stiffen slightly.

In using coverings of canvas or sand it is essential not to damage the concrete surface by placing the covers in direct contact with the concrete before it has hardened sufficiently. Until final set has taken place there is usually sufficient surface moisture adequately to maintain the required conditions for curing. Sometimes, however, the surfaces of large slabs dry out before they can be saturated, and surface cracks occur within an hour or so of placing the concrete.

These cracks may only be a few inches in length and of no appreciable depth, and can be repaired by pointing with neat cement or cement-sand mortar before applying the saturated covers or flooding.

In wet weather watering can be dispensed with, but during the time rain is actually falling concreting should be suspended. Should rain occur within an hour or two of placing, the concrete should be protected by boards, sacks, or other material from the direct fall and from the flow of rainwater, otherwise the exposed surfaces will be pitted or denuded of cement.

The surface of freshly-placed concrete should also be protected from the action of strong winds ; precautions against the drying effect of such winds are particularly necessary in elevated or riverside or marine structures. Strong

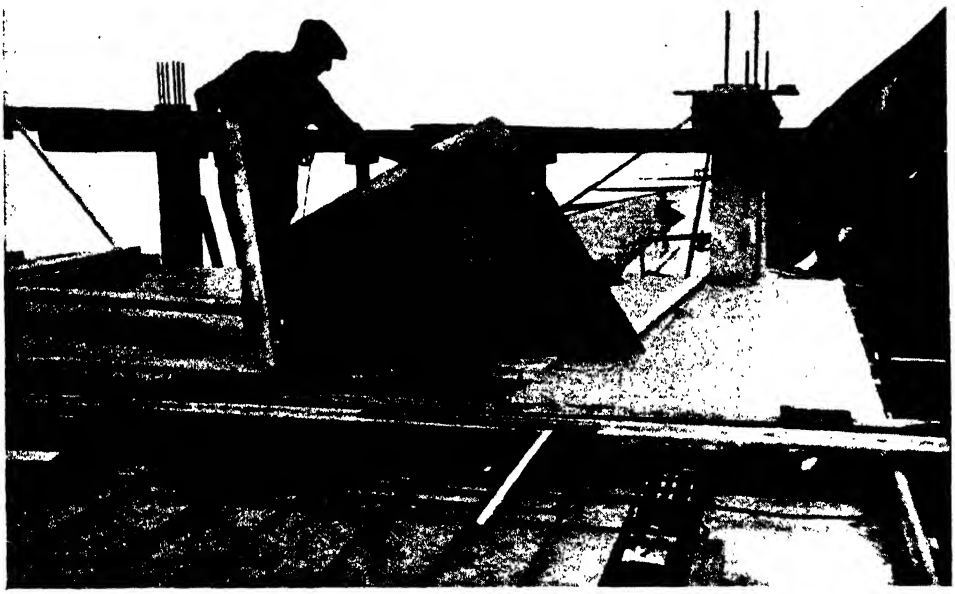


FIG. 175.—APPLYING CURING PAPER.

waterproofed paper can be used as a covering for newly-laid concrete for protection against frost, drying winds, heavy rains, or high temperatures. The process of laying sheets of paper on the surface of a concrete slab is illustrated in *Fig.* 175. The paper is laid an hour or so after concreting and timber battens prevent the sheets being disturbed by light breezes.

The surface of high-alumina cement concrete must be prevented from becoming dry at any time within 24 hours of mixing. Finally it should be mentioned that experience of the methods required in curing concrete in the United States are by no means applicable in Great Britain where the atmosphere is more humid and the temperature range less extreme.

While maturing concrete structures must be protected from influences that may overstrain or otherwise damage the concrete. Overstressing due to premature removal of the shuttering has been discussed in a previous chapter.

Materials should not be stored on floors in excess of the load for which they have been designed, nor should these loads be applied until the engineer estimates that the assumed strength has been attained. Piles of aggregate material, stacks of bricks, bags of cement, and similar loads should be located over beams in preference to being stacked in the middle of the floor panels, and if the load on the beam is excessive the props should be kept in position throughout the duration of the load. Aggregate should not be piled against panel walls unless the engineer gives permission, nor should back-filling behind retaining walls be placed until it is estimated that the concrete has acquired the desired strength.

If it is necessary to bring the permanent live and dead loads on to a structure before the concrete has attained the desired strength, it is essential that temporary props or other supports should be introduced to relieve the concrete of the load for the time being. The props and their soleplates should be designed to take the whole load.

FIG. 176.—PROTECTING STAIR TREADS.

Structures should be kept free from undue vibration while maturing; in bridge construction, particularly, it is necessary to ensure this by suitably controlling the speed of the road or rail traffic passing under or adjacent to the bridge. The piers of bridges over waterways should be protected from damage due to floating debris, by piled timber fenders or by cribs, and in culverts and riverside work the concrete should not be brought in contact with rapidly flowing water until the exposed surface is sufficiently hard to withstand the scouring action. Steps should be protected from early wear by attaching boards on the treads and risers as shown on *Fig.* 176.

Although it is advantageous in promoting proper conditions for curing to cover the bottoms of tanks, reservoirs, and similar structures with a few inches of water, the depth of water should not be such as to produce excessive pressures on the bottom or walls until these are sufficiently mature. It is usually recommended that open-air baths and tanks should be kept filled during the winter months, except during periods of extreme frost, when, in the case of narrow tanks, the ice may exert outward horizontal thrusts in excess of what they might reasonably withstand ; in this consideration the width of the tank is a prime factor, as in a wide tank there is a probability of the restrained ice sheet buckling before the thrust on the walls becomes dangerous.

SECTION III. STRENGTH OF CONCRETE.

Factors Affecting Concrete Strengths.

The strength attained by concrete is susceptible to variations within very wide limits due to differences in a number of factors. The chief among the latter are the age of the concrete, the richness of the mix, and the water content ; but cleanliness, quality, grading and other properties of the aggregate, soundness and hardening properties of the cement, conditions of curing, temperature, efficiency of mixing and placing, also affect more or less considerably the strength of the concrete. Conditions conducive to reduction in strength are usually due to lack of experience or conscientiousness, and inadequate supervision. It is important that the extent to which adverse conditions reduce the strength should be clearly understood by clerks of works and contractors.

The diagrams in *Fig.* 177 show typical variations of strength with age, water-content, and cement-content. The strength-age curves are given for rapid-hardening Portland cement and aluminous cement, as well as for normal Portland cement. If the conditions are suitable for using rapid-hardening Portland cement, the three months' ultimate strength of a concrete made from this material may be slightly superior to that attained by normal Portland cement. The ultimate strength attained by a concrete made from a good brand of high-alumina cement will generally be greater than that of a Portland cement concrete. From the curves it is seen that the gain in strength is considerable during the early days and decreases with age. The increase may be progressive at a very slow rate over a number of years.

The curve of variation of strength with water content is typical for normal concretes. There is a certain ratio of water to cement content that gives the maximum strength. Less than this amount produces a very rapid decrease in strength, and more than this amount a less rapid decrease. Too much water produces a weak, segregated mix, and too little a weaker unworkable mix. An increase in the cement content is accompanied by an increase in strength if the water content is kept constant, but a considerable increase in the water-content introduced to preserve workability may offset the advantage gained by adding extra cement. If the cement-water-ratio is kept constant no change in strength is to be expected. The curve represents the variation in strength that might be expected if a constant consistency (uniform slump) is maintained, other conditions being kept constant.

The effect of temperature during mixing, placing, and curing is generally to cause a reduction in strength if low temperatures prevail. The reduction at low temperatures is more marked with rapid-hardening Portland cement than with normal Portland cement up to a few days old, but at the end of one month the reduction is materially the same for both cements. The strength attained at the end of a given period is usually less if low temperatures prevail at the beginning of the period than if these temperatures occur towards the end of the period only. If the concrete is frozen immediately after placing the hardening processes are practically suspended and, in the most favourable cases, the increase in strength during the period of frost is very small compared with that attained under normal conditions. Wet mixes suffer more seriously, due to low temperature, than drier mixes.

Experience and research seem to indicate that generally the higher the curing temperature (so long as the concrete is kept moist) the stronger the con-

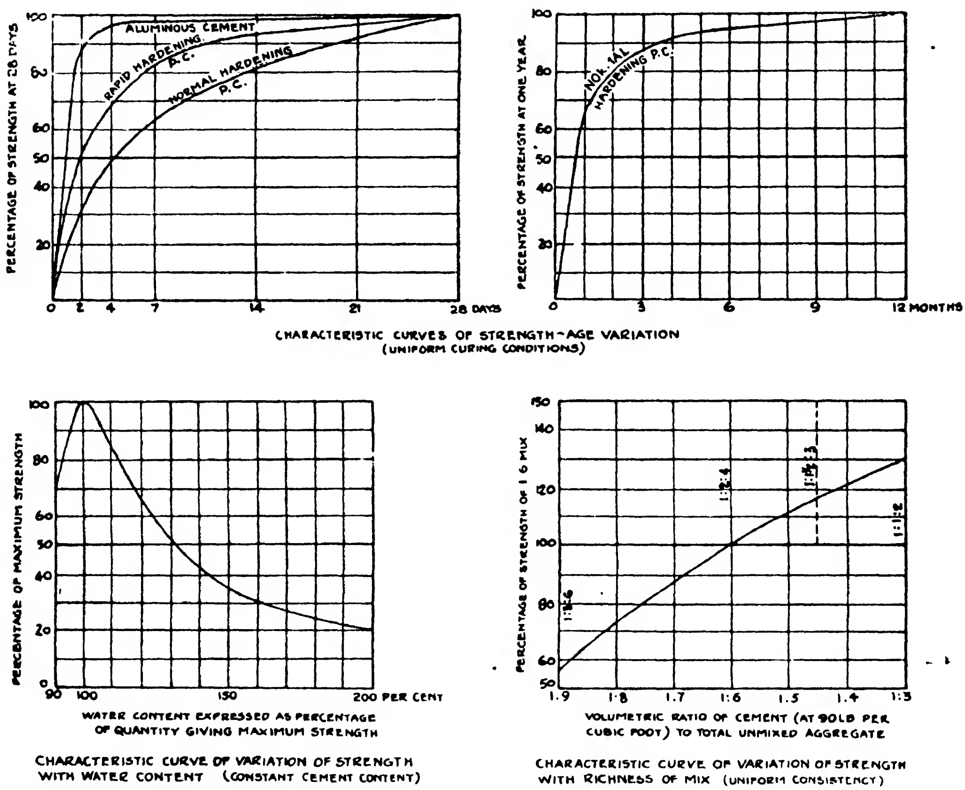


FIG. 177.—STRENGTH OF CONCRETE.

crete, the exception being when the temperature during placing exceeds the curing temperature, in which case the concrete attains a lower strength than the same concrete cured at a temperature equal to or higher than the placing temperature. A temperature of 212 deg. F. destroys the setting process if applied at mixing, but if occurring after initial set, it may have a beneficial effect.

Among the other factors controlling the strength of concrete are such obvious causes as the use of unsound cement and allowing the concrete to dry out too rapidly. The necessity of having a strong aggregate has already been stressed; properly graded aggregates ensure a denser and stronger concrete. Dirty aggregates also reduce the strength; although it is not unknown for test results to indicate the reverse of this, in these cases the reduction in strength when using recently washed aggregate may be due to the fact that the moisture in the washed aggregate augments the amount of water in the mix, the lower strength then being attributed to the higher equivalent water-cement-ratio, or it may be due to too much fine material having been washed away.

Inefficient mixing or separation during placing leads to lack of uniformity in the mix and variable strength. Insufficient tamping tends to produce a porous and weak concrete.

How one or more of these factors can in practice affect the strength of concrete is best illustrated by examples taken from practice. In one case the early batches of concrete showed a strength of only 80 per cent. of the specified strength, the deficiency being probably due to excess water and to the fact that the cubes had been cast in rough timber moulds from which they were taken after a few days and left lying in a dry heated room until dispatched to the testing station. Stricter control of the water content, the provision of metal-lined moulds, and proper care of the cubes resulted in average strengths of 83 per cent. in excess of the specified minimum. As the initial stricter supervision relaxed slightly, the strength dropped to within 25 to 50 per cent. in excess of the requirements of the specification. After the job had been in progress for some while, circumstances arose necessitating a change of control. Under the new regime a wetter concrete was at first used, probably to hasten placing, but this was reflected in the strength dropping to 86 per cent. of the specified value. Subsequently supervision of water content was again tightened, with the result that from that time until the completion of the contract the ultimate strengths varied from that specified to an excess of 17 per cent.

On another contract a number of 6-in. test cubes was made from a wet hand-mixed 1 : 1 : 2 concrete, using a rapid-hardening Portland cement, a screened but unwashed crushed granite aggregate, and a fairly fine sand. The temperature at the time of mixing was only a degree or so above freezing point, and this temperature prevailed until the day previous to testing. The cubes were made in rough timber moulds, the faces of which were not quite plane. With so many adverse factors it was not surprising that at eight days the crushing strength was only 2,730 lb. per square inch, while one cube, the faces of which were not quite parallel, showed an ultimate strength of only 1,780 lb. per square inch instead of over 4,000 lb. per square inch that might be expected from this mix. A second series of test specimens was made, these being 4-in. cubes cast in well-made timber moulds. The composition of the concrete was the same as before, but it was hand mixed by men not familiar with the process; the low temperature conditions could not be avoided. A consistent strength of 2,000 lb. per square inch was now obtained at six days, but even allowing for the difference in age and the smaller size of cube the reduction in strength was serious and far below that required by the design. This example demonstrates the necessity of using the best materials and employing experienced workmen when working under abnormal conditions (in this case low temperature and hand mixing). No doubt a superior concrete could have been made if a coarser sand had been available, if the dust adhering to the crushed aggregate had been removed, and if a drier mix had been insisted upon.

The importance of avoiding excessively wet mixes is shown in the case of a contract where, with the same washed gravel aggregate, sand, and rapid-hardening Portland cement, the 28-day cube strengths varied from 3,510 lb. to 6,100 lb. per square inch for a 1 : 1½ : 3 mix; the difference in the concretes was almost entirely that of water content, although in extreme cases the materials did not separate or become too stiff to place among intricate reinforcement.

The specification on another contract required a minimum of 2,000 lb. per square inch on 1 : 2 : 4 concrete at 28 days. Cubes tested at seven days gave an average strength of 1,580 lb. per square inch, and, on the assumption that the 7-day strength is between 60 and 70 per cent. of the 28-day strength, the concrete might be considered as only just up to specification although made by experienced contractors. With identical materials and under similar conditions another contractor obtained strengths of 2,830 lb. per square inch at twenty-eight days with a 1 : 1⅝ : 3½ mix.

A mix of the same proportions using a special selected coarse sand brought from a distance, rapid-hardening Portland cement, and a dusty crushed stone aggregate mixed with river gravel, gave a strength of only 1,780 lb. per square inch at seven days on carefully made cubes, this being barely 80 per cent. of the specified strength. This can be compared with a 1 : 2 : 4 mix using rapid-hardening Portland cement, a coarse sand, and a washed graded gravel for which strengths of 4,000 lb. per square inch at twenty-eight days were obtained. The characteristics of an aggregate have an obvious effect on the strength of the concrete, but comparative results of two aggregates always require careful consideration. In one case, two series of cubes were made, one with limestone and the other with a good clean gravel aggregate. The pieces of limestone were covered with a film of dust, and from inspection of the two materials it would have been thought that the gravel would have produced the stronger concrete. The reverse was demonstrated by a test, although the water content and other conditions were maintained as constant as practicable. The explanation may have been that the limestone absorbed some of the mixing water and thus reduced the water-cement-ratio with a resulting increase in strength.

Strengths largely in excess of conservative specified strengths, far from being uncommon, are to be expected if good practice in all stages of concrete manufacture is adopted. As an example, consider the resulting strength of 3,650 lb. per square inch at fourteen days obtained on a 1 : 1½ : 3 mix, using normal hardening cement and a well graded gravel aggregate on a contract controlled by a specification requiring 1,425 lb. per square inch at this age.

Specified Strengths.

The foregoing examples are exclusively concerned with cubes made on the site (as opposed to laboratory-made cubes), and on jobs where the specified strengths were conservative. The minimum strengths that an average concrete for use in reinforced concrete work should reach are similar to those tabulated in columns A and C of *Table* XIX. Nowadays 28-day strengths are the more commonly accepted standard with normal-hardening Portland cement, or 7-day strengths with rapid-hardening Portland cement. Strengths such as those given in column A would be associated with incidental concrete work in a non-concrete building, or in work carried out under conditions of special difficulty as regards labour, materials, weather, or accessibility. The design stresses would be correspondingly moderate.

TABLE XIX.

STRENGTH OF CONCRETE.

(Specified ultimate strengths, lb. per square inch.)

	Column	A	B	C	D
		—	Ministry of Transport	L.C.C. By-laws Ordinary concrete	L.C.C. By-laws Quality A
Mix	1 : 2 : 4	2,000	2,250	2,250	2,850
	1 : 1½ : 3	3,400	2,700	2,550	3,300
	1 : 1 : 2	2,600	3,600	2,925	3,750
Age	Normal cement	28 days	28 days	28 days	28 days
	Rapid-hardening cement	7 days	7 days	—	—

With ordinary care and experience in the selection of the materials, consistency, and curing, the minimum strengths that should be ordinarily obtained are those tabulated in columns B and C.

If great care is taken with the selection and proportioning of the materials, and particularly with the control of the water content, and if the job is subject to strict supervision, the strengths attainable can show considerable improvement on those in columns B and C, and the values given in column D may be expected. Examples of higher strengths than these are not infrequent.

The specified strength should be related to the stresses assumed in the design, the former being usually, at twenty-eight days, three times the latter. Some regulations, although basing the criterion of strength on the 28-day value, allow a 7-day test to be made if time does not permit making the longer period test. In these conditions a 28-day test should also be made, and the 7-day results should be at least 70 per cent. of the 28-day results.

The procedure in the matter of testing concrete is for the engineer to specify minimum strengths for concrete in various parts of the structure. Unless there are special conditions in the contract, such, for example, as materials being supplied to the contractor, the latter is usually responsible for the maintenance of the strength of the concrete throughout the course of the job. Before work commences the contractor should make preliminary tests to assure himself that the materials, proportions, and consistencies can produce a concrete complying with the specification, unless his experience has shown that satisfactory strengths are obtainable. During the progress of the contract tests should be made periodically to prove that the minimum strengths are being maintained. When the specified strengths are of the order of those given in column D of *Table* XIX tests should be made at least twice weekly or at any time when the materials, mix, or other conditions are changed. If preliminary tests show a deficiency in strength, alterations in the proportions, treatment of materials, or water content should be made until a satisfactory concrete is obtained. If these alterations infringe or improve upon the requirements of the specification in any way it is necessary to have the engineer's consent before carrying the emendments into practice; unit costs may also need adjustment.

The test results from any series of cubes made from the same concrete should be fairly consistent, and it is suggested that the greatest acceptable variation should be 15 per cent. from the average of the maximum and minimum of three cubes. If any of the tests made during the progress of the work fall below the specified minimum it is an indication that the standard of conditions is not being maintained, and steps should be taken to find the cause and to reinstate the original conditions. In falling below the specified standard it may be proved that the contractor has contravened the specification, and the engineer must decide whether the deficiency in strength is sufficient to warrant the removal of the weaker concrete. Specifications are sometimes definite upon this point insofar as they indicate that if the progress tests show results more than 10 per cent. below the required minimum strength the concrete represented by such tests shall be cut out and made good at the sole cost of the contractor. A less severe procedure is to subject the members made with the defective concrete to a test load of not less than the design load. This load should not be applied until the concrete has matured. An examination of a broken piece of concrete will usually reveal any defects inherent in the material, and the evidence can be briefly summarised as follows.

The presence of air pockets or bubbles denotes insufficient mixing or inefficient tamping, the latter being the more probable. The larger stones should be fairly well spaced, otherwise poor mixing is indicated with dry concretes, or poor mixing, separation, or excess tamping in wet mixes.

If the fracture does not pass through the stones, a defect in the mortar is indicated; if the latter is crumbly, either the cement is defective or the sand contains an undue amount of loam. Failure of the stones to remain fixed in the mortar indicates unclean aggregate, usually a dust or clay film on the surface of the stones.

Test Cubes.

Since the onus of producing a concrete of the specified strength usually falls upon the contractor, he should ensure that the full value of the concrete is shown in the test results. Unless care is exercised throughout all the stages of preparing and curing the test cubes, the increase in quality due to careful selection and proportioning of the materials and due to control of the water content may be overshadowed by defects in the cubes that may lead to apparent strengths much below the true strength of the concrete as placed. The most prevalent defects are due to faces of the cube not being parallel, and to poor ramming and indifferent treatment after casting.

Most tests are made on 6-in. cubes, although tests on cylinders are sometimes required. A test on a 6-in. diameter cylinder may give a result only 80 per cent. of that obtained on a 6-in. cube cast from the same concrete, and when interpreting cylinder test results this fact should not be overlooked. Six-inch cubes usually give better results than 4-inch cubes, but the latter are sometimes allowed if the mix is wet. For concretes of medium or plastic consistency a 6-in. cube is preferable, and for dry concretes 8-in. cubes are sometimes specified.

The size of the cube is limited by the capacity of the testing machine, but most laboratories possess 100-ton machines that can test 6-in. cubes having ultimate strengths up to 6,200 lb. per square inch.

The importance of the mould being die square is self-evident. Although wooden moulds are frequently used it should be remembered that the test results of cubes cast in them are invariably inferior to those of cubes cast in metal or metal-lined moulds. The decrease in apparent strength may be of the order of 20 per cent.

Cast-iron or steel moulds (*Fig.* 178) with machined faces and made to standard dimensions are on the market or can be hired from testing laboratories. Metal moulds should be oiled before being filled.

Timber moulds lined with zinc produce good cubes if well made and coated with mineral grease before filling. The types illustrated in *Fig.* 179 are sturdy and can be easily dismantled for extraction of the cube.

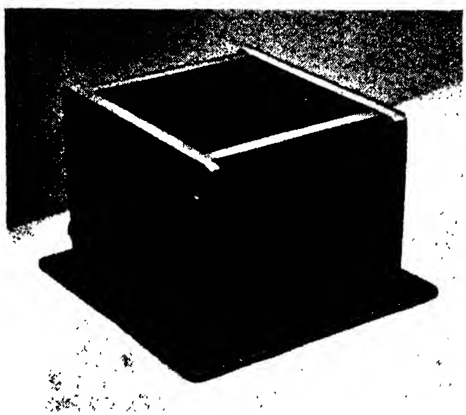


FIG. 178.—STEEL MOULD FOR TEST CUBES.

The moulds should be filled with concrete representative of that being placed and should be taken either from masses that have just been placed or from a point of delivery as near the final placing position as practicable. The filling should take place as soon as possible after the selection of the sample, and during the whole of the operation, including the transference of the concrete to the place of making the cubes, care should be taken that no water is lost from the mix. To obtain comparable results the concrete should be placed in the moulds in three layers, each layer being rammed into place. For mixes with slumps exceeding 1½ in. each layer should be given twenty-five strokes with the rammer, but drier concretes should be given thirty-five strokes.

When making test cubes for vibrated concrete it is best to attach the moulds to a board fixed to the shuttering, the vibration of the latter being transmitted to the moulds.

When the test cubes are completed the moulds should be stored for twenty-four hours on the site in a damp place free from vibration, and at the end of this period, which can be subject to a tolerance of half-an-hour either way, the cubes should be taken from the moulds and kept in water or damp sand until sent to

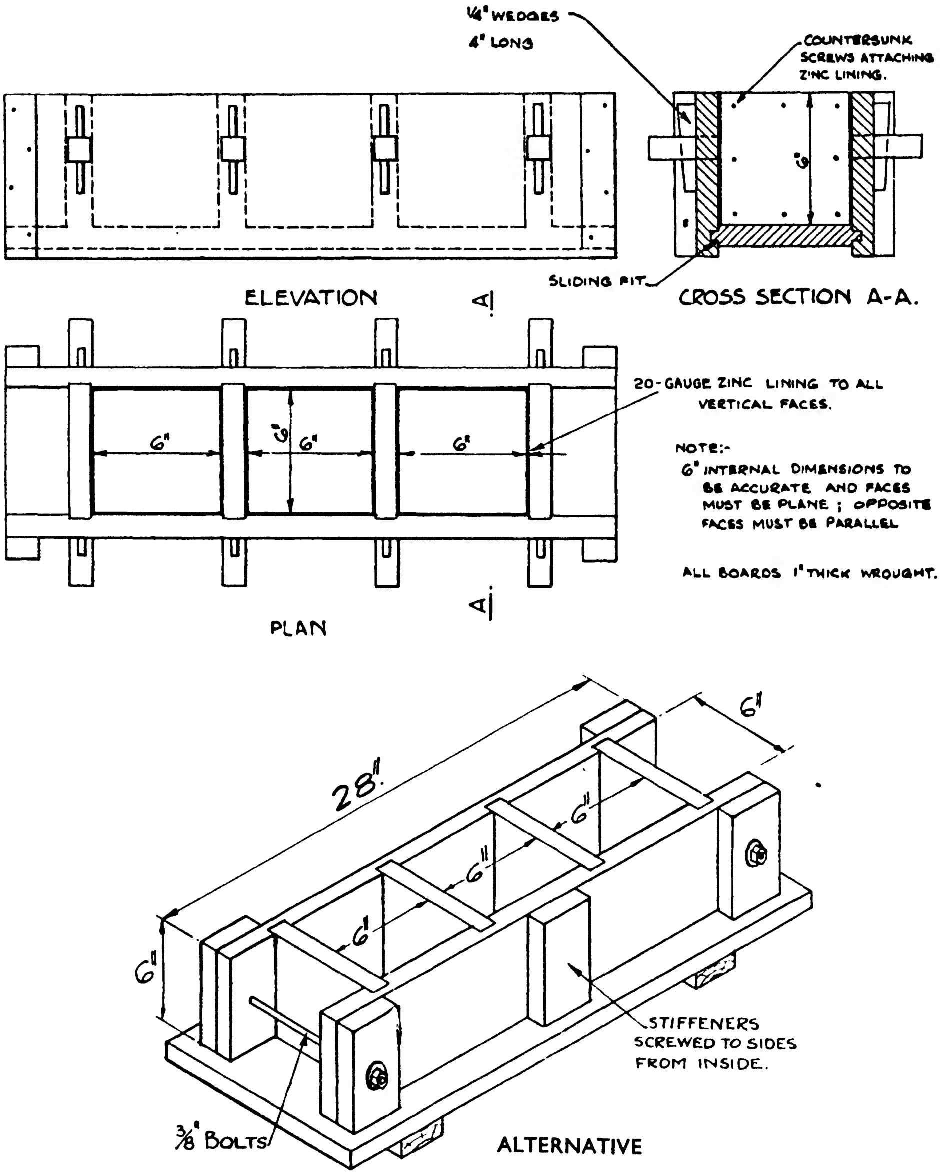


FIG. 179.—TIMBER MOULDS FOR TEST CUBES.

the laboratory. The period during which the cubes are retained on the site should be about three-quarters of the age at test. During this period, the storage temperature should not fall below 40 deg. F., while unduly high temperatures due to natural or artificial heat should be avoided.

CHAPTER VII

FOUNDATIONS

SECTION I.—EARTHWORK.

Characteristics of Soil.

In connection with excavation, transport of spoil. and backfilling, the properties of soil must be considered. These are given in *Table* XX. Excavation being generally paid for as measured net in the solid, it is necessary to know by what proportion it "bulks" when dug out. One cubic yard measured in the ground may occupy up to 1½ cu. yd. when excavated and thrown out or thrown into containers. Thus removal, transport, and subsequent handling deal with greater volumes than the net measurement. Rock may increase in volume up to 100 per cent., the sizes of pieces of rock or chalk affecting the amount of bulking. Large blocks bulk up to 80 per cent., and medium size 70 per cent. Blasted rock bulks more than material from crushers, and the more uniform the grading of crushed rock the less the bulking. Earth and clays may bulk up to 30 per cent. Sands and gravels bulk less, say, up to 20 per cent. Values for different types of soil are given on *Table* XX.

A given quantity of excavated material will occupy less volume when refilled than when loose, the amount of ramming, the type of soil, and time affecting the amount of settlement. *Table* XX gives a number of representative settlement values. Rock filling subsides least, and the reduction may vary from zero to one-sixth or one-quarter of the original loose volume. In hand-packed rock filling there may be no settlement. A common allowance for settlement of loam or earth filling is 1 in. per foot, but in soils with large clay contents the subsidence may be greater than this and may be progressive over a number of years. When filling is to be tipped on mud or reclaimed ground the actual amount deposited may be more than twice the net measured volume. When subsequent construction is to be carried out on filled ground it is often convenient to arrange that the lorries travel over the filling so as to accelerate consolidation. By varying the positions of the roads and tracks a large area can be consolidated.

TABLE XX.

SOIL CHARACTERISTICS.

DESCRIPTION OF MATERIAL	BULKING	SETTLEMENT	WEIGHT	BEARING	STABILITY		
					PERMT	TEMPORARY	
	VOLUME OCCUPIED BY ONE CUBIC YARD OF SOLID WHEN DUG OUT - CU. YD.	VOLUME OCCUPIED BY ONE CU YD. OF LOOSE MATERIAL AFTER NATURAL SUBSIDENCE - CU. YD.	LOOSE SOIL. CUBIC FEET PER TON.	MAX. SAFE BEARING PRESSURE TONS PER SQ FT.	NATURAL ANGLE OF REPOSE. DEGREES.	SLOPE OF FRESHLY DEPOSITED MATERIAL	VERTICAL FACE WITHOUT SUPPORT FEET.
EARTH :- VEGETABLE, GENERAL	1 1/4	0·90	21 (PUNNED) 24 (NOT PUNNED)		30° TO 40°	1·1 TO 1:1/2	2 TO 3
DRY.					30°	1·1 3/4	
DRAINED LOAM.					40°		5 TO 8
MOIST.					45°	1·1	
VERY WET						1:3	
ALLUVIAL SOIL.				1/2			
MADE GROUND.				1/2			
DRY PEAT			64		45°		
WET PEAT.			32		15°	1. 1 1/2	
RIVER MUD.			25	0 TO 1/4	5° TO 20°		
CLAYS :- GENERAL.	1 1/5		17 1/2	-			
WET CLAY FILLING	1 1/6		19		15°-20°	1· 3 1/2	
SOFT.	-			1		1·1	
FAIRLY DRY.	1 1/2-1 2/3		20-30	2	30°	1:1 3/4	
SANDY.			20	2	35°		
FIRM AND DRY. (UNDISTURBED)			19	3	45°		9 TO 12
HARD AND COMPACT. (e.g. LONDON BLUE CLAY.)				4			
SANDS :- GENERAL.	1/8 TO 1/6		20	-			
COMPACT.	-			4			
FAIRLY DRY AND FINE.	1 1/10		24	2	35°	1. 1 1/4	1 TO 2
LOOSE	-		22	1			NIL
WET.	1 1/20		19	1	25°	1·2	-
VERY WET.	-			1/2		1:1 1/4	NIL
GRAVELS :- GENERAL.	1 1/10 TO 1 1/6		19	-			
AS FILLING.				-	35°		
DRY.			22 1/2		35°	1·1	1 TO 2
COMPACT.				4			
DRY AND SANDY.			19		30°	1:2	
WET AND SANDY			17		25°		
SHINGLE.			20 TO 23 1/2		35°	1:1 1/2	NIL
ROCK :- FILLING.	-	NIL	15 TO 22 1/2		45°		.
CHALK.	1 1/2-2	-	24 30	6	-	.	ANY DEPTH.
SHALE.	1 1/2-1 3/4	-	20-24	10	-	.	
SOFT ROCK.	1 1/2 TO 1 4/5	.		10	VERTICAL	.	
HARD ROCK (GRANITE)	1 1/2 TO 1 4/5	.	20-23	20	VERTICAL	.	
LIMESTONE	1 3/4-2	.	22-24				
BASALT	1 2/3	-	20-24				
SANDSTONE	1 3/4	-	24-30				

Filling should be as uniform as possible, especially when it is required to present a constant bearing resistance to foundation loads. For this reason the soil should be spread in thin layers, say, 6 in. to 12 in. deep, each of which should be rammed or rolled before the next is laid. Hand ramming is suitable for these thicknesses, but a mechanical rammer is required to consolidate effectively layers up to 2 ft. thick.

When filling behind retaining walls, the engineer's directions should be carefully followed so that the structure is not subjected to excessive pressure. The backfilling should not be tipped from a height, and an interval of a day should be allowed between depositing successive layers of clays that are inclined to form a slurry.

When the excavation is principally in a hard soil but in part runs out into softer material, unless the foundations have been specially designed for this condition it is preferable to dig to the level of the harder strata all over the site and to make up to the required foundation level with a non-compressible filling, such as packed broken rock, well rammed rubble, or mass concrete best resembling the exposed harder strata. This ensures a more uniform bearing resistance.

When freshly dug, the sides of an excavation in almost any moderately dry ground will stand with a vertical face for limited depths and for a limited time. It is safe to assume that slightly damp soils will stand better than dry material. Damp soils may dry out partly on exposure, resulting in crumbling or slipping of vertical or steep faces. Excavations in brick rubbish or similar filling will stand almost vertically for a depth of several feet if there are no bands of clay to cause slipping. The usual depths to which excavations can be left temporarily with a vertical face without slipping are also given in *Table* XX, together with the approximate slope which various materials assume when thrown in heaps. These slopes may only be maintained temporarily, as prolonged exposure will ultimately cause the slope to become flatter and more in accordance with the angle of repose (*Table* XX) assumed in designing retaining works. In the construction of sewage works and coal-handling plant, angles in excess of 50 deg. are often possible in excavation and the excavation is often taken out to this slope for some depth. Whereas in firm dry clay it may be possible to work to such a slope, in gravels and sands, especially if they are waterbearing, it is difficult to maintain it and special design and timbering may be required.

Excavating has an effect on the characteristics of clays and soils containing clay, and repeated handling in damp conditions may change these materials to a slurry and reduce the angle of repose. The sides and bottoms of clay excavations may swell in wet weather, whereas abnormal warmth or dryness will produce contraction, cracking, and possibly crumbling. Thus, if excavations in clay are left open for any length of time, it is usually necessary to trim them to the required sizes before resuming construction.

Excavating.

Methods of Excavating.—The method of excavating depends primarily upon the extent of the work, the nature of the soil, the depth of the excavation, and whether timbering and pumping are required.

Surface excavation in shallow trenches for footings or shallow holes for column bases is usually performed by hand. Hand excavation is also used in deep holes and trenches, the material being brought to the surface by throwing it with long-handled shovels on to a succession of platforms formed in the excavation at convenient vertical intervals. If a crane is available, or if the volume of earth to be dealt with warrants its provision, deep, heavily-timbered excavations can be carried out by hand digging, the material being loaded into skips or buckets that are lifted out of the hole by the crane. In the illustration in *Fig.* 180 a locomotive crane for handling skips of excavated material is mounted on a track traversing the excavations.

Open excavations involving large volumes, as in dock or reservoir construction or in levelling steeply-inclined sites, are dug by steam navvies or grabs. Levelling gently-sloping sites, or removing surface soil from a large area, may

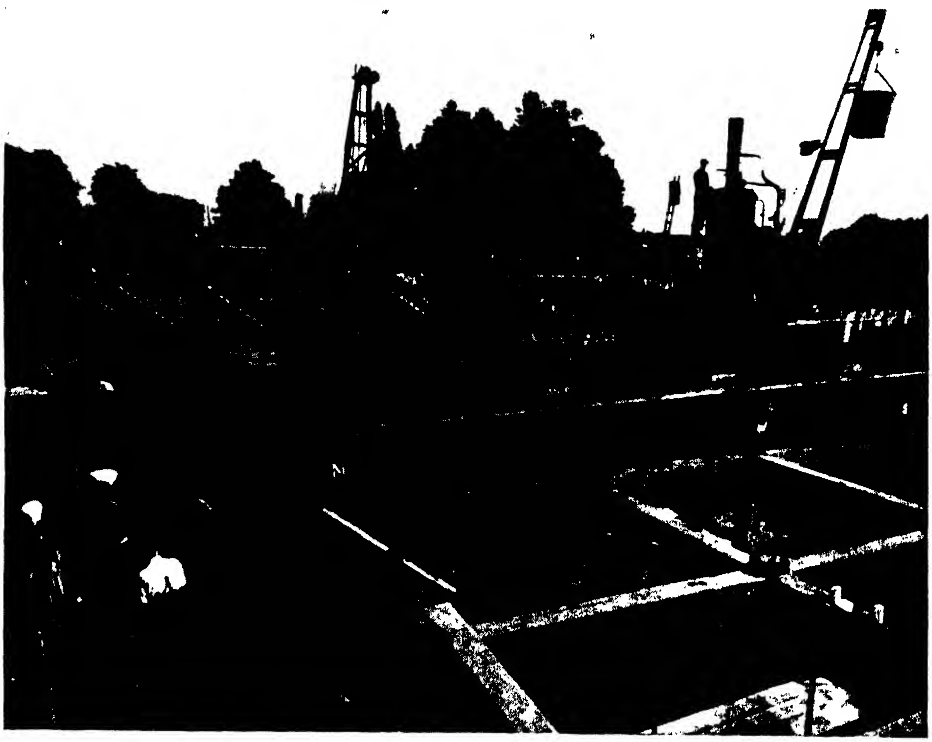


FIG. 180.—HANDLING EXCAVATED MATERIAL BY CRANE SKIPS.

be performed by mechanical scrapers in certain classes of ground possessing little cohesion. In heavier soils a combination of ploughing and scraping is often successful. Mechanical methods are often advantageously combined with hand digging. For example, in excavating within a cofferdam a grab suspended from a crane can be used to dig through the soft top layer, the lower and harder strata being loosened by hand and loaded into skips handled by the crane. Excavation for basements in city buildings is often taken out by first digging by hand a trench around the edge of the site in which the retaining wall is constructed, the "dumpling" in the middle of the site afterwards being removed by a drag-excavator. *Fig.* 181 shows the closing stages of an excavation for a basement.

The retaining wall construction is finished and the drag-line excavator seen in the background is completing the removal of the dumpling.

Long pipe-trenches of moderate depth may be dug by a mechanical excavator. In wet ground or in wet seasons, as the pipe-laying may be held up by a flooded trench, short sections are better. If streets have to be held from collapsing, short lengths with connecting headings are generally used.

Removal of ground below water is done by dredging in open channels, or locally by grabbing within cylinders and cofferdams. Rock excavation, unless loose, requires blasting followed by hand or mechanical collection of the spoil.

Hard dry clay may prove as difficult to cut as rock and make ordinary spade work impossible. A jack-hammer or pneumatic drill fitted with a spade-head may then be used.

FIG. 181.—EXCAVATION FOR BASEMENT.

The method adopted in a given case would in principle follow the foregoing indications, but the details of the method would be settled after going into the comparative costs of alternatives. These alternatives depend upon the plant available and suitable for the work. Examples of the preparation of such estimates are given in subsequent paragraphs.

In loose ground a man can usually dig and throw out about 1 cu. yd. per hour, but in compact gravelly soils only half this amount can be accomplished, while digging in stiff clay is even more difficult. The output per man will also be seriously curtailed if timbering is required or the ground is waterlogged. The overall cost of excavation by hand covers the operations of digging, pick-work (if any), successive vertical throws or loading into crane skips, loading into wagons, hauling to and depositing on the dump, or returning part of the spoil and ramming. The cost covers the cost of hoisting and transport plant, together with the cost of timbering and pumping. In loamy soils, gravels, and soft clays this overall cost may vary as follows for increase in depth :

From ground level to 6 ft. deep	. . . .	cost = 1
,, 6 ft. to 12 ft. deep	. . . .	= $1\frac{2}{3}$
,, 12 ,, 18 ,,	. . . .	= 2
,, 18 ,, 24 ,,	. . . .	= $2\frac{2}{3}$
,, 24 ,, 30 ,,	. . . .	= $3\frac{1}{3}$
,, 30 ,, 36 ,,	. . . .	= 4

Mechanical Excavation.—Mechanical excavators may be any of three principal types: shovel, grab, or drag-line, operated by steam, petrol, or fuel-

FIG. 182.—SINGLE-CHAIN DREDGING GRAB ($\frac{1}{4}$ CU. YD. CAPACITY) WITH DISENGAGING HEAD.

oil engines, or by electric power, and may be arranged to run on rails or, for greater mobility, may be mounted on caterpillar tracks. Grabs are more efficient in loose or loosened material, but in hard material the cutting edge should be provided with teeth as in *Fig.* 182.

Various machines have been designed for special purposes, such as digging trenches, grabbing within cylinders, dredging under water, or surface digging in ballast pits or quarry work. Many machines can be used for more than one purpose ; for example, a unit, in addition to being used as a shovel, can be converted into a drag-line, drag shovel, skimmer, or grab, or the machine can be used as a crane. An example is a machine fitted with a $\frac{3}{8}$-cu. yd. shovel and which as a crane has a lifting capacity of $2\frac{1}{2}$ tons; this is operated by a 30-h.p. petrol-paraffin engine. Catalogues give possible performances, from which, making due allowance for site conditions, estimates of cost can be prepared.

The capacities of mechanical shovels vary from $\frac{1}{4}$ cu. yd. to 10 cu. yd. and even more, but machines of $\frac{3}{8}$ cu. yd. to $1\frac{1}{2}$ cu. yd. are convenient sizes for normal construction work. A feature of mechanical excavation is that the material is taken directly out of the ground and dumped into the wagons. To employ a machine at its highest efficiency sufficient empty wagons must always be at hand to take the spoil when the machine is working at full capacity. If an ample supply of wagons is available the cost of mechanical excavation may be reduced to a few pence per cubic yard. Net costs of a few pence per cubic yard are often obtained for very large quantities, but the lowest costs are only obtainable with the machine most suitable for the conditions of the job and the possibility of keeping it working continuously.

In considering the cost of excavation by machine the following data, based on records of actual contracts, may be of value. The time a machine is actually digging may be only between 60 and 70 per cent. of the total working hours. While digging in ordinary soft soil the average output per hour is from 30 cu. yd. for a $\frac{3}{4}$-cu. yd. machine (drag-line or bucket), 60 cu. yd. for a $1\frac{1}{4}$-cu. yd. machine, 90 cu. yd. for a $1\frac{3}{4}$-cu. yd. machine, and 100 cu. yd. for a 2-cu. yd. machine. For drag-lines and cranes mounted on caterpillar tracks fuel and oil consumption per 1,000 cu. yd. of excavation in moderately firm soils are as follows:

$\frac{3}{4}$-cu. yd. drag-line.—80 gallons of petrol and 20 lb. of lubricating oil; or 2 cwt. of fuel oil and 15 lb. of lubricating oil.

$\frac{3}{4}$-cu. yd. crane.—$3\frac{1}{2}$ tons of coal and 35 lb. of lubricating oil.

$1\frac{1}{4}$ and 2-cu. yd. drag-lines.—$3\frac{1}{4}$ cwt. of fuel oil and 30 lb. of lubricating oil.

$2\frac{1}{2}$-cu. yd. drag-line.—3 tons of coal and 35 lb. of lubricating oil.

The running cost of machines can be divided among repairs and renewals, labour operating the machine, and fuel and lubricants. Generally the running costs per cubic yard of excavation for steam-operated machines are higher than for petrol driven, which are again higher than for machines using fuel oil, although in any given instance the relative costs of the various types of fuel and the availability of spare parts would considerably affect the actual cost. Similarly the running cost per cubic yard of excavation for small capacity steam-driven drag-line, crane, or bucket excavators is higher than for large-capacity machines, although the reverse is true for fuel-oil machines.

For steam-driven machines, repairs, etc., represent 20 to 30 per cent. of the total running cost, labour 30 to 40 per cent., and fuel 40 to 45 per cent. For petrol-driven machines the proportions are approximately 30 per cent. repairs, 30 per cent. labour, and 40 per cent. fuel. For fuel-oil machines, the proportions are approximately 30 to 45 per cent. for repairs, 35 to 45 per cent. for labour, and 20 per cent. for fuel. In all cases small-capacity machines approach the lower limit for repairs and the upper limit for labour, and vice versa for larger machines.

Removal of Water.—To maintain an excavation dry it is usual to dig one or more sumps below the general level of the bottom. The water collecting in these sumps is pumped out. In large areas, shallow channels dug across the site and draining to a single sump will effectively and cheaply divert any water seeping through cofferdams or sheeting, or upwards through the bottom. Where the bottom of the excavation exposes an irregular rock surface, more than one sump may be necessary. For seepage water the pumping usually need only be intermittent, but when the flow into the excavations is considerable continuous day and night pumping may be essential. In removing water from excavations in which concrete foundations are being constructed it is necessary to arrange the drainage channels so as to avoid washing the cement from the freshly-deposited concrete.

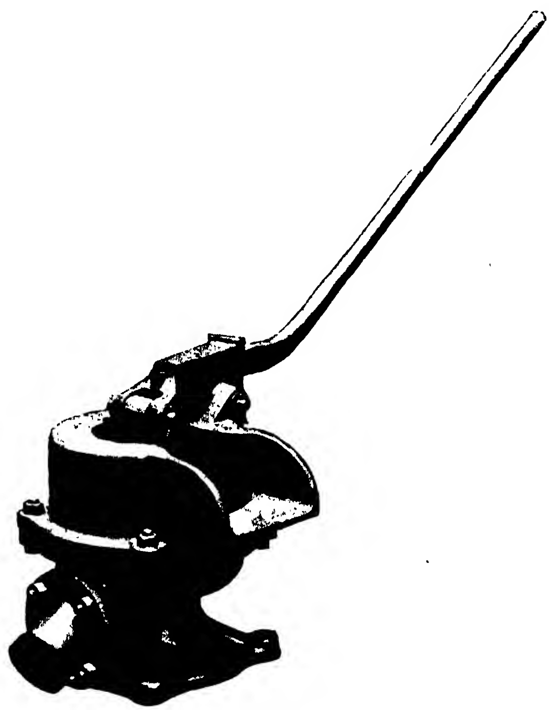


FIG. 183.—DIAPHRAGM HAND PUMP.

Pumps suitable for use on contracts are of three principal types, namely Pulsometer, reciprocating, and centrifugal. Reciprocating pumps are most common and can be either hand operated or driven by a petrol engine or electric motor or, in older models, by steam engine. The most common type of hand pump is that illustrated in *Fig.* 183 by means of which one man can pump water from depths up to 15-ft. The suction hose is 3-in. diameter and is usually armoured hose supplied in a length of 20 ft., the lower end being fitted with a cast-iron or brass strainer.

Portable power-driven pumps may be of the ram, diaphragm, or bucket types. A diaphragm pump with a 3-in. diameter suction pipe fitted with a $1\frac{1}{2}$-h.p. petrol engine will pump about 4,000 gallons per hour, but this type of pump, although an economical machine for free delivery and low suction heads, is not such a universally useful piece of equipment as a ram or a bucket-type pump, since with over 10-ft. suction the efficiency falls off to such an extent that it is not usually recommended for more than 15 ft. depths. For suction depths between 12 ft. and 28 ft., a bucket-type pump having a stroke of 3 in. and a 12-in. diameter barrel capable of pumping about 4,000 gallons an hour is a suitable machine, but if delivery is to be made more than a few feet above the level of the pump a ram-type pump (described below) is necessary. If a bucket-type pump is fitted with a gutta-percha packing ring it is possible to pump dirty and gritty water.

A horizontal-ram pump of the type illustrated in *Fig.* 184 with an 8-in. diameter ram and a 3-in. stroke driven by a 3-h.p. petrol engine and fitted with

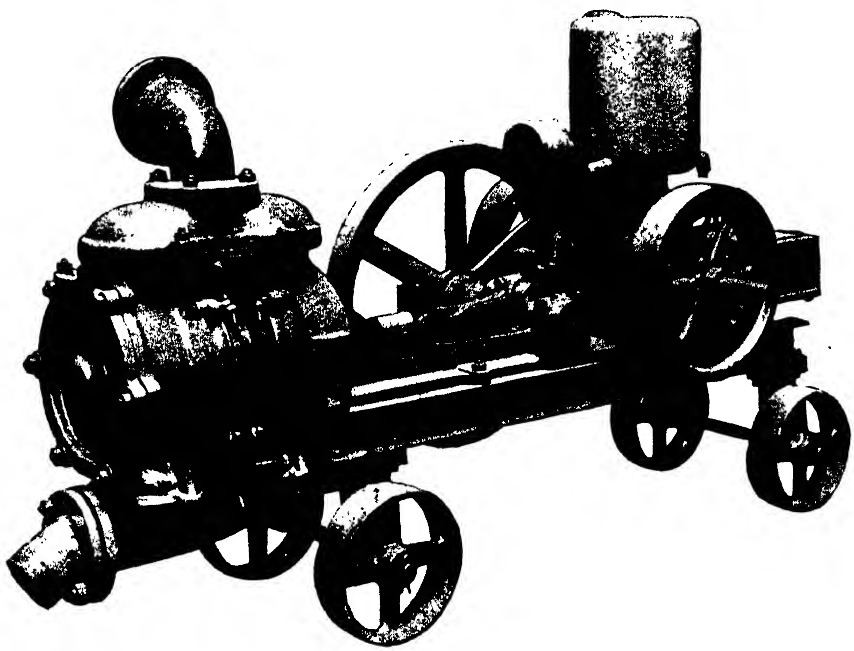


FIG. 184.—PORTABLE DOUBLE-ACTING POWER RAM PUMP.

3-in. suction and delivery pipe will pump about 3,000 gallons of water an hour against a combined suction and delivery head of 60 ft. The maximum suction limit is 30 ft. for all pumps of this type. With a 12-in. ram operated by a 4-h.p. petrol engine the capacity is increased to about 8,000 gallons an hour, the combined suction and delivery head being 35 ft. Such pumps continue to work even if drawing air due to the strainer being partly uncovered. Special pumps are designed to deal with capacities up to 20,000 gallons an hour and for pumping from depths beyond the normal suction limit.

Centrifugal pumps are the most economical machines for pumping water. A small pump of this type having a 3-in. diameter suction and driven by a $2\frac{1}{2}$-h.p. petrol engine will pump about 10,000 gallons an hour against a combined suction and delivery head of 10 ft., or 3,000 gallons per hour against a 30-ft. head. A medium-size pump with a 4-in. suction and 6 to 8-h.p. engine can pump 20,000 gallons an hour against a 25-ft. head, or 7,500 gallons against a 45-ft. head. A large centrifugal pump with a 5-in. suction and 8 to 10-h.p. engine will pump 12,000 gallons an hour against a 50-ft. head. When pumping water from excavations it is an advantage if the pump operates when the strainer is only partly submerged, and for this purpose a self-priming device is fitted to centrifugal pumps ; this attachment reduces the capacity of the pump below the quantities stated.

When continuous pumping is required to keep an excavation dry a Pulsometer pump is suitable if a steam supply is available. The steam pressure should not exceed 70 lb. per square inch. Among the factors that make these pumps useful for building contracts are the absence of moving parts and the ease with which they can be installed as the pump requires no fixing and can be simply suspended in the hole which is being drained. The maximum lift of a Pulsometer pump is about 80 ft., but the pump should be placed in such a position that the suction lift is about 6 ft. to 9 ft. If the total lift exceeds 30 ft. it is recommended that a back-pressure valve be provided immediately above the pump. A small pump with a 3-in. suction pipe and 2-in. delivery pipe will lift nearly 5,000 gallons an hour through 30 ft. Pulsometer pumps vary in size from this up to pumps with a 6-in. delivery pipe discharging 40,000 gallons an hour.

Excavating in Rock.—The excavation of rock by blasting requires special plant and experience. In soft rock each cubic yard excavated may represent 2 to 3 man-hours, in medium rocks 3 to 4½ man-hours, and in very hard rocks requiring drilling and blasting, 4½ to 6 man-hours. The cost of the drilling and blasting equipment may represent an additional 10 per cent. on the labour cost, while the explosives may represent an extra 20 per cent. The plant costs will naturally vary with the volume of rock to be dealt with, and these figures may fluctuate considerably due to restriction of the working face and according as the work is in the open or in tunnelling.

Blasting shots are usually placed at about 3-ft. centres, and the depth of each drift may be from 6 ft. in soft rocks down to 3 ft. in hard igneous rocks. With one free face the effective depth of the blast may be 3 ft., and perhaps 5 ft. if there are two adjacent free faces at right angles. The amount of rock shifted at each blast can be increased, and time can be saved, by using a series of graduated delayed-action detonators. The rate of drilling drift holes by machine may approach 20 in. per minute, although a practical average for estimating purposes is 10 in per minute. With a jack-hammer 5 in. per minute is possible, and with hand drills in granite the speed may be 5 in. to 10 in. per hour. Sharpening drilling tools is an expensive item. In granite the tools may require sharpening about every 10 in. to 12 in., the length of metal wasted being about one-tenth of the depth penetrated.

For rock transport by hand-operated wagons, one filler or loader is required for each three men at the rock face, and for each filler one wheeler is required for each 50-yd. run. Estimates for blasting costs must therefore include the labour and equipment for scaffolding, drilling, changing and sharpening drills, blasting (including explosive, detonators, fuses, wiring, etc.), power generation, and handling and removing spoil.

Timbering.

Whether timbering is required depends on the type of ground to be excavated and, in some soils, on the probable length of time for which the excavation will remain open. For excavation in shallow trenches in stiff clay or similar ground it is usually sufficient to provide poling boards, say 9 in. by 3 in. or 7 in. by 2 in. at intervals of about 6 ft. and strutted horizontally by 4-in. by 4-in. struts at 4-ft. vertical intervals. Dry hard clay may not require timbering, but an occasional strut bearing on a vertical board on each face of the trench is advisable as a safety measure against slips. In moderately firm ground, such as soft clay or large gravel, the poling boards may be at closer centres and bear on horizontal walings (say 9 in. by 3 in.) strutted apart. In loose soil continuous sheeting

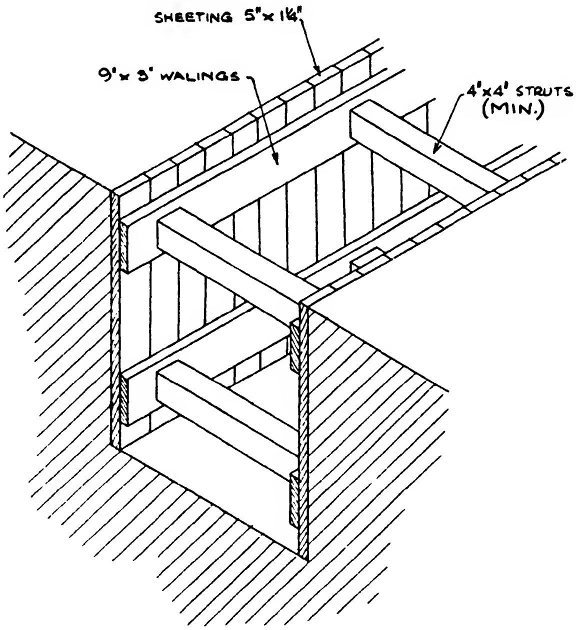


Fig. 185.—Timbering for Shallow Trench in Fairly Loose Soil.

of poling boards held by walings and struts may be required on both sides of the trench ; for a shallow trench the arrangement of timbering would be as shown in *Fig.* 185. The size of the struts depends on the width of the trench ; for trenches from 4 ft. up to 10 ft. wide, 1 in. per foot of width is a practical rule to determine the dimensions of square struts which are not braced or stiffened.

For shallow excavations in loose sand, a good method of timbering is shown in *Fig.* 186, where the sheeting runs horizontally, the boards being inserted separately and temporarily strutted as soon as the depth of digging permits. When, say, four boards are in position the temporary struts are replaced by a vertical timber strutted at the top and bottom, successive similar lifts of sheeting being carried to the bottom of the trench. If the sand is dry the sides of the

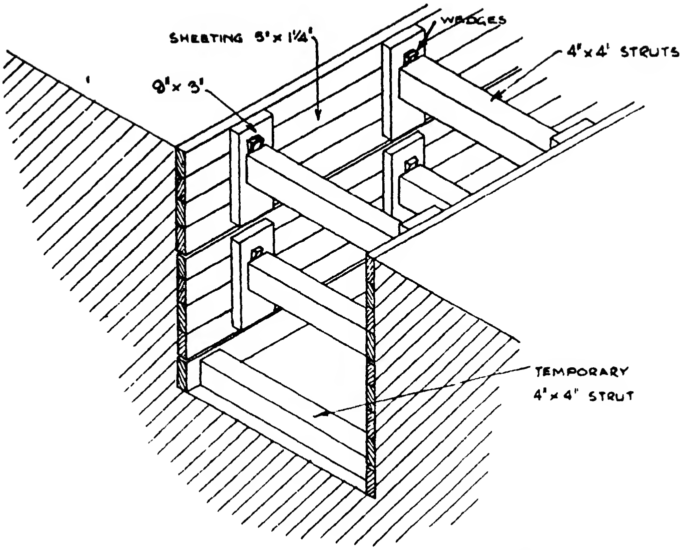


FIG. 186.—TIMBERING FOR SHALLOW TRENCH IN VERY LOOSE SOIL.

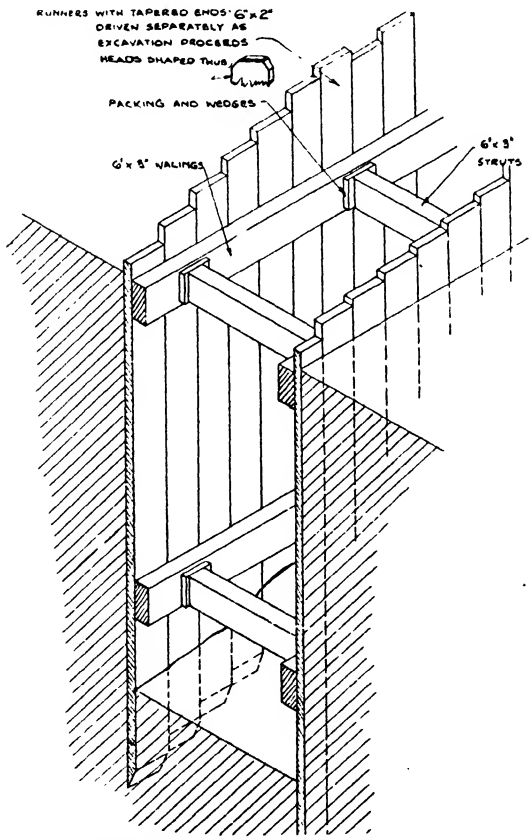


FIG. 187.—TIMBERING FOR DEEP TRENCH IN LOOSE SAND.

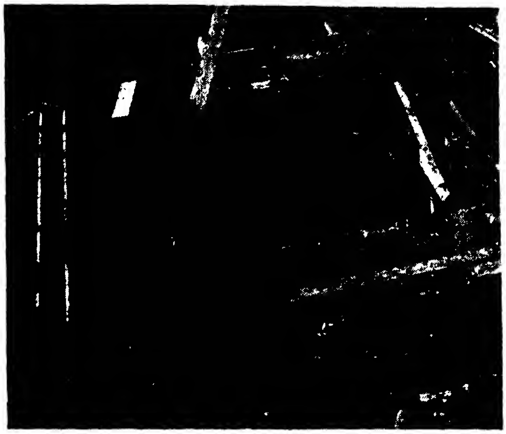


FIG. 188.—TIMBERING FOR SMALL RECTANGULAR EXCAVATION.

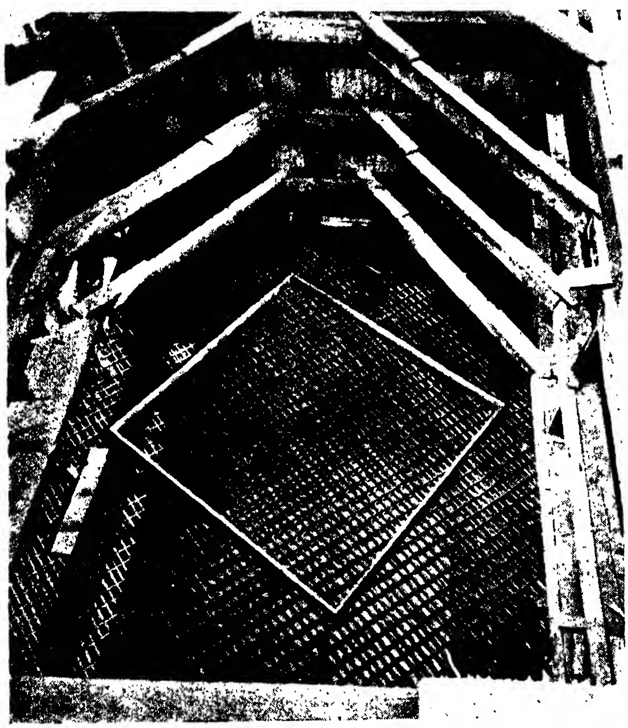


FIG. 189.—CORNER TIMBERING FOR COLUMN EXCAVATION.

trench can be vertical, but if they are wet the sides should be battered at, say, 1½ in. per foot. It is important to wedge the boarding in position, the wedges being frequently tightened, since in dry sand the material may trickle out from behind the upper boards when excavation is proceeding below. For deep trenches in loose wet soils, a convenient method of timbering is that illustrated in *Fig.* 187.

Long vertical runners are driven by hand hammer a foot or so at a time into the ground as the excavation proceeds, these boards bearing on a system of walings and struts which are inserted as the excavation deepens. The runners should be driven close together, any leaky joints being caulked with straw or sacking. The same principle can be applied to timbering small rectangular excavations for isolated foundations, the transverse struts being dispensed with. If the side

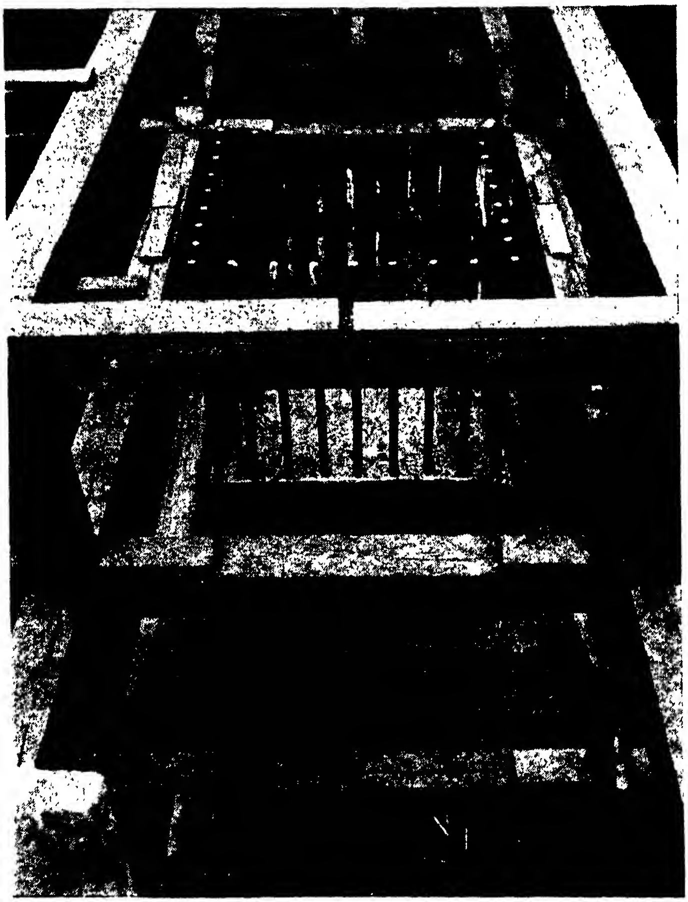


FIG. 190.—STRUTTING FOR COLUMN EXCAVATION.

of the hole is long it may be necessary to stiffen the walings by struts across the corners as illustrated in *Fig.* 188. This method avoids the use of direct cross struts which might interfere with the construction of piers or columns in the hole. Another view of similar timbering is given in *Fig.* 189, where the foundation reinforcing mat is shown in place and a template for the superimposed column is indicated. Where the excavation is of such a size that cross-corner struts are insufficient, direct cross struts should be arranged as shown in *Fig.* 190 to clear the base and shaft of the column that is to be built within the excavation.

When the base has been concreted the reinforcement in the pier or column is fixed before erecting the column shuttering. *Fig.* 191 shows the parts of the shuttering ready for assembling in the excavation.

When the excavation is of such a depth that more than two walings are required the runners must be kept clear of the walings during driving, otherwise the resistance to driving would render driving by hand or by light power-

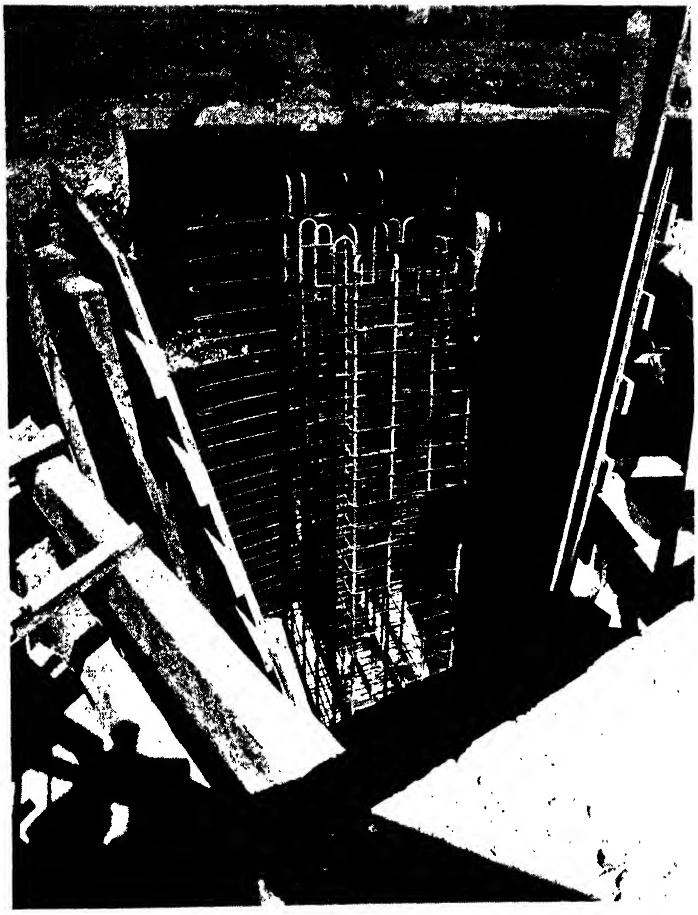


FIG. 191.—COLUMN SHUTTERING PARTLY IN EXCAVATION.

hammers impracticable. When a runner has been driven to the required depth, wedges are driven between it and each waling as shown in *Fig.* 192. This ensures that the boards bear hard against the earth backing. The tops of the runners should be shaped and chamfered as shown in *Fig.* 187 to prevent their splitting while they are being driven. In particularly wet soils the runners may not be driven tightly enough to prevent leakage of water into the excavation, in which case they would be replaced by interlocking steel piles as shown in *Fig.* 193,

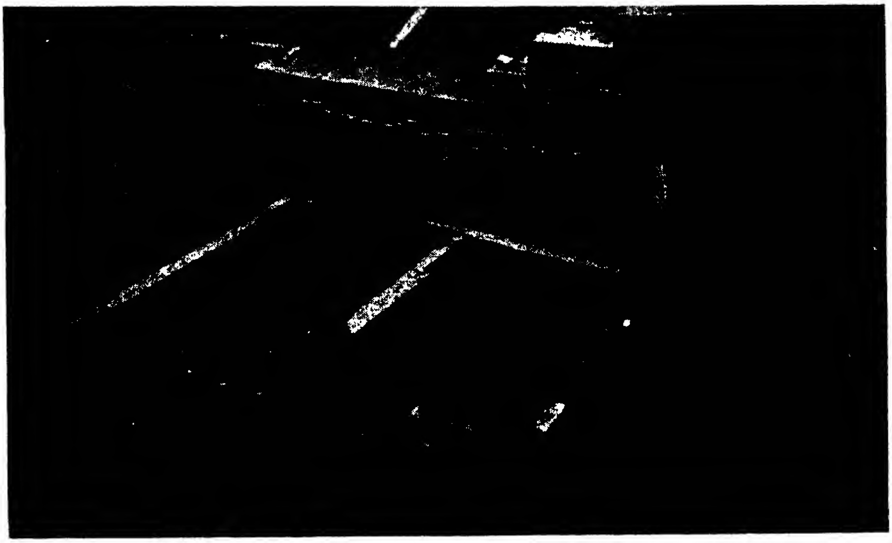


FIG. 192.—WEDGING-OUT RUNNERS.

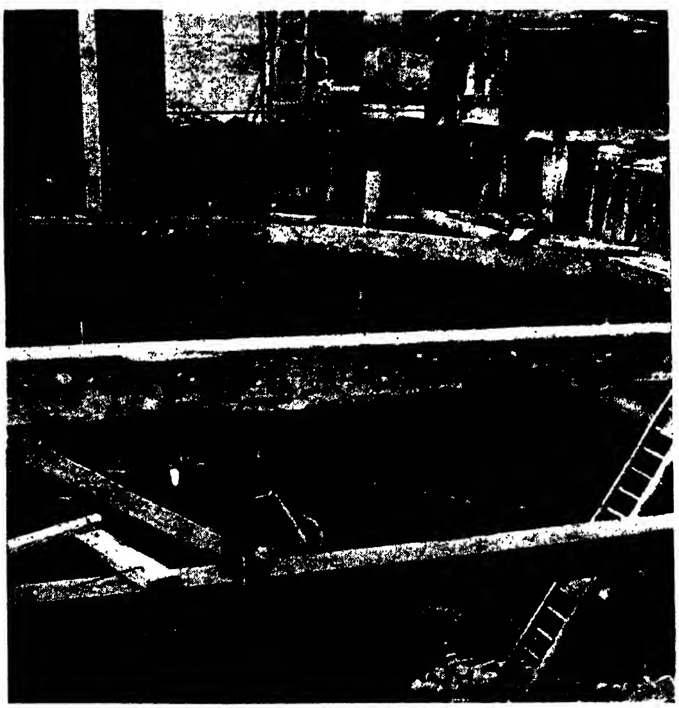


FIG. 193.—STEEL SHEET PILING WITH TIMBER WALINGS.

wedges, walings and struts being provided as with timber sheeting, except that the walings might be at wider spacing on account of the higher strength of the steel sheeting.

For trenches in moderately dry and compact ground, an arrangement of poling boards in 2-ft. 6-in. lengths, each row with a centre waling and strutted at 5-ft. to 6-ft. intervals can be used. Each frame of timbering is put into position as the excavation deepens.

If cleats are attached to the walings below the end of each strut the latter can be used for supporting a series of platforms on which the excavated earth is successively thrown to bring it to the surface. These platforms are not required if the earth is loaded directly into crane skips lowered between the struts into the trench bottom.

For wide and deep excavations in water-bearing ground heavier timbering is required, consisting of substantial timber or steel sheeting supported by 8-in., 10-in., or 12-in. square walings and struts arranged as indicated diagrammatically in *Fig.* 194. The transverse struts across the short way of the trench are usually

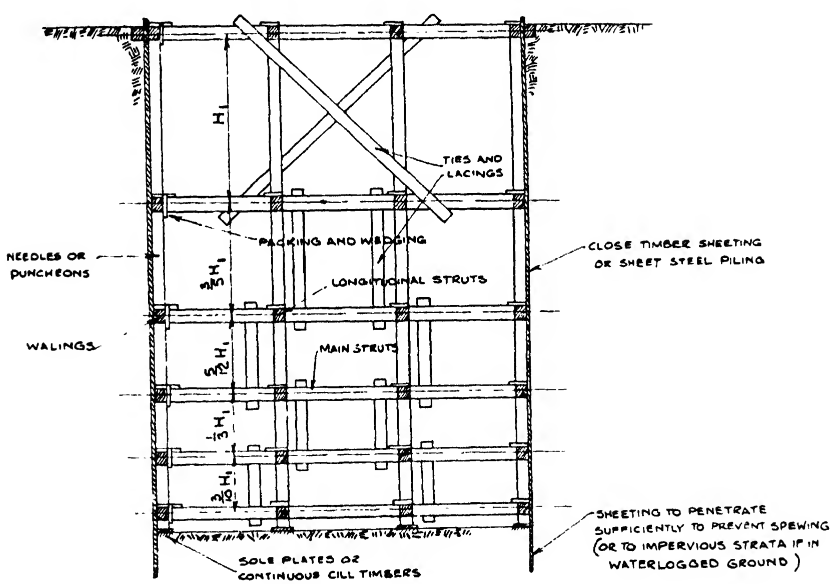


FIG. 194.—TIMBERING FOR DEEP EXCAVATION.

in a single length between opposite walings, and the longitudinal struts in short lengths wedged between the transverse struts. A detail of timbers at the intersection of struts is shown in *Fig.* 195, the struts running across the illustration being continuous. The longitudinal struts, together with the vertical members, termed puncheons or needles, assist in increasing the buckling resistance of the transverse struts. Vertical timbers should also be inserted at the ends of the struts to pick up the weight of the walings and struts. Sole-plates or continuous sill timbers are required to provide a bearing for the packings below the bottom frame of struts.

The sheeting may be either 3-in. close-driven timber boarding or steel sheet piling, the latter being trough-section, rolled joist section with clutches, or stiffened steel plates. The strength of steel sheeting of the types stated decreases for equal weights in the order given, hence more walings will be required for steel-plate sheeting than for troughed sheeting. Whatever form of sheeting is used, it must be driven into the ground sufficiently far below the bottom of the excavation to anchor the end of the sheeting and to prevent the earth "spewing." With

FIG. 195.—INTERSECTION OF STRUTS.

sheeting driven insufficiently far there is a danger in soft soils of the bottom of the excavation "blowing up" due to the unequal heads of earth inside and outside the sheeting. Apart from the fact that the excavation must be continuously cleared, there is also the risk that the supports of the internal timbering may be rendered insecure, thus leading to the total collapse of the timbering and sheeting. The theoretical depth to which sheeting should be driven to prevent this action is given by

$$h = \frac{k^2 H}{1 - k^2},$$

where $k = \dfrac{1 - \sin \theta}{1 + \sin \theta}$

H = the depth of the excavation (ft.), and

θ = the natural angle of repose of the ground (see *Table* XX).

If the calculated value is increased by 50 per cent. to ensure that more than the bare theoretical depth is provided, the minimum depth (h) to which sheeting should be driven in common types of soil is :

	θ (deg.).	h (ft.).
Ordinary dry earth	40	$\frac{H}{14}$
Dry sand	35	$\frac{H}{9}$
Moist sand	25	$\frac{H}{3{\cdot}5}$
River mud	15	$\frac{H}{1{\cdot}25}$

When the excavation is below water level or in water-bearing strata the sheeting must be driven sufficiently deep to penetrate an impervious stratum, such as clay (see "Cofferdams," page 286).

The spacing of the walings depends on the nature and wetness of the soil and is further controlled by the strength of the sheeting and by the size of waling used. Conditions are seldom so favourable that the vertical spacing of walings for deep excavations can exceed 5 ft., except between the upper tiers; in water-logged soils the intervals may be less. Such close spacing makes the handling of the lower settings of timber difficult if the single pieces are long; short lengths spliced together are therefore more convenient. The spacing of the walings should be decreased as the depth below the surface increases, and, having established an initial spacing between the top and second tiers, the values given in *Fig.* 194 for the spacing of successive tiers will ensure equal loading being carried by each waling if the external pressure on the sheeting increases proportionately with the depth, as in the case of water-bearing soils. The maximum initial spacing H_1, if dependent upon the strength of the sheeting, can be obtained as follows:

The bending moment on the top lift of sheeting is

$$\frac{pH_1^2 \times 12}{2 \times 7\cdot5} = \tfrac{4}{5}pH_1^2 \text{ in.-lb. per foot,}$$

if the sheeting is continuous over at least three walings; where p (lb. per square foot) is the pressure on the sheeting at the level of the second waling from the top. If the sheeting is subject to water pressure $p = 62\cdot4H_1$ and if the soil is dry and loose $p = 27H_1$. Other conditions of damp and wet soils would lie between these limits, but if the ground is water-logged the full water pressure ($62\cdot4H_1$) should be used.

If $M =$ the safe moment of resistance of the sheeting in in.-lb. per foot width, the value of H_1 (ft.) must not be less than

$$\sqrt{\frac{5M}{4p}}.$$

or if $$p = k_1H_1$$

$$H_1 \nless \sqrt[3]{\frac{5M}{4k_1}}.$$

The value of M depends upon the material and thickness of the sheeting; if steel sheet piling is used the value will be found in the manufacturer's catalogue. If timber sheeting of thickness d in. is used the value of M can be taken as $3{,}360d^2$ in.-lb. per foot width, using a working stress of $\frac{3}{4}$ ton per square inch for sound timber.

Take as an example an excavation in waterlogged ground, that is $k_1 = 62\cdot4$. Using 3-in. timber sheeting,

$$M = 3{,}360 \times 3^2 = 30{,}240 \text{ in.-lb. per foot.}$$

$$H_1 = \sqrt[3]{\frac{5 \times 30{,}240}{4 \times 62\cdot4}} = 8 \text{ ft. } 6 \text{ in.}$$

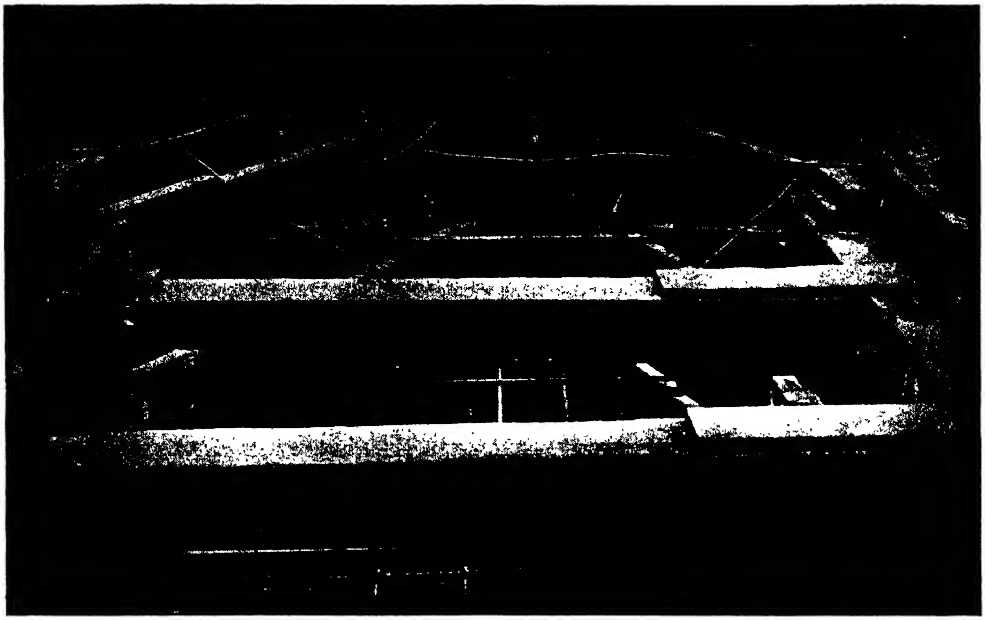


Fig. 196.—Timbering in Narrow Excavation.

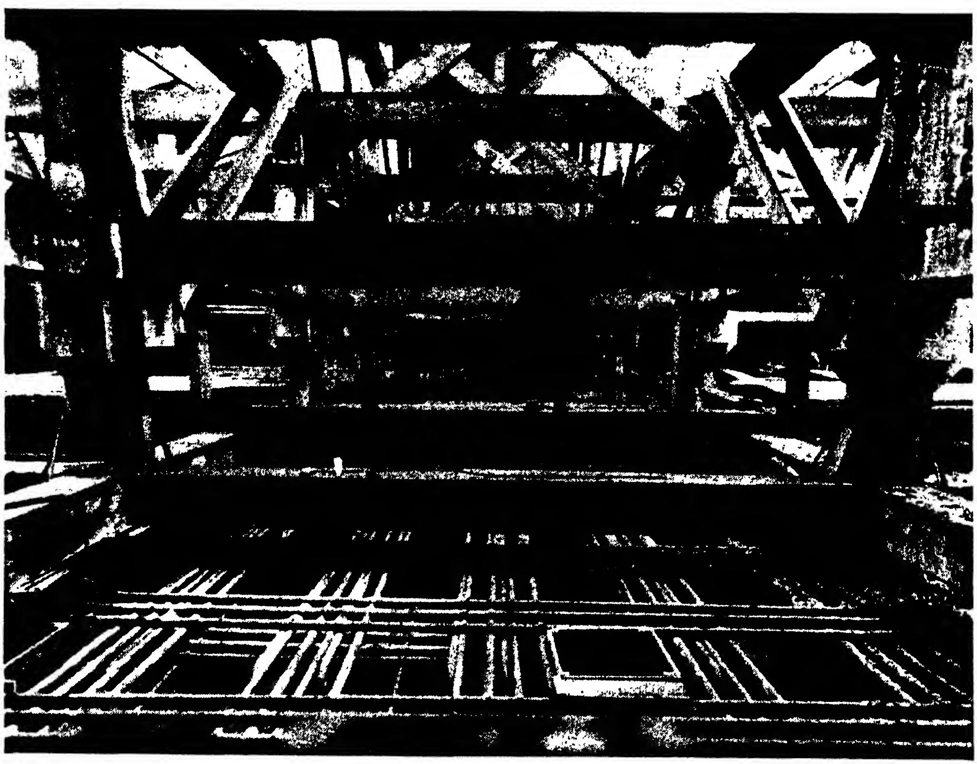


Fig. 197.—Heavy Timbering for Deep Excavation.

Some examples of heavily-timbered excavations are illustrated in *Figs.* 196. 197, and 198, each presenting features of interest. The timber sheeting in *Fig.* 196 is placed in short lengths, each lift, inserted as excavation proceeds, being supported by a setting of walings and transverse struts. This method is suitable in ground that is not waterlogged. As the excavation is not wide, longitudinal struts are not required, support to the walings across the end of the trench being provided by diagonal corner struts. In place of the blocks and wedges commonly provided to ensure that the transverse struts bear firmly against the walings, screw jacks are employed.

By suspending the lower settings of timbers illustrated in *Fig.* 197 an uninterrupted space is obtained below the bottom setting to allow the construction of a heavy reinforced concrete raft to proceed unhindered. The lower setting is suspended from the upper bolted timber trusses, while the second setting from the bottom is supported on the vertical members by bearing on heavy cleats. The bottom layer of horizontal struts is suspended by steel dogs.

The excavation illustrated in *Fig.* 198 is lined with steel sheet piling sup-

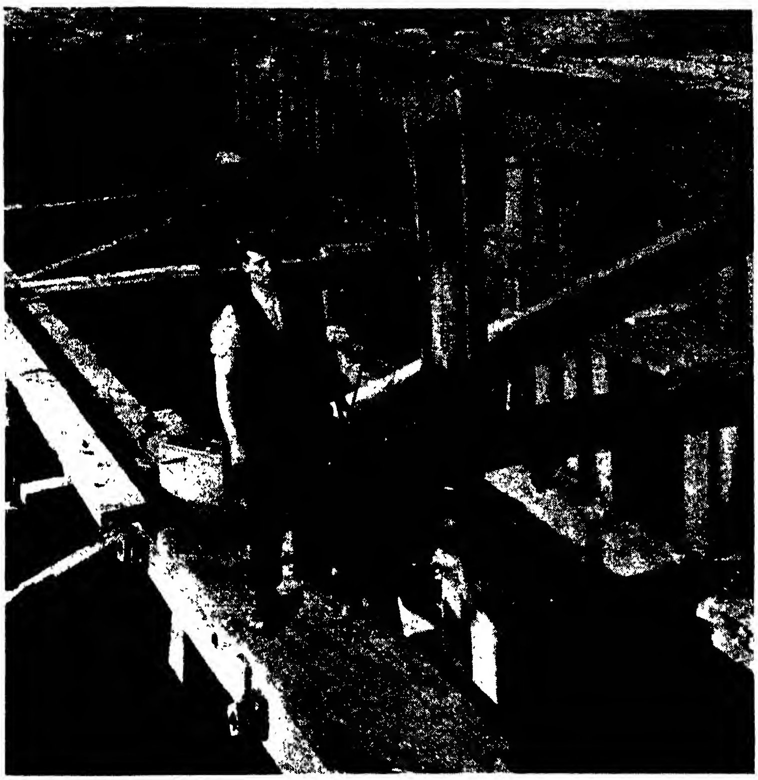


Fig. 198.—Sheet Piling for Excavations.

ported by timber walings and struts. Sheeting of this kind is usually driven and extracted by a light pneumatic or steam double-acting hammer placed directly on the pile-head, a light derrick or crane being used to handle the hammer as in *Fig.* 199. In the illustration in *Fig.* 198 the sheeting is being driven by a percussion ram. These machines are driven by benzol motors, and are also largely used for ramming refilled ground. Heavy-section sheeting in long lengths requires to be driven by a heavy drop hammer or a single-acting steam hammer, as illustrated in *Fig.* 200, this type of hammer also being used for extraction.

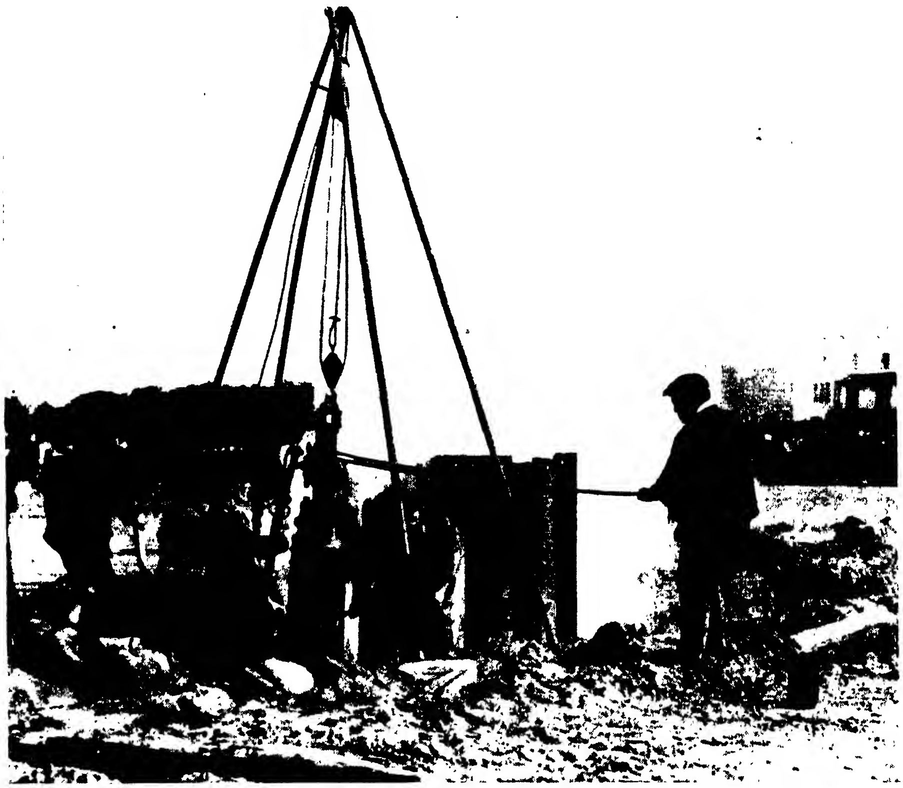


FIG. 199.—DRIVING SHEET PILING WITH PORTABLE HAMMER.

When timbering an excavation in the face of an earth bank a complete system of horizontal strutting cannot be provided and diagonals, as shown in *Fig.* 201, have to be provided. The size of the sheeting, the depth to which it must be driven to prevent spewing, and the spacing of the walings are determined as previously described, except that the additional pressure due to the slope (if any) of the bank behind the sheeting must be allowed for. If the ground slopes upward from the back of the excavation at an angle equal to the natural angle of repose, the horizontal pressure on the sheeting may theoretically be $2\frac{1}{2}$ times as great in ordinary earth or loose sands as if the ground surface is level. For any other slope the pressure from dry loose soil is on the safe side if taken as

$$p_h = [27 + 1{\cdot}15\phi]H_1 \text{ lb. per square foot}$$

at a depth of H_1 ft. below the top of the sheeting; ϕ represents the slope of the bank measured in degrees.

The total pressure transmitted to the walings from the sheeting is transferred through the horizontal and diagonal struts to a sole-piece at the opposite side of the excavation. This sole-piece must be designed to distribute the pressure over an area sufficiently large to prevent movement by compression of the soil.

FIG. 200.—DRIVING SHEET PILING WITH HEAVY HAMMER.

The safe pressure that can be exerted on a vertical face of ordinary earth, sand, or similar fairly loose strata is about 200 lb. per square foot for each foot of depth below the surface, up to a limiting pressure corresponding to the normal safe bearing pressure on the particular class of soil. If the sole-piece is inclined, as in *Fig.* 201, the length should be such that the vertical height multiplied by the width and multiplied by 200 lb. per square foot should not be less than the total horizontal force on the walings. If there is no bank or insufficient bearing area for the sole-plate the reaction to the force in the struts must be taken on stakes driven well into the ground.

The thrust in the diagonal struts is accompanied by a tendency for the struts to lift upwards. The value of this upward force is $\frac{a}{b}.F$ where a and b are the dimensions (*Fig.* 201) representing the slope of the strut and F is the horizontal force which the strut resists (that is, the reaction from the waling). The upward force is in part resisted by the weight of the timbering and in part by the friction of the timbering against the earth. In dry soils the total value of the friction can be assumed as one-third of the total earth pressure acting on the sheeting; this resistance is available to assist in resisting the sum of the

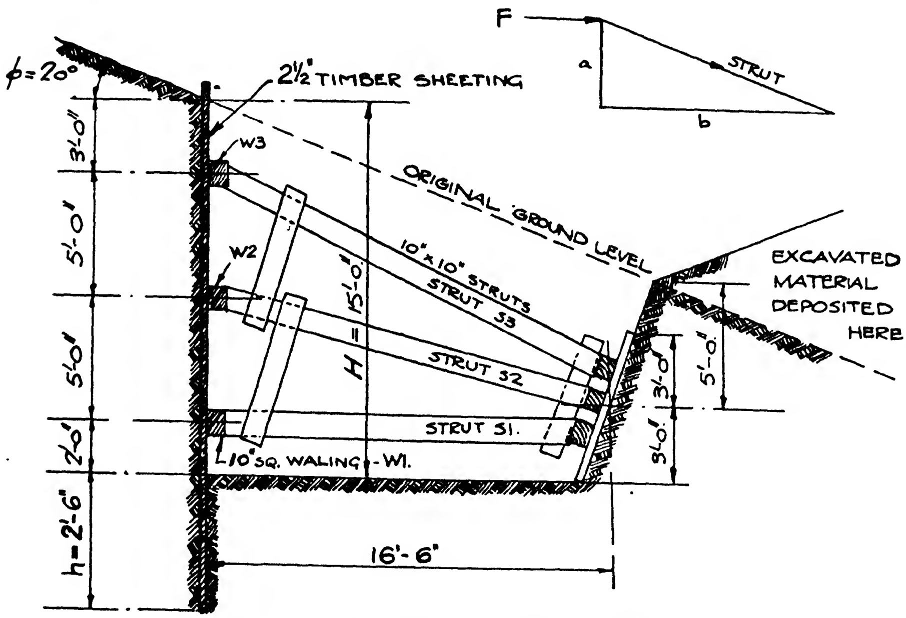


FIG. 201.—EXCAVATING IN FACE OF BANK.

upward forces on all the diagonal struts if the latter are laced together as shown in *Fig.* 201. In wet clay the frictional resistance may be nil. If the available resistance, together with the weight of the timbering, is insufficient to resist the upward forces, additional loading in the form of bricks, rails, etc., would have to be added to prevent the timbering from rising. The following example indicates the method of designing the timbering for an excavation, 15 ft. deep, in the face of a bank, as shown in *Fig.* 201.

Assuming dry loose sandy soil, the minimum depth h to which the sheeting should be driven in accordance with the expression given previously is $\frac{15}{9} = 1{\cdot}67$ ft. below the bottom of the excavation. Making some allowance for the fact that the ground surface is not level, this depth will be made 2 ft. 6 in. The horizontal pressure at any depth H_1 with $\phi = 20$ deg. is

$$p_h = [27 + (1{\cdot}15 \times 20)]H_1 = 50H_1.$$

To determine the thickness of the vertical sheeting, if walings are provided at 5-ft. centres, the critical bending moment will occur between the walings $W1$ and $W2$. The average depth of earth behind this span is 10 ft. 6 in. and the average earth pressure is $50 \times 10{\cdot}5 = 525$ lb. per square foot. If the sheeting is driven in a single length and is therefore continuous over more than two spans (three supports) the bending moment on a 1-ft. width is

$$\frac{525 \times 5^2 \times 12}{12} = 13{,}100 \text{ in.-lb.}$$

Since the resistance moment of 2-in. boarding is $3{,}360 \times 2^2 = 14{,}440$ in.-lb. per foot width, 2-in. boarding will be sufficient, but if the sheeting is in short lengths the bending moment would be 50 per cent. greater, that is about 20,000 in.-lb. The resistance moment of $2\frac{1}{2}$-in. boarding is $3{,}360 \times 2{\cdot}5^2 = 21{,}000$ in.-lb. per foot width. It would be preferable to provide $2\frac{1}{2}$-in. timber.

If the struts are provided at 7-ft. 6-in. intervals the calculation for the lower waling $W1$ proceeds as follows: The average earth pressure on this waling is, say, $50 \times 13 \times 5 = 3{,}250$ lb. per foot run, the total load on the waling being therefore $3{,}250 \times 7{\cdot}5 = 24{,}400$ lb. Allowing for the walings to be placed in short lengths (about 8 ft. minimum), the bending moment is

$$\frac{24{,}400 \times 7{\cdot}5}{8} \times 12 = 272{,}000 \text{ in.-lb.}$$

The moment of resistance of a 10-in. square timber at a working stress of $\frac{3}{4}$ ton per square inch is $\frac{3}{4} \times 2{,}240 \times \dfrac{10 \times 10^2}{6} = 280{,}000$ in.-lb. The upper walings do not require to be so heavy since their loading is less, but for convenience the same section of timber would be used. The load on waling $W2$ is

$$50 \times 8 \times 5 \times 7{\cdot}5 = 15{,}000 \text{ lb.,}$$

and on $W3$ the load is $50 \times 3 \times 4 \times 7{\cdot}5 = 4{,}500$ lb.

The force in strut $S1$ is the reaction from waling $W1$, that is 24,400 lb., and the length of the strut is about 15 ft. The load along the inclined struts is the reaction from the corresponding walings multiplied by the factor $\dfrac{1}{b}\sqrt{a^2 + b^2}$. Thus the load along strut $S2$, which is about 15 ft. long, is

$$\frac{15{,}000}{15{\cdot}5}\sqrt{4^2 + 15{\cdot}5^2} = 15{,}800 \text{ lb.}$$

The load along strut $S2$, which is about 17 ft. long, is

$$\frac{4{,}500}{16}\sqrt{8^2 + 16^2} = 5{,}000 \text{ lb.}$$

From *Table* V (Chapter I) it will be seen that a 10-in. square strut of these lengths is ample to sustain the axial loads on the struts.

The total horizontal pressure on the sole-piece is the combined horizontal reaction from each of the walings, that is $24{,}400 + 15{,}000 + 4{,}500 = 43{,}900$ lb. The average depth of earth behind the sole-piece is 5 ft., hence the total resistance per foot width is 200 lb. × 5 ft. × 6 ft. = 6,000 lb. The width of the sole-piece required is therefore $\frac{43,900}{6,000}$, say, 7 ft. 6 in. As this corresponds to the spacing of the struts, the bearing must be continuous and would consist of three longitudinal 10-in. square walings bearing on 2½-in. close-spaced boarding.

It only remains to check the resistance to uplift due to inclined struts.

$$\text{Due to strut } S2 \text{ the uplift is } \frac{4}{15\cdot5} \times 15,000 = 3,860 \text{ lb.}$$

$$\text{Due to strut } S3 \text{ the uplift is } \frac{8}{16} \times 4,500 = 2,250 \text{ ,,}$$

$$\text{Total} = 6,110 \text{ ,,}$$

The total weight of sheeting, walings, and struts in a 7 ft. 6 in. length of trench is 70 cu. ft. at 50 lb. per cubic foot = 3,500 lb. This will in part offset the uplift of 6,110 lb., the balance of the latter being easily provided by resistance between the sheeting and the earth, since, if dry, the limiting value of this friction would be about $\frac{1}{3}$ × 43,900 = 14,600 lb.

Cofferdams.

To enable constructional work below water level to proceed in the dry, cofferdams are built around the site and the area inside the dam is pumped dry. The simplest type of cofferdam is a bank of clay or a wall of sandbags. If the latter is used the wall should be two bags thick, the bags not being filled tight, so that each may accommodate itself to the shape of those next it and

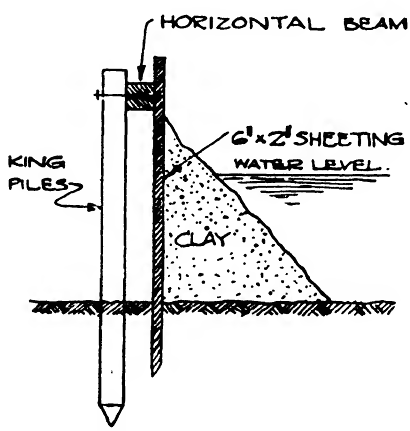


FIG. 202.—SINGLE-SHEET COFFERDAM.

seal the interstices. This kind of construction is suitable for heads of water up to 3 ft., and the little water that seeps through it can usually be dealt with by intermittent pumping.

For moderate heads a single row of closely-driven sheeting can keep out water, and if the timbers are wrought the water will swell the wood so that the gaps are practically sealed. If the sheeting is not closely driven, watertightness can be obtained by dumping clay behind the wall as shown in *Fig. 202*. In this illustration the sheeting is stiffened by bearing against a horizontal beam carried on the head of a series of well-driven timber king piles. For such construction the sheeting could be 3-in. thick, the beam and the piles being 10-in. square section and the latter driven at 10-ft. or 12-ft. centres.

More effective for greater heads of water is a clay-puddle wall consisting of a double row of sheeting, the space between the sheeting being packed with well-rammed puddle clay. If the bed of the stream is hard enough to prevent vertical timber sheeting being driven, the methods illustrated in *Figs.* 203 and 204 can be adopted. When the depth of water permits wading a shallow trench can be dug in the stream bed. A line of 4-in. by 4 -in. stakes is driven and horizontal

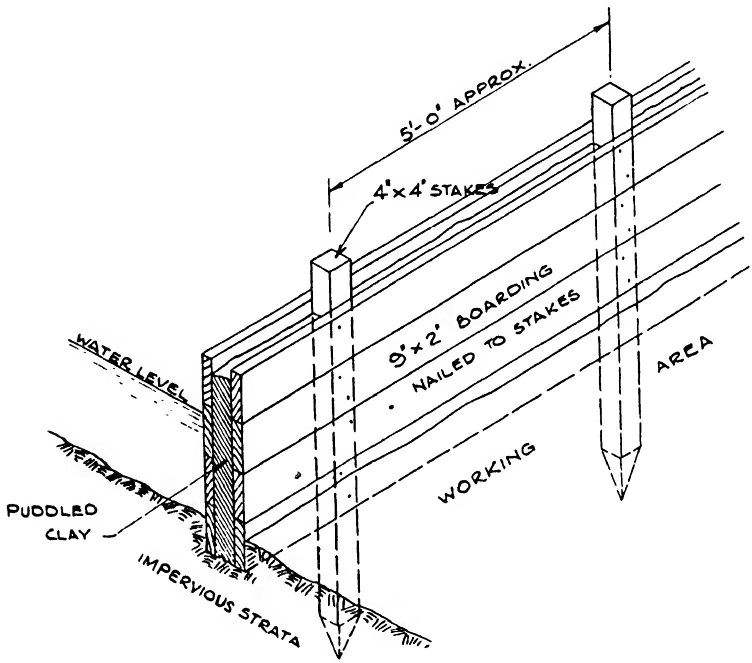


FIG. 203.—COFFERDAM FOR SHALLOW WATER.

boards are merely nailed to the stakes as shown in *Fig.* 203, an arrangement that is suitable for about 2-ft. of water. With depths of 3-ft. and upwards of water greater lateral stability is required and a double row of stakes is driven as shown in *Fig.* 204. The stakes can be timber or, when the ground to be penetrated is hard, pointed steel angles or other sections. The illustration shows the heads of the stakes secured by a timber batten or by twisted wire ties, but mild steel straps, bolts, or angles can also be used for this purpose. The pressure from the rammed puddle clay keeps the sheeting in close contact with the stakes, and the tie at the top prevents the stakes from spreading.

With depths of water exceeding 10 ft. substantial lateral support is required for the sheeting, and alternative methods are shown in *Fig.* 205. At (*a*) two rows of timber king piles are driven, the piles being spaced at about 10-ft. centres longitudinally and 4 ft. to 5 ft. laterally with the heads of opposite pairs of piles connected by a 9-in. by 3-in. timber tie. A little above water level a double

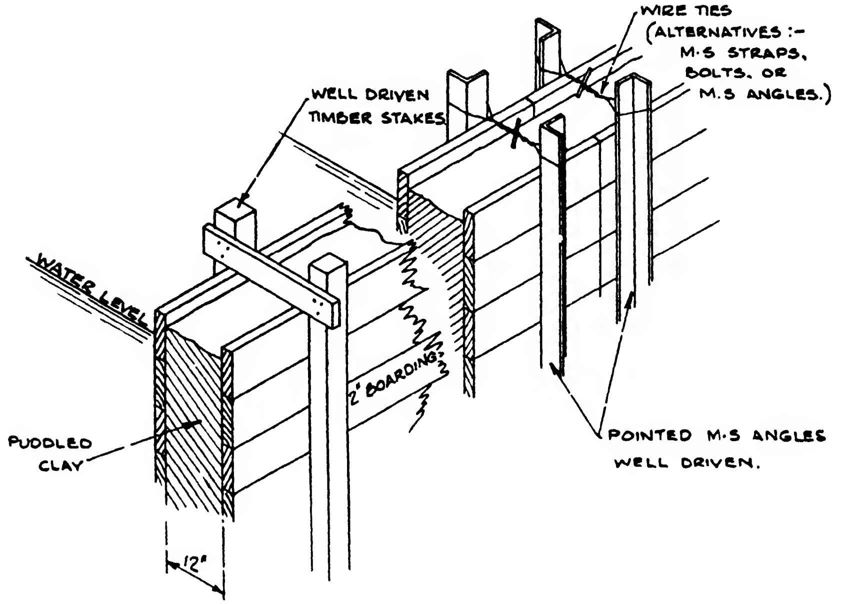


Fig. 204.—Cofferdam for Moderate Depths of Water.

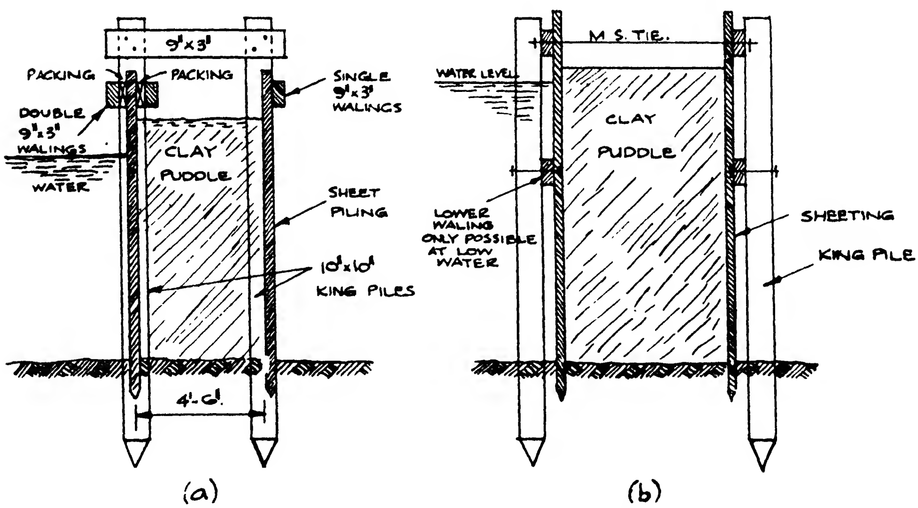


Fig. 205.—Alternative Designs for Cofferdams.

waling is bolted to the row of piles on the water side and a single waling on the opposite row of piles. The 9-in. by 3-in. vertical timber sheeting is driven between the piles as shown, and clay puddle is deposited between the two rows of sheeting.

Packings should be placed between the waling and the sheeting on the water side, but on the opposite side ramming the clay puddle will be sufficient to ensure contact at the top of the sheeting.

In the alternative at *Fig.* 205 (*b*), the sheeting is driven inside the rows of king piles, the walings bolted to the latter being between the piles and the sheeting. Steel ties passing through the walings at 10-ft. intervals connect the two lines of sheeting. The sheeting on the water side of the dam should be attached to the walings, but no attachment is required between the sheeting and the walings on the opposite face.

The cofferdam should penetrate the river bed to a sufficient depth to obtain an effective seal and sufficiently far below any excavation that may take place within the dam to prevent spewing. Although a penetration of a few inches may be sufficient in the case of shallow water and compact or clayey ground, several feet of penetration will be necessary in soft soils. In this case sheeting is driven down to an impervious stratum, or, if this is not within practicable reach, the sheeting is driven sufficiently far to reduce the inflow of seepage water to limits that can be dealt with by pumping.

Timber sheeting can only be used for moderate depths, and for large cofferdams or for driving into hard ground a single row of steel sheet piling is more suitable. For deep dams, sheet piling construction will be similar to that described for timbering in soft waterlogged soils, walings and struts being provided to support the sheeting. In most cases the joints in the steel sheeting will be tight enough to prevent a considerable inflow of water, although the vibration due to any pile driving needed within the area confined by the dam may be sufficient at times to cause leakage. In some cases this leakage can be stopped by depositing ashes in the water adjacent to the joints requiring sealing. In other cases seepage can be reduced by inserting thin strips of soft wood in the outer joints of the clutches of the steel sheeting.

The method of dealing with corners in sheet pile dams depends on the section of the sheeting used. Manufacturer's catalogues give details of corners and descriptions of driving and extracting equipment.

Where cofferdams are driven to rock having an irregular surface, special means of sealing the contact between the sheeting and the rock are necessary. Cement grout forced down outside to the foot of the sheeting before the water is pumped out makes an effective seal.

Cofferdams and other means of diverting streams during the construction of culverts are dealt with in a later chapter.

Cost of Excavation.

The following examples show in detail the various factors that must be included in analyses of the total cost of excavations where manual or mechanical means are employed.

Example (i).—Excavating shallow trenches by hand; spoil loaded into lorries by hand; no timbering or pumping.

Six men loosening fairly firm ground and throwing out average ½ cu. yd. each man per hour = 3 cu. yd. per hour measured in solid, or, say, 4 cu. yd. loose. Men loading lorries can handle an average of 1 cu. yd. of loose soil per hour, hence four loaders will be required to deal with the output of six men digging.

		s.	d.
Total number of men = 10 at 1s. 1d. per hour	=	10	10
Add 33⅓ per cent. for supervision, small tools, insurance, head office and general costs, profit, etc.	=	3	7
		14	5

This represents the cost of excavating 3 cu. yd. in the solid, thus the unit rate is

$$\frac{14s.\ 5d.}{3} = 4s.\ 10d. \text{ per cubic yard.}$$

If the excavation is in fairly soft soil and is confined to the surface of the site, say not more than 12 in. deep, the excavator may be able to fill barrows, jubilee wagons, or similar containers direct and the output of each man may be ½ cu. yd. per hour, resulting in a net cost of 2s. 2d. per cubic yard, or an overall cost of 3s. 0d. per cubic yard.

These costs cover the work up to the stage of loading into wagons for disposal of the spoil. The cost of disposal depends on the cost of hiring lorries and the distance of haul. Unless the contractor owns lorries this work is usually carried out by sub-contract.

EXAMPLE (ii).—An excavation 75 ft. long by 18 ft. wide by 20 ft. deep in waterlogged ground; steel sheet piling around the sides with timber walings and struts; excavating by hand, and loading into skips lifted by derrick and emptied into lorries.

The total volume to be excavated is 1,000 cu. yd. measured in the solid. The rate of working will depend on the number of men that can work in the space available at the bottom of the excavation. For economy in timbering and sheeting, the excavation can be carried out in two equal sections, it being assumed that subsequent construction permits the release of the material from the first section for use in the following section. Thus the working area will be 37 ft. 6 in. by 18 ft., the free space in which is interfered with by the timbering. This would allow four groups of three men to dig and fill the buckets. At the beginning of the excavation for each section more than twelve men may be accommodated, while towards the end of each section three or four men only may be usefully employed in digging. Thus twelve may represent the average number of men digging and filling, with an additional man attaching and detaching crane slings from the buckets. Each man may loosen, dig, and load into the skip an average of ¾ cu. yd. an hour, measured in the solid. At the beginning of the excavation this rate may be exceeded, whereas when digging among the lower settings of timber a smaller output is to be expected. Thus each group of men will excavate an average of $3 \times \frac{3}{4} = 2\frac{1}{4}$ cu. yd. an hour, which will fill three 1-cu. yd. skips with loose material, that is one skip will be filled by each group in twenty minutes. To deal with the output of the four groups, the crane must handle one skip every five minutes at the average rate of working. This allows ample margin for periods when the output exceeds the average and sufficient at other times for the crane to handle sheet piling, timber, and the hammer driving the sheeting.

Since each section of the work comprises 500 cu. yd. of material, the number of working hours of actual digging will be $\frac{500}{4 \times 2\frac{1}{4}} = 56$ hours, say, 1½ working weeks. The crane will therefore be occupied as follows:

One week handling and driving sheet piling for the first section of excavation.

One and a half weeks lifting and unloading skips for the first section and handling timbers for this section.

Three weeks in an intermediate period while concreting proceeds in the first section, upon completion of which timbering and piling can be withdrawn from the first section and piling for the second section can be driven.

One and a half weeks lifting and unloading skips for the second section and handling timbers for this section.

Three weeks while concreting proceeds in the second section, after which the piling and timbering can be withdrawn.

The total time is therefore ten weeks. Of this period of ten weeks, it can be assumed that the crane is occupied for seven complete weeks on excavation, sheet piling, and timbering, the remaining three weeks being allocated to placing the concrete in the foundation. The crane will no doubt be required on the work in the superstructure, and for the present case we will consider that, of the total charge of installing the crane, one-third is allocated to excavation and two-thirds to the superstructure. Thus, if the total charge against the contract for the crane, including five skips, depreciation and repairs, erection and dismantling, and carriage to and from site is £240, a sum of £80 should be charged against the excavation; to this must be added seven weeks' running expenses, including coal and other stores consumed and the wages of the driver and labourer attending the crane.

The cost of the sheet piling can be estimated by assuming that the piles will be 25 ft. long. The area to be provided in the first section will be $2(37\frac{1}{2} + 18) \times 25 = 2{,}775$ sq. ft. Since one 18-ft. length of piling will be common to both sections, the area to be driven in the second section is $[(2 \times 37\frac{1}{2}) + 18] \times 25 = 2{,}325$ sq. ft. Thus the total area to be driven and extracted is $2{,}775 + 2{,}325 = 5{,}100$ sq. ft.

The weight of piling required will be, say, 2,775 sq. ft. at 20 lb. per square foot = 25 tons, and the cost to be charged against the contract will consist of the carriage to and from the site, the cost of unloading and re-loading, together with either the cost of hiring the sheeting for say, three months, or a depreciation charge of, say, 10 per cent. of the purchase price. To this total a further charge should be added for the hire or depreciation of a double-acting pneumatic hammer and air-compressing plant. In the present case an inclusive sum of £130 will be allowed. This sum depends to a considerable extent on the sheeting available and the distance the material has to be transported. The method of arriving at the total of £130 is as follows:

			£	s.	
Sheet piling:	10 per cent. of purchase price	=	1	2	per ton.
	Two-way carriage	=	2	0	,,
	Total	=	3	2	,,

			£	s.	d.
	Total cost: 25 tons at £3 2s.	=	80	0	0
	(Loading and unloading at site included in crane charges.)				
Hammer and compressor:	Depreciation, etc.	=	25	0	0
	Carriage to and from site	=	15	0	0
	Erection and movement about the site	=	10	0	0
	Total	=	130	0	0

The plant and labour charges for handling the sheeting and handling the hammer when driving are included in the crane charges, and it is only necessary to add the cost of the additional labour during driving and extracting and the power consumed by the hammer. This additional labour may be assumed to consist of three men employed for three weeks during the driving and extracting periods.

The timbering for the first section may require 300 cu. ft. of timber, which is sufficient for the walings and transverse struts in three settings, and at 2*s.* per cubic foot costs £30. As most of the timber will be undamaged when extracted, only £20 need be written off against the contract, this sum representing part of the purchase price and carriage from the site. For five weeks three men will be employed setting or extracting the timber.

The last item to be considered among the primary charges is pumping, which will be practically continuous for a period of six weeks, but the pump may be on the site for eight weeks. Assuming that an electrically-operated pump is hired, the charge against the contract will be the hiring charges for eight weeks, the running expenses for six weeks (including one man in constant attendance), carriage to and from the site, and the general installation charges including the provision of discharge pipes and channels.

The foregoing costs for both labour and plant can now be brought together to arrive at a unit rate for the excavation.

	£	*s.*	*d.*
Labour—			
Excavation: 13 men for 3 weeks at £2 15*s.* per week	107	5	0
Crane: Driver—7 weeks at £4 8*s.*	30	16	0
Labourer—7 weeks at £2 15*s.*	19	5	0
Sheet Piling: 3 men for 3 weeks at £2 15*s.*	24	15	0
Timbering: 3 men for 5 weeks at £3 5*s.*	48	15	0
Pumping: 1 man, 3 shifts per day, at £2 15*s.* per week per shift for 6 weeks	49	10	0
Total labour	£280	6	0
Add 33⅓ per cent. for general on-costs, supervision, insurance, small tools, profit, etc.	93	9	0
Total labour charge	£373	15	0

The gross labour cost based on 1,000 cu. yd. is therefore 7*s.* 6*d.* per cubic yard.

	£	*s.*	*d.*
Plant and Materials.—			
Crane: Plant charge	80	0	0
Coal: 7 weeks at 3 tons per week at £2 per ton	42	0	0
Oil, stores, and water, 7 weeks at £2	14	0	0
Sheet Piling: Plant charge	130	0	0
Power for hammer: 3 weeks at £5	15	0	0
Timber: Material	20	0	0
Pumping: Hire of pump and hose, 8 weeks at £5	40	0	0
Power: 10 kw. per hour for 24 hours per day for 6 weeks at 1*d.* per unit	42	0	0
Carriage on pump	5	0	0
	388	0	0
Add 15 per cent. for general on-costs, profit, etc.	58	0	0
Total plant charge =	£446	0	0

Based on 1,000 cu. yd. this is equivalent to 8*s*. 11*d*. per cubic yard.

Thus the total estimated average cost of the excavation is 16*s*. 5*d*. per cubic yard, exclusive of the cost of carrying away and disposing of the surplus. It is interesting to note that the cost of digging and bringing the soil to the surface is less than 5*s*. per cubic yard, the remainder being due to the cost of providing timbering, sheeting, and pumping to deal with the water-logged condition of the ground.

EXAMPLE (iii).—An open excavation involving 30,000 cu. yd. measured in the solid; excavating by means of a ¾-cu.-yd. mechanical shovel digging into a gently-sloping hillside to level a site, material being deposited directly into jubilee wagons and transported and tipped on an adjacent part of the site.

The cost will depend principally on the output of the shovel and the charges against it. When working at an average speed the shovel digs about thirty times an hour, this rate allowing for delays, repairs, waiting for wagons, and movement about the site. In a 12-hour working day the output would be $30 \times 12 \times \frac{3}{4} = 270$ cu. yd. and the complete excavation would take $\frac{30,000}{270} = 111$ working days, that is twenty working weeks. The running charges of the machine will therefore be based on this period, while the plant charges will be based on a period of six months to allow for the time the machine is being transferred to and from the site and to cover possible delays.

The annual charge of the machine may be expressed as 10 per cent. for depreciation, 2½ per cent. for repairs, boiler inspection, etc., and 5 per cent. interest on capital outlay; that is, 17½ per cent. on a purchase price of £2,000, or £350 per annum. For a period of six months the charge would be £175. If this is increased to £200, allowance is made for carriage to and from the site and for storing the machine during idle periods between contracts.

The labour charges for a 12-hour day, allowing the driver and fireman one extra hour for getting-up steam, would be:

	£	s.	d.
Driver at 1*s*. 10*d*. per hour for 13 hours	1	3	10
Fireman at 1*s*. 4*d*. per hour for 13 hours		17	4
Three labourers attending machine, and clearing up at 1*s*. 1*d*. per hour for 12 hours	1	19	0
Net labour cost per day	4	0	2
Add 33⅓ per cent. for on-costs on net labour	1	6	9
Total labour cost per day	£5	6	11

The materials consumed per day by the machine would be:

	£	s.	d.
Coal: ¾ ton at 38*s*. per ton	1	8	6
Oil, stores, and sundries, say,		7	6
Water		3	0
	1	19	0
Add 15 per cent. on-cost		5	10
	£2	4	10

The total cost of excavation and loading into wagons will therefore be:

	£	s.	d.
Plant	200	0	0
Labour: 20 weeks at £5 6s. 11d. per day	593	0	10
Materials: 20 weeks at £2 4s. 10d. per day	202	11	8
	£995	12	6

The unit rate of this excavation will therefore be $\frac{£995}{30{,}000} = 8d.$ per cubic yard. It is clear that if the machine is used continuously, without delays on the site awaiting wagons, resulting in the machine working at almost twice the assumed average rate, the cost will be greatly reduced.

Bearing Capacity of Soils.

The design of a foundation to carry a given load is controlled primarily by the bearing resistance of the ground at the selected depth, this problem being more in the province of the designing engineer than those in control of site operations. In view of the fact that costly delays to contracts frequently occur due to preliminary troubles in the construction of foundations, the engineers on the site should understand the conditions assumed in the design. Most specifications forbid placing concrete in any foundation until the exposed ground has been inspected by the responsible engineer or his representative. It is good practice to acquaint engineers on the site with the assumptions made concerning the type of ground and safe bearing pressure under each section of the foundations. Any variation in the ground should be reported immediately to the designing engineer to allow alterations to be made at once and prevent delay or the risk of unsatisfactory foundations. It is essential for all parties concerned that responsibility for the assumed safe bearing pressure should be clearly defined in the contract documents.

When the depth of excavation required to expose or to penetrate a given distance into a specified stratum varies at different parts of the site, a record should be made of the depths to which the foundations are taken. This information, referred to an unmistakable datum level, should be marked on a plan of the foundations, which after approval by the designing engineer, should be sent to the clients. A record of this kind may prove of great value in subsequent extensions on the same or adjacent sites.

For the stability of raft foundations it is necessary that the ground resistance should be fairly uniform over the whole area of the foundation. Excavations may reveal exceptionally hard or soft patches. Unless allowed for in the design, hard patches such as occasional large boulders, existing buried walls, old foundations, or tree-stumps should be removed or cut away to a depth of at least 2 ft. below the proposed foundation level. The cavity so formed should be filled and rammed to simulate in compactness as closely as possible the surrounding ground. Existing pits, basements, and similar cavities, unless allowed for in the design, must be filled sufficiently firmly to ensure a bearing resistance neither much more nor less than that of the adjacent ground. In some cases where there is known to be a general change from hard to soft ground over the area of a foundation, the shape and design of the bases can be accommodated to give pressures that vary approximately with the probable safe ground resistance.

Commonly accepted values of the safe bearing pressures on different classes of ground are given in *Table* XX (page 262). In normal cases these pressures should be taken as the greatest permissible values and deductions should be made for ground to which any of the descriptions cannot be correctly applied. For deep foundations the values can be increased by about 100 lb. for each foot of depth of the foundation below the free surface. In basement or similar construction the free surface should be taken as the basement floor-level. Normally if a given soil is considered as having a safe bearing resistance of 1 ton per square foot and the foundation level on the same type of soil is 10 ft. below the lowest adjacent ground level, the working pressure may be taken as

$$2{,}240 + (10 \text{ ft.} \times 100 \text{ lb. per cubic foot}) = 3{,}240 \text{ lb. per square foot}$$
$$= 1{\cdot}45 \text{ tons per square foot.}$$

If at this level of 10 ft. below the surface there is encountered a different stratum having normally a safe bearing resistance of 2 tons per square foot, the safe pressure could still be increased by $10 \times 100 = 1{,}000$ lb. per square foot, giving a working pressure of 2·45 tons per square foot.

Some engineers allow the normal safe ground resistance to be increased by, say, 10 per cent. when the applied pressure from the foundation varies across the width or length of the base due to eccentric loading.

The least depth to which foundations should be taken depends on the nature of the ground. In the first place it is necessary to go to a certain minimum depth to prevent the ground spewing from under the base, and such minimum depths are given on *Table* XX (page 262) for those types of soil exhibiting this phenomenon where it may determine the foundation level, that is, in earths, sands, clays, and gravels. With these soils it is usually necessary to go at least 2 ft. below the normal surface before reliable virgin soil can be found, and unless the foundation happens to be on rock or on material of similar compactness 2 ft. should be considered the minimum. Greater depths should be adopted if a penetration of 6 in. to 12 in. into virgin soil is not obtained or if greater depth is required to prevent spewing. In clays minimum depths of 5 ft. and upwards are usually prescribed, since clay soils are more susceptible to drying and swelling due to weathering and frosts. It is desirable, therefore, to locate the loaded plane sufficiently deep to be well below the detrimental effects of such causes. When rafts over considerable areas are provided, it may not be essential to take the foundations to such a depth, as the effect of spewing and weathering may only be felt around the edges of the raft and allowed for accordingly.

Determination of the class of ground beneath the surface of a site is made by digging trial holes, sinking bores, or driving test piles. Trial holes can only be carried down to moderate depths and seldom exceed 20 ft. deep; generally they are much shallower. They afford the contractor an opportunity of studying the nature of the ground in its undisturbed form and of determining the difficulty of excavation and the need for timbering and pumping. An objection to trial holes is the localised nature of the inspection, which can only be overcome by digging a large number of holes. Usually the minimum number of holes should be one near each corner of the site and one or more holes near the centre. Trial holes left open over-night or for longer periods should be covered by boarding, or other available material, to prevent accidents to workmen or others frequenting the site.

A bore can be sunk to a much greater depth than a trial hole, but is still only a localised investigation. Owing to the ease and cheapness with which bore holes can be sunk once the apparatus is on the site, more bores can usually be sunk than the number of trial holes that could be conveniently dug.

Driving test piles is seldom warranted unless the foundation is to be piled. A test pile does not indicate the nature of the ground penetrated, but shows the relative hardness of successive strata or the depth at which rock is found below the surface. The results obtained from driving a test pile, combined with knowledge obtained from local information or from a single trial hole may, however, serve as a basis for deducing the geological formation below the surface. A test pile may be particularly useful in determining the thickness of top crusts of good bearing value when these overlie poorer strata.

A common method of ascertaining the reliability of strata exposed in a trial hole is to drive a $\frac{3}{4}$-in. or 1-in. diameter rod into the bottom of the trial hole, observing the variation of resistance at various depths. Generally, if the resistance increases with the depth driven the stratum exposed is uniform in character, but a marked increase in resistance at any depth indicates the presence of better ground below. Any falling off in resistance, especially a sudden decrease, indicates that the exposed stratum is of limited depth and may overlie a much weaker stratum. Such a test, owing to the ease with which it is made and its practically negligible cost, can be carried out not only at trial holes but at every excavation for a series of column bases if there is any doubt about the consistency of the ground.

The bearing values of certain soils can be increased to a limited extent by superficial ramming or considerably more by impregnation with cement grout. Consolidation by rollers (power or hand operated) or by hand, percussion, or "frog" type rammers is often applied to artificial filling. Consolidation of this kind is most essential to the stability of structures that are erected on recently-filled ground.

Grouting with Cement.—When poor loose ground, such as tipped stone or other fairly hard material, is encountered, and limitations exist that necessitate loading this ground beyond the normal safe pressure, its bearing capacity can be improved by injecting cement grout under pressure. The procedure is to drive a series of open-ended iron pipes 4 ft. or 5 ft. into the ground below foundation level on and around the site of the proposed foundation. The number of pipes would depend on the extent of the foundation and on the spacing, fixed by the results obtained, required to give the desired degree of consolidation. As a preliminary trial, pipes 2 in. or 3 in. in diameter may be inserted at 5-ft. to 10-ft. centres. The ends of each length of pipe should be threaded for the subsequent attachment of the grout feed-pipe from the pump, or to enable additional lengths to be fixed when driven to depths in excess of a single length. The pipes are driven by a sledge-hammer, the blow being taken on a cast-steel dolly fitted into the end of the pipe and protecting the threads.

When each pipe has been driven, the soil in the pipe is removed with an auger and water is poured into the pipe to find whether there is a free exit at the lower end into the stratum to be consolidated. It is sometimes possible to break any local obstruction encountered during driving by thrusting a heavy pointed bar into the pipe after the earth has been removed with the auger.

The hose or pipe to the cement pump is now connected up and cement slurry is pumped through the pipe into the ground. A suitable consistency for the slurry is about 150 gallons of water to one ton of cement. The use of rapid-hardening Portland cement results in earlier consolidation of the stratum. The grout should be pumped into each pipe in turn until refusal. If any bore-hole shows no signs of sealing, a limited amount of grout, say, the equivalent of 1 ton of cement, should be inserted. After an interval, injection should proceed on another hole, later returning to the unsealed hole. In this way channels facilitating the escape of the grout may be sealed more expeditiously than by concentrating on a single hole. An interval of, say, three hours should elapse between completing injection in one hole and commencing another within a few feet of the previous hole.

During the injecting process it is advisable to keep a watch on land drains, cellars, streams, or other low-level works or watercourses near the operations to detect any continuous escape of grout. Pipes similar to those used for injection, driven into the ground around and within the area being treated, are useful in detecting the flow of the grout and estimating the extent of the consolidation.

A development of the principle of pressure grouting is the chemical consolidation of ground now practised under patented processes. A number of pipes is sunk into the ground, and while the pipes are being driven a sodium silicate solution is pumped into the ground. While the tubes are being withdrawn calcium chloride is pumped through the pipes. The mixed solutions set hard, and consolidate the ground. By this treatment water-bearing sandy ballasts are rendered as solid as sandstone and can be excavated with a vertical unsupported face.

Bearing Tests.

When there is any doubt about the bearing capacity loading tests may be prescribed. The type of soil and available loading material control the actual details of the test but the underlying principles are :

(1) The ground tested should be representative of the normal ground under the foundation ;

(2) The loaded area should be as large as practicable, especially in clay soils, although it is usually limited by the amount of the available weights and the methods of handling them ;

(3) The load should be gently and uniformly applied in equal increments at intervals of two to three hours ;

(4) The maximum load applied should be at least 50 per cent. greater than the anticipated safe loading. On grounds of low bearing resistance the loading should proceed until definite breaking down is evident ;

(5) The duration of the test should be for as many days as practicable ;

(6) Readings of the settlement should be taken immediately before and after each increment of loading ;

(7) The readings should be referred to a datum that will be unaffected by the

test, and should therefore be several feet away from the tested area, as the effect of a high test load may be to force up the level of the surrounding ground.

(8) Readings should be taken at a sufficient number of points to detect any tilting.

The following descriptions of methods of testing are taken from practice.

The simplest form of test is made with a timber sole-plate 1 ft. or 2 ft. square placed at the bottom of the trial hole. A timber post bearing on the centre of the plate carries a loading table and is guyed to prevent tilting. The weights of the post, plate, and table, and of each increment of loading, should be accurately known.

A similar, but improved, apparatus is shown in *Fig.* 206. A pit 10 ft. to 15 ft. square is excavated to the proposed foundation strata, and a smaller pit, say

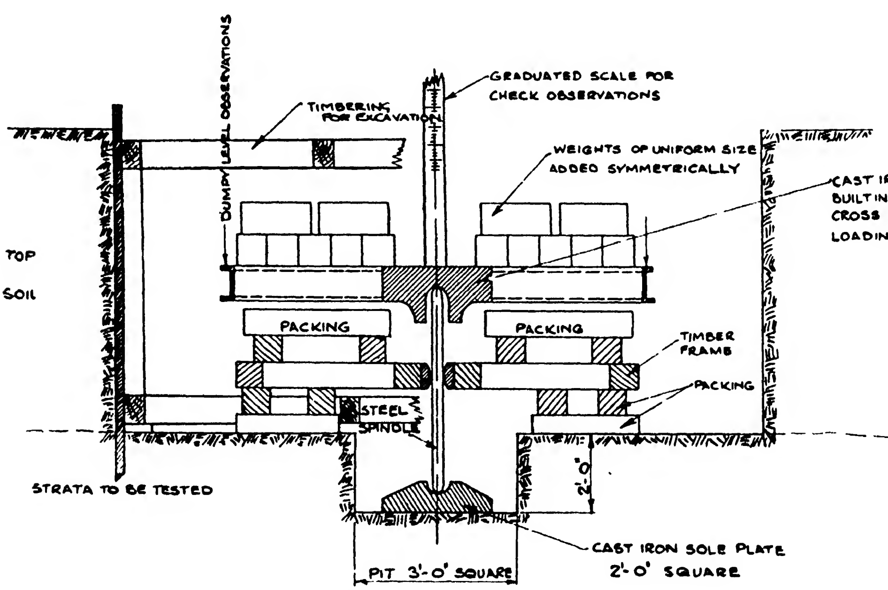


FIG. 206.—GROUND TESTING MACHINE.

3 ft. square, is sunk about 2 ft. farther. The sides of the main excavation may need to be timbered, but in moderately firm ground it may not be necessary to sheet the smaller pit. If water is found arrangements should be made to drain it into a sump in one corner of the main pit from which it can be pumped. The bottom of the smaller pit should be kept dry, but in so doing care must be taken not to disturb the surface so as to turn it into mud. The testing apparatus consists of a metal sole-plate, say, 2 ft. square, on which rests a metal spindle with a cast-iron cap supporting a steel or timber cross-head carrying the load. The ends of the vertical spindle are hemispherical and bear in cups on the sole-plate and cap, thus ensuring vertical reactions. The spindle passes through a guide in the frame, on which packing can be placed to support the cross-head should the latter become unbalanced. Otherwise throughout the whole of the test the cross-head is carried entirely by the spindle. Settlements are read with

a dumpy level, a check reading being taken on a graduated scale attached to the machine. The most convenient loading is a number of equal iron weights.

In *Fig.* 207, the loaded area is a platform of concrete weighted with pig

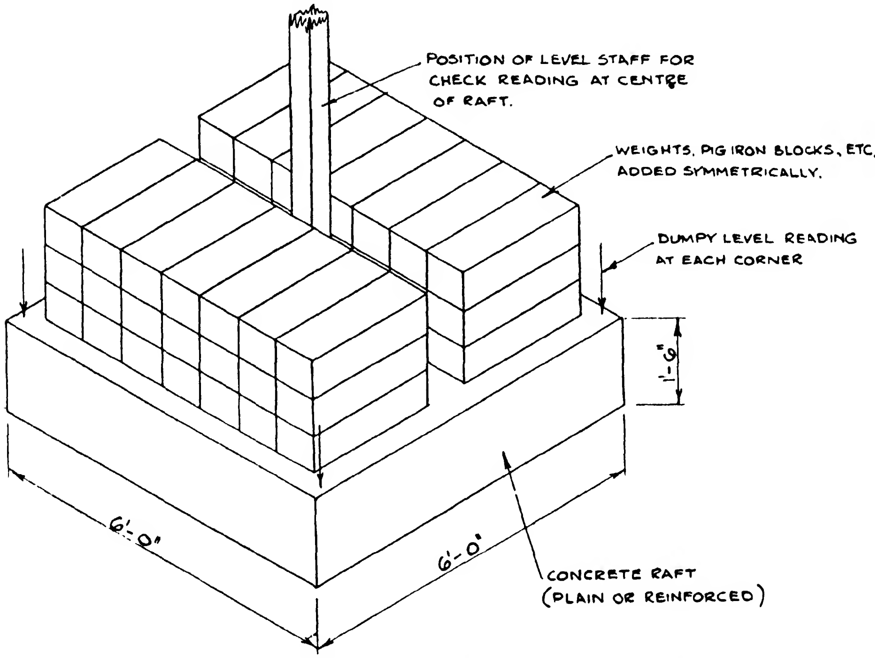


Fig. 207.—Testing Large Area of Ground.

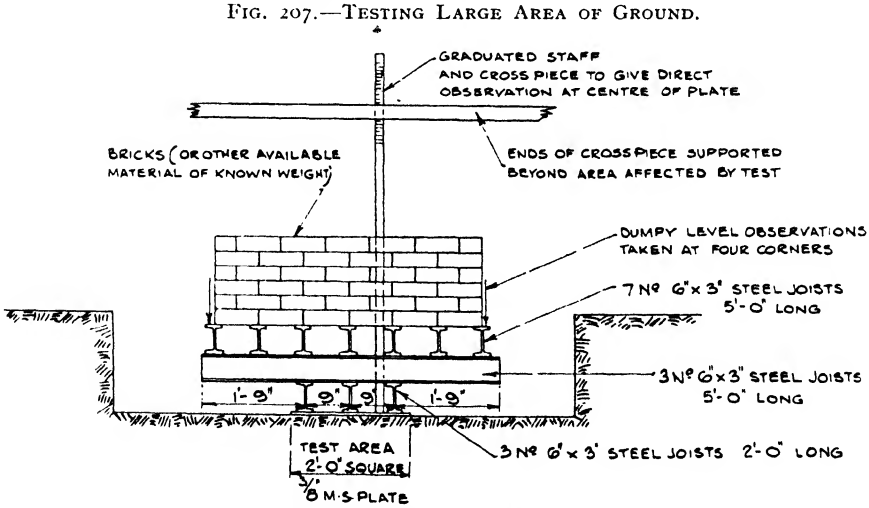


Fig. 208.—Testing Small Area of Ground.

iron, the pigs being added in a symmetrical manner to preserve uniformity of loading. Measurements are taken at each corner and in the centre. The weight of the base gives an initial load of 2 cwt. per square foot, and the pigs are added

RECORD OF GROUND TEST — CONTRACT No W 536. — TEST I :– NORTH-EAST CORNER OF SITE.

Date and Time.			Loading. Load Applied.	Loading. Total Load.	Pressure on Ground.	Observation by Dumpy Level. (Feet) Corner A	Corner B	Corner C	Corner D	Centre.	Mean Settlement	Remarks.
WED.	OCT. 2ND	2.0 P.M.	PLATFORM ONLY	1350 LB.	0·151 TONS/FT.²	6·846	6·812	6·823	6·831	6·828	-	
		3.0 "	1750 LB.	3,100 "	0·346 "	6·853	6·820	6·832	6·839	6·836	3/32"	
THURS.	" 3RD	10.0 A.M.	-	3,100 "		6·854	6·820	6·831	6·839	6·836	3/32"	
		11.0 "	1750 "	4,850 "	0·542 "	6·856	6·822	6·833	6·841	6·838	1/8"	
		3.0 P.M.	-	4,850 "		6·858	6·825	6·837	6·844	6·841	5/32"	
		4.0 "	1750 "	6,600 "	0·736 "	6·864	6·830	6·841	6·849	6·846	7/32"	
FRI.	" 4TH	9.30 A.M.	-	6,600 "		6·863	6·830	6·842	6·849	6·846	7/32"	
		10.30 "	1750 "	8,350 "	0·932 "	6·869	6·836	6·848	6·855	6·852	9/32"	
SAT.	" 5TH	3.0 P.M.	-	8,350 "		6·870	6·838	6·851	6·857	6·854	5/16"	
		4.0 "	875 "	9,225 "	1·030 "	6·877	6·844	6·856	6·863	6·860	3/8"	
SUN.	" 6TH	12.0 NOON	-	9,225 "		6·877	6·844	6·856	6·863	6·860	3/8"	NO ADDITIONAL LOAD.
MON.	" 7TH	10.0 A.M.	-	9,225 "		6·879	6·846	6·858	6·865	6·862	13/32"	APPROXIMATELY ONE TON/FT² THROUGHOUT WEEKEND.
		11.0 "	1750 "	10,975 "	1·222 "	6·880	6·848	6·861	6·867	6·864	7/16"	
TUES.	" 8TH	9.30 "	-	10,975 "		6·882	6·849	6·861	6·868	6·865	7/16"	
		10.30 "	1750 "	12,725 "	1·425 "	6·887	6·854	6·866	6·873	6·870	1/2"	
WED.	" 9TH	10.0 "	-	12,725 "		6·890	6·858	6·870	6·877	6·874	9/16"	
		11.0 "	1750 "	13,475 "	1·505 "	6·896	6·864	6·876	6·883	6·880	5/8"	
THURS.	" 10TH	10.30 "	-	13,475 "		6·905	6·874	6·887	6·893	6·890	3/4"	
		11.30 "	1750 "	15,225 "	1·703 "	6·915	6·885	6·899	6·904	6·901	7/8"	
FRI.	" 11TH	9.0 "	-	15,225 "		6·924	6·895	6·910	6·914	6·911	1"	UNLOADING PROCEEDING.
		9.15 "	UNLOADING COMMENCED	-								
SAT.	" 12TH	10.0 "	PLATFORM ONLY	1350 "	0·151 "	6·908	6·880	6·895	6·900	6·896	13/16"	TEST COMPLETED AND PLATFORM DISMANTLED.

WEIGHT OF UNLOADED PLATFORM COMPLETE : 1350 LB.
AREA OF BASE PLATE : 2 FT. x 2 FT = 4 SQUARE FEET.
PLATFORM LOADED BY BRICKS : AVERAGE WEIGHT OF EACH BRICK = 8¾ LB.
EACH INCREMENT OF LOAD = 200 BRICKS = 1750 LB.
(EXCEPT ON 5TH INST. = 100 BRICKS = 875 LB.)

WEATHER THROUGHOUT TEST – FINE
NO FROST OR RAIN.

LEVEL OF TEST PLATE : + 2·45 FT. ABOVE DATUM.

FIG. 209.—RECORD OF BEARING PRESSURE TEST.

so as to give 5-cwt. increments twice during each day. Thus at the end of the third day a ground pressure in excess of 1½ tons per square foot would be obtained. This test is suitable for an extensive raft foundation with a safe pressure not exceeding 1 ton per square foot. Even at a maximum test pressure of 1½ tons per square foot the application of the load would require handling 50 tons of pig iron.

When a large quantity of loading material is not available a smaller area can be tested as illustrated in *Fig.* 208. In this case a 2-ft. square ⅜-in. mild steel plate is laid on a carefully levelled area at the proposed foundation level. Tiers of joists are added to form a platform 5 ft. square upon which the load, in the form of bricks, pigs, or weights, is placed. Measurements are taken at the four corners of the platform and in the centre of the plate. The overhang of the joists is not sufficient to produce measurable deflections.

The record of a loading test would be made in the form illustrated in *Fig.* 209. The amount of settlement permissible before considering the ground to be overloaded depends on the type of ground, foundation, and structure. Generally the safe bearing pressure would be some value much less than that at which progressive settlement occurs.

SECTION II.—FOUNDATION CONSTRUCTION.

Setting-out Foundations.

When a site is handed over to the contractor it is necessary to give the positions of

(*a*) the "building line" along one frontage at least;

(*b*) a point on the latter marking the position of a particular feature, say, one corner; and

(*c*) a datum in respect to which all foundation, floor, and other levels are referred.

In *Fig.* 210, which indicates the preliminary stage in setting out the column foundations of a rectangular building, the given point A on the building line represents one corner of the structure. The first step is to establish the centre point of each column foundation and to place marks from which the column centre-lines can at any time be picked up. The plans of the building will show the distances *a, b, c, d*, etc., from point A to column B and between the centre lines of each two columns. These distances are set out with a steel tape along the building line to establish the points B, C, D, etc., at each of which a wooden peg is driven into the ground. Points B and E represents the points where the centre line through the end rows of columns intersects the building line, and through these points it is necessary to set out lines normal to the building line. If a theodolite is not available this can be done with a tape as follows. Set out from B towards C a distance of 12 ft. and drive in a peg at F. Now locate the point G which is 20 ft. from F and 16 ft. from B. This can be done by attaching a piece of string 20 ft. long to a nail in the peg at F, and another piece, 16 ft. long, to a nail in the peg at B; where the free ends of the strings meet when stretched taut will be point G, and the angle at B will be a right angle. Another method is to hold the end of a tape at B and the 36 ft. mark at F. Holding this length of the tape at the 16-ft. mark, pull the tape taut ; the position of the 16 ft. mark will be the point G. A linen tape is necessary.

Having established the point G, stretch a string from the nail in the peg at B over the top of the peg at G to some extreme point H.

In a similar manner establish a point J, corresponding to H, on a line through E perpendicular to the building-line. As a check on the positions of J and H, the lengths EH and JB should be measured and compared with their calculated values. Along each of the lines BH and EJ (*Fig.* 211) drive in pegs at the distances *e*, *f*, *g* and *h* as read from the drawings, thus establishing the points K, L, M, N and P, Q, R, and S which are the positions of the columns. Stretch

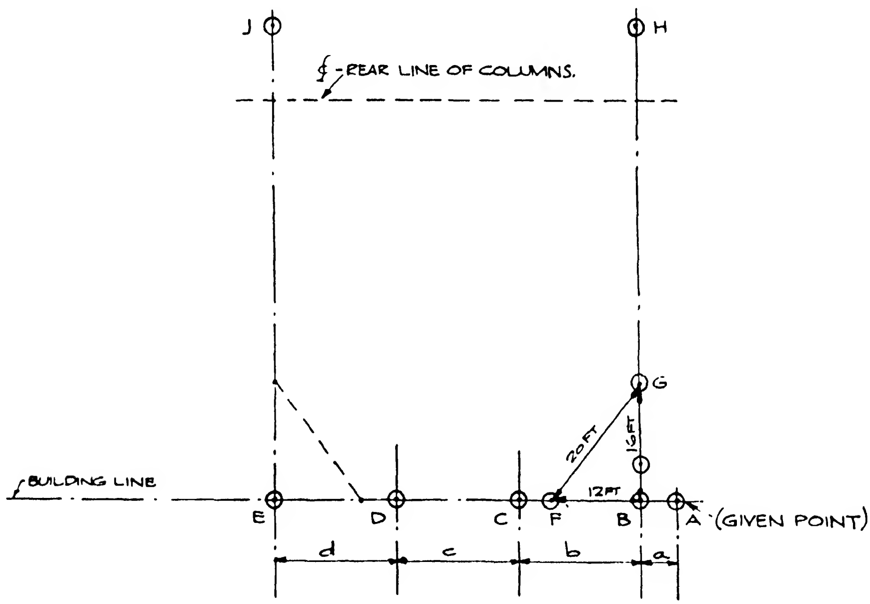


FIG. 210.—PRELIMINARY STAGE IN SETTING-OUT BUILDING.

lines from K to P, L to Q, M to R, and N to S, these lines representing the centre-lines of each transverse row of columns. With the distances *b*, *c*, and *d*, the points T and U on line NS can be established and strings stretched from T to C and from U to D will represent the centre-lines of intermediate rows of columns. The intersections at V, W, X and Y will establish the centres of the remaining columns and pegs should be driven at these points.

At this stage it is convenient to check the foregoing setting-out. This is best done by first comparing the measured distances KP, PS, SN and NK with the corresponding overall dimensions given on the drawings. If these agree, the accuracy of the angles can be checked by comparing the distance PN with KS ; these two dimensions should be identical ; if not, the whole of the setting-out should be adjusted.

The next stage is to set up marks (*Fig.* 212) outside the area to be excavated so that when the pegs representing the column centres are removed during digging, the column centre lines can be easily picked up again. The usual method is to drive a line of stakes parallel to each side of the building and attach horizontal

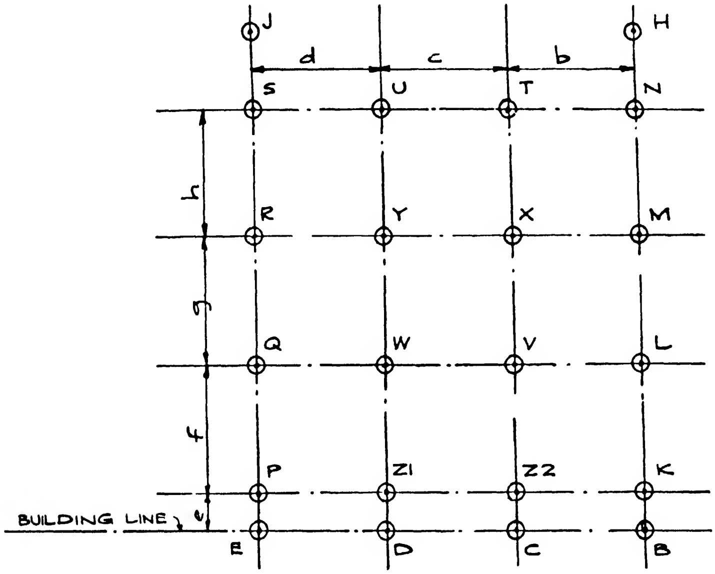


FIG. 211.—SECOND STAGE IN SETTING-OUT BUILDING.

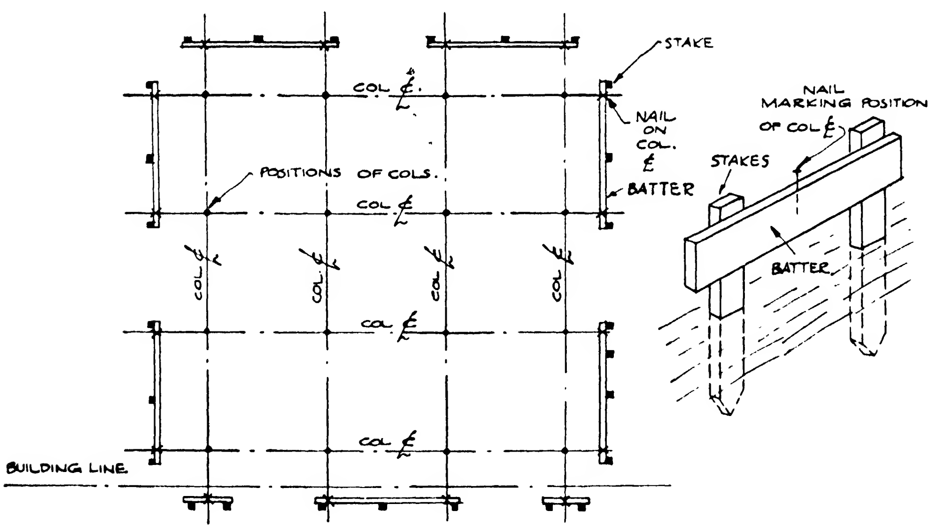


FIG. 212.—FINAL STAGE IN SETTING-OUT BUILDING.

boards called batters, to the stakes, a foot or so above the ground. If these boards are not continuous no obstacle to free access to the site is presented. Each of the column centre-lines can now be extended to the corresponding batters, a nail being driven into the latter at the exact point of intersection. The lines can now be removed.

If it is necessary to locate any column centre it can now be simply done by stretching transverse and longitudinal lines between the appropriate pairs of nails on the batters. The lines may be of string or cord, but for the main setting-out lines of major structures (for example, the centre line of a bridge) piano wire stretched taut is more durable and accurate.

Pegs are usually cut from 2-in. by 2-in. timber with one end sharpened and well driven into firm soil. For more durable marks the peg should be embedded in a small concrete block. Marks outside the building area for the purpose of picking up column centre lines can often be conveniently established by scribing a line on adjacent brick walls, stone kerbs, steel rails, or elsewhere as convenient.

The principles of setting-out described in the foregoing comparatively simple example are generally applicable to most structures, although complications arise when buildings are not square on plan or on sites that are not level. Most of the difficulties encountered in setting-out are solved by methods evolved by careful consideration of the problem on the site, and, since responsibility for the accuracy of the setting-out is usually laid upon the contractor, the services of an agent experienced in site surveying are necessary in all cases. It is advantageous if the setting-out of the principal lines can be checked by an independent engineer, for example by the resident engineer or clerk of works, before actual construction begins.

Preparation of Ground Surface.

When an excavation has been taken out and pumped dry (if necessary), it is preferable to " blind " the exposed surface of the ground before laying the foundation reinforcement so as to provide a moderately smooth and clean working space. This blinding layer can be a lean concrete, say, 1 part of cement to 10 parts of aggregate, and its thickness should vary with the nature of the surface. A nominal thickness of 2 in. may be sufficient for a fairly firm soil such as compact gravel, firm alluvial soil, or firm clay, but loose gravel may require a thickness of 3 in. On poor wet clay a layer of ashes may be spread before laying a 1-in. blinding layer of concrete, otherwise several inches of concrete are necessary to provide a suitable surface. In some cases as much as 12 in. must be used on clay. Firm damp sand does not usually require a blinding layer, but in other cases, whether a layer is specified or not, its provision by the contractor saves its cost in increased efficiency due to better working conditions and clean reinforcement. The aggregate used for the blinding need not be of such high quality as the material specified for the structural parts of the building. Broken bricks can be used, and for this purpose it is not necessary for them to be free from lime mortar as specified for structural concrete. If the excavation is in gravel or sandy gravel, this material will form a readily available aggregate for the blinding concrete. Around pile-heads the chippings resulting from cutting down the piles can be spread over the ground and grouted in, thus forming a conveniently prepared surface upon which to build the pile caps and tie beams between the caps.

Blinding layers not only prevent the concrete from becoming contaminated with loose ground, but prevent the water in the concrete being absorbed by dry subsoils. The same results have been obtained by impregnating the formation under concrete road slabs with oil and by spreading cement bags or layers of waterproof paper on loose porous filling.

To prevent concrete coming into contact with the earth, the vertical faces of beams constructed in the ground should be timbered. The timber necessary

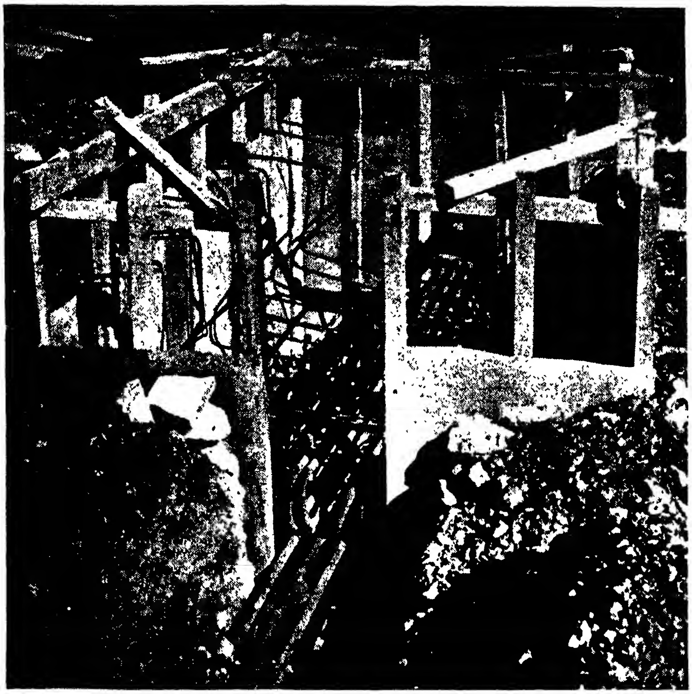


FIG. 213.—PLYWOOD SHUTTERS FOR STANCHION BASE.

for forming the beam moulds need only be rough, used planking requiring little or no cutting and therefore having a high salvage value. Plywood is sometimes used as shuttering for ground beams and stanchion bases (*Fig.* 213). The timber can sometimes be removed after the concrete has matured, but this may lead to delay in proceeding with subsequent operations. To avoid delay, or alternatively the loss of the timber, the sides of the excavations for beam trenches can be lined with lean concrete of 10 : 1 or 12 : 1 mix made with any available cheap aggregate (for example, breeze in the case of gas works or colliery contracts).

Concreting Foundations.

Shuttering may not always be necessary for foundations if the excavations are shallow and in firm ground and are taken out to the exact size of the concrete bases. The vertical, or practically vertical, sides of excavations for shallow foundation beams in soft soils can be stabilised by being roughly rendered with a lean cement mortar mixed as dry as possible. The bottom of the beam trench may be covered with lean concrete. With these precautions timber shuttering is unnecessary.

In retaining walls and similar structures, where the reinforcement is adjacent to the earth face of the concrete work, shuttering is necessary to ensure that loose earth does not mix with the concrete; exceptions can be made in the case of rock, sandstone, or similar compact ground, or when building mass concrete walls. It is often convenient in sheeted excavations to use the timbering for shuttering, and concrete can be successfully placed against temporary steel sheet piling. In the latter case, however, it is necessary to protect the joints between the piles by thin metal sheets or similarly, to prevent cement grout running into the clutches and hindering the extraction of the piles.

When the sides of an excavation are untimbered or unprotected in any other way, it may be necessary to protect from falling earth the concrete freshly placed

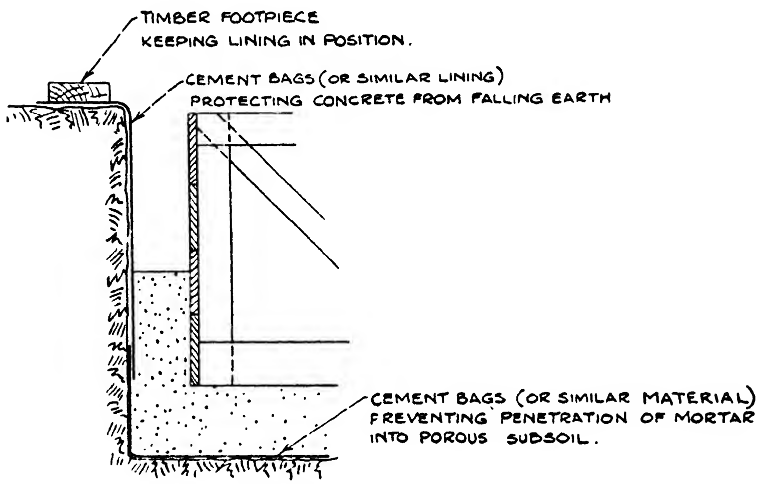


FIG. 214.—PROTECTION OF FOUNDATION CONCRETE.

in the bottom of the excavation. This can be prevented in small excavations by laying sacks or old cement bags along the top edge of the excavation as illustrated in *Fig.* 214.

Foundation construction plays an important part in planning the work, and ill-thought-out schemes may often cause serious delays in the erection of the superstructure. If reinforced concrete foundations are provided for steel buildings, the programme should be so arranged that the foundations have matured and are ready to take the steel stanchions as soon as the latter are delivered. A method used in America to prevent delay when extensive foundations are to be constructed is to provide a temporary foundation sufficient to support the dead weight of the steelwork for the lower floors of the building, the erection of which can proceed while the larger foundations for the complete structure are being constructed. Although this method has been developed for skyscraper buildings, the principle is worthy of consideration for some large buildings erected in this country.

Foundation construction may involve underpinning existing structures in plain or reinforced concrete. As the construction can generally be approached from one working face only, there are practical difficulties in bringing the concrete underpinning into close contact with the underside of the footing or base unless resort is made to pressure grouting. It is, therefore. better to build up the concrete to within 9 in. to 18 in. (depending on the horizontal depth of the work) of the underside of the existing work, and when the concrete has hardened to finish off with brickwork set in cement mortar with a slate-in-cement topping. If the load to be carried is heavy, wire-cut Staffordshire blue bricks bedded and jointed in Portland cement mortar should be used. Alternatively, grouting in a 1-in. steel packing plate between the concrete and the underside of the existing work to be supported will produce the same result when the bearing area is limited to a few inches in each direction.

When underpinning columns or wall piers it is essential to provide temporary supports to transfer the load from the column, etc., to the ground while excavation is proceeding under the column.

In underpinning walls work should be carried out in short lengths, say 3 ft. to 5 ft. at a time, thus avoiding the need for temporary supports during construction. If the work being underpinned is in a bad state of repair, support in the form of vertical posts should be provided to guard against possible damage. It is advisable, before commencing to underpin existing buildings, to examine thoroughly both faces of the wall to detect fractures or incipient cracks, and to report the presence of such to the owners before beginning excavation. This precludes the possibility of the underpinning contractor being blamed for the production of these defects. A dated and witnessed photograph of the wall before commencing operations may often prove of value in settling subsequent disputes.

The cost of concrete in underpinning work in short lengths may be 20 per cent. more than the cost of normal work, since the labour costs may be increased by 60 per cent. or more.

Ground Water.

Concrete work buried in saturated soils may be adversely affected if the ground waters contain mineral sulphates. The commonest deleterious salts are sodium sulphate and magnesium sulphate (Epsom salts), while calcium sulphate (gypsum) and potassium sulphate may also be present. Sulphates are frequently found in clays, gypsum for example being met with in the London clay in the form of nodules. Both magnesium and sodium sulphate are very soluble, and waters in contact with, or draining from, clays containing these salts may contain appreciable concentrations of sulphates. The chlorides and carbonates of sodium and magnesium have little or no effect on concrete. Carbonic acid in solution attacks concrete and may be present in moorland waters that contain few dissolved salts. Water that is slightly acid due to a humic acid content derived from decaying peat may not harm concrete. Very pure waters may attack the surface of Portland cement concretes by dissolving the free lime. Hard waters, on the other hand, have no deleterious effect on dense concrete. Sulphuric acid and sulphates may occur in ground waters in the vicinity of gas works and deposits of trade wastes. In the soil under or near coal storage yards, iron sulphate derived from pyrites in bituminous coals may be present, while sites of old brickyards, and dumps of clinker, ashes, or old bricks may lead to contamination of the soil by sulphates.

The effect of sulphates on concrete is due primarily to the combination of sulphates with the hydrated calcium aluminates in the concrete, causing expansion and disintegration. Calcium and sodium sulphates soften the concrete, while attack by magnesium sulphate is recognised by the hardness of the particles. Attack may be more virulent in the case of retaining walls in clay soils and where one face of the concrete is in contact with wet sulphate-impregnated clay and the opposite face is exposed to the air. The passage of water through the concrete and evaporation at the free surface concentrate salts in the mass of the concrete. To resist attack by sulphates the concrete must be impermeable. A strong concrete with a high cement content (not less than 650 lb. per cubic yard of set concrete is recommended) is required, the higher the compressive strength the greater the resistance to sulphate attack. Durable aggregates should be used. Crushed stone and gravel offer the greatest resistance to attack ; flint and sandstone have proved deficient. The use of crushed limestone to neutralize the acid water in the vicinity of coal storage yards has been found to be unsuccessful, due probably to an insoluble film of iron and calcium salts being formed on the limestone particles.

A slump of 3 in. to 4 in. is recommended to give the greatest resistance ; there should also be ample cover on the reinforcement. The usual precautions should be taken to ensure proper curing, and the concrete should not be exposed to contact with deleterious ground water until it has matured. This can be conveniently done with pre-cast members, but with in-situ construction more difficulty may be experienced, pumping or other means to keep the ground water in check being necessary. Protection has been successfully afforded in the case of in-situ work by lining the excavation with canvas treated with bitumen.

Portland cement concrete is more liable to attack than high-alumina cement concrete, owing to the greater quantity of free lime in the former. High-alumina cements appear to be practically immune from attack by sulphates of any kind and in any concentration normally found in ground water, or by very pure or acid waters, this immunity lasting over long periods of years. The resistances of pozzolanic cements, which are used more on the Continent than in this country, or Portland cements with an addition of trass, are intermediate between those of normal Portland cement and high-alumina cement.

In a suspected case of contamination it is advisable to analyse the ground water to detect the nature and quantity of any deleterious substances. If the ground water contains less than 0·01 per cent. of sulphur trioxide, present usually as gypsum, there appears to be no fear of deterioration, and with dense concretes up to 0·02 per cent. may be harmless. Quantities of 0·10 per cent. and upwards have been found to be definitely harmful to Portland cement concrete. Where possible a cement known to have a resistance to the anticipated concentration of sulphates should be employed.

Where the concrete is exposed to attack, surface protection can be obtained by coating with asphalt, tar, pitch or special paints. Protection has been provided by applying hot three successive coats of boiled linseed oil. A successful coating has been provided by applying two coats of a mixture of 3·4 parts by weight of tar to one part of pitch, the proportion being varied slightly with the temperature. One gallon of the mixture covers nearly 2½ sq. yd. (two coats).

Protective paints are usually applied cold with a soft brush, three coats being required. The concrete surface to receive these paints should be moistened before application, and each coat should be allowed to dry before applying the next.

It is necessary to ensure that surface protection applied to the set concrete will adhere to it.

SECTION III.—PRE-CAST PILES.

Design of Piles.

Reinforced concrete piles are of two principal types, pre-cast and cast-in-situ, the latter being usually formed by patented processes, descriptions of which are not given in this volume.

Pre-cast piles are commonly square with chamfered edges, or octagonal, in cross-section. One end is tapered to facilitate ground penetration and may be protected with a metal shoe. The reinforcement consists of four, six, eight, or more bars, symmetrically arranged and bound together with separate links or helicals, forks or special hoops being provided to retain the longitudinal bars in position. Holes are usually formed at convenient points through the pile to attach the lifting and guiding gear.

The size, reinforcement, and length of pile are usually specified by the engineer, but there is generally a fairly constant relationship between these three factors. Up to 30 ft. in length piles may be 12 in. square; up to 45 ft., 14 in. square; and up to 65 ft., 16 in. square. Piles over 70 ft. long are exceptional but are sometimes used.

The required lengths of piles are usually ascertained by driving test piles. An assessment of the length based on adjacent contracts or other suppositions may be good enough for preliminary estimates, but may lead to delay due to lengthening piles or cutting away excess lengths when driving commences. In some instances it has been necessary to abandon piling schemes based on assumed lengths because at a later date the depth at which a satisfactory set was obtained was excessive. It generally pays to drive one or more test piles, the number depending on the uniformity of the ground and the extent of the site, before commencing to make the bulk of the piles. The contract conditions should be followed as closely as possible if the results given by the test pile are to be of any value. The size of the pile, the weight and drop of the hammer, and the ultimate set should correspond exactly. Misleading results have been obtained by driving timber test piles instead of concrete piles and by using a hammer of different weight from that to be used in the contract. The depth at which a given set will be obtained with a heavy hammer and a small drop cannot always be assessed by trial with a light hammer and bigger drop. For example, a set of ten blows per inch may be obtained at a certain depth with a ¾-ton hammer dropping 4 ft., but a much greater depth will be required under contract conditions with a 1½-ton hammer dropping 27 in. In another case the length of the concrete piles was estimated by driving timber piles with a ⅝-ton drop hammer. The least length required for the final piles, driven by a 1½-ton single-acting steam hammer, was twice the anticipated length.

In certain soils, for example where piles have to be driven to rock or another well-defined firm stratum, test piles can be dispensed with, the depth at which the bearing stratum lies being obtained more cheaply by bores. Where the level of the stratum is irregular a large number of bores should be taken to obtain its contour. A few test piles, of equal or greater cost than the bores, would probably give misleading results in this case. A further value of bores is that the probable thickness of the bearing strata may also be obtained.

The lower end of a pre-cast pile may be either tapered or blunt and may be shod or not. Pile shoes are made of cast iron, cast steel, or steel plate; the most usual type consists of a cast-iron point attached to the concrete by wrought-iron straps. Various types of pile ends, with and without shoes, are shown in

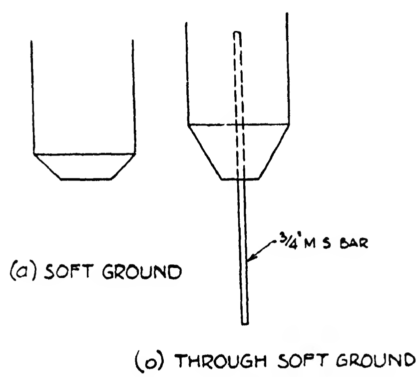


FIG. 215.—PILE TOES WITHOUT SHOES.

Figs. 215 and 216. When driving for example, into soft soil, loose filling, vegetable earth, river silt or soft clay down to a firm stratum, shoes may be dispensed with, and to obtain the required bearing resistance it may be convenient to form a blunt end to the pile as in *Fig.* 215(*a*). If considerable penetration of the bearing stratum is not anticipated, an unshod pile with a projecting bar as in *Fig.* 215(*b*) is used. The more common type of shoe illustrated in *Fig.* 216(*a*) may weigh between ½ and 1 cwt., depending on the size of the pile and the hardness of the ground. Standard designs are given in makers' catalogues. The wrought-iron straps may be 1¼-in. by ¼-in. to 2-in. by ½-in. flats and are sometimes provided with bolts that pass right through the pile, but, although such bolts are essential for timber piles, they are not necessary for concrete piles as sufficient connection can be made by means of the tongues embedded in the concrete. To prevent planes of weakness in the pile the straps are sometimes staggered, but this refinement is not common to standard shoes except for narrow sheet piles. When ordering shoes from standard designs, care should be taken to ensure that the tongues on the straps do not foul the proposed position of the longitudinal reinforcing bars. The point of the shoe may be provided with four holes or with a large recess to accommodate the ends of the bars. A shoe of this type, together with the assembled reinforcement, is illustrated in *Fig.* 217.

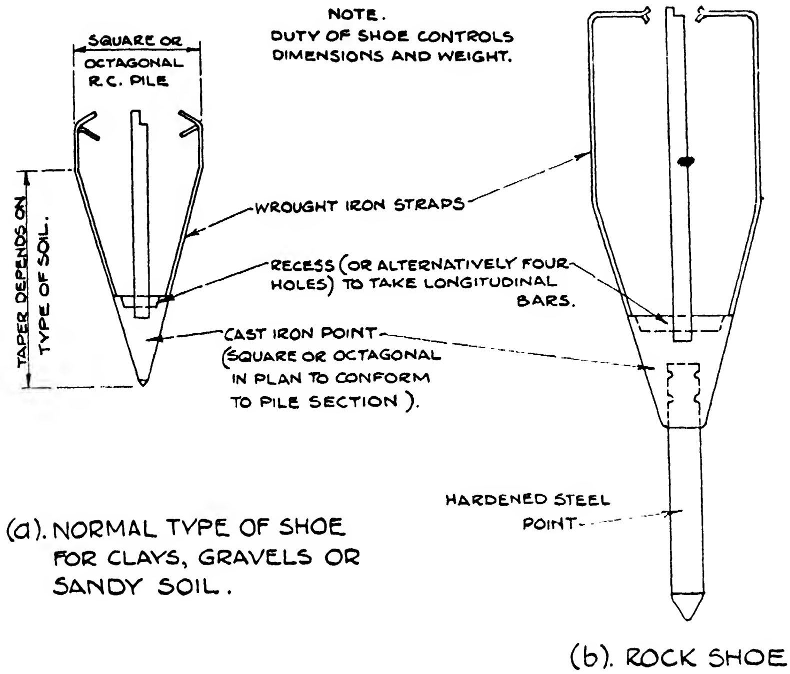


FIG. 216.—PILE SHOES.

FIG. 217.—DETAIL OF PILE SHOE AND REINFORCEMENT.

The "sharpness" of the point depends on the material into which the pile is to be driven, and the taper on each side may vary from 1 : 2 to 1 : 4, the blunter piles being suitable for clays, the medium for gravels, and the sharper for sands.

For piles driven to rock a heavier type of shoe of a different pattern is used as shown in *Fig.* 216(*b*). A hardened steel point 3 in. or 4 in. in diameter is inserted in the cast-iron body of the shoe; these shoes weigh from $1\frac{1}{2}$ cwt. to 3 cwt. complete.

In fitting shoes it is essential that the point should be in line with the axis of the pile to ensure straight driving.

Manufacture of Piles.

The process of making pre-cast reinforced concrete piles involves assembling the reinforcement and shuttering, fitting the shoes, concreting, removing the piles from the shuttering, and stacking for curing.

The shuttering for piles can be made with timber, or when large numbers are involved, steel moulds, or alternatively moulds formed with steel sides and timber bottoms, are economical and can be quickly assembled, struck, and cleaned. *Fig.* 218 gives details of alternative constructions for square piles, the underlying principle being that the sides can be stripped first. Generally it is best to attach the fillet pieces forming the bottom chamfers permanently

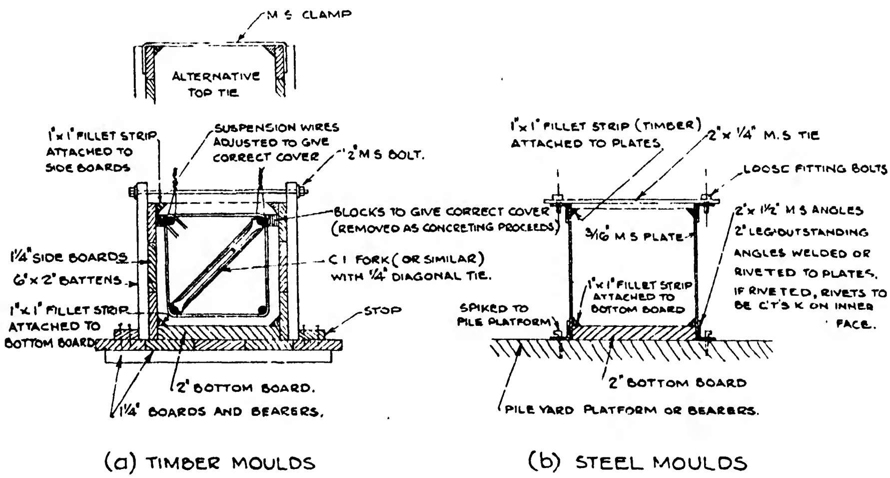


FIG. 218.—SHUTTERING FOR PILES.

to the bottom boards and fix the top fillet pieces to the sides; the top chamfers may also be formed by pressing loose fillet pieces into the wet concrete, or the top edges of the steel side plates may be bent over to form the chamfer. An illustration of a series of moulds with steel sides and timber bottoms is given in *Fig.* 219. As for normal shuttering the moulds should be oiled to facilitate removal; for this purpose crude linseed oil has been successful with timber bottoms and steel sides.

The number of complete moulds required depends upon the least number of piles it is necessary to cast each day to keep pace with the rate of driving, taking into account the specified age of each pile before it can be removed from the mould to the stacking ground. Since the side shutters can be struck at a much earlier date than that at which the piles can be moved, fewer side moulds than bottom boards are required. Well-made pile moulds give long service, as arrangements can be made in their design to facilitate assembly and removal without injury. Bottom boards may give an almost unlimited number of re-uses, and timber sides, well greased, may give up to thirty uses, although fillet pieces may require renewal after four to six uses. Steel sides can also give unlimited uses if not damaged in transit.

The bottom boards are usually laid on a platform formed of stout timbers spaced at a few feet intervals. It is necessary for the platform to be firmly

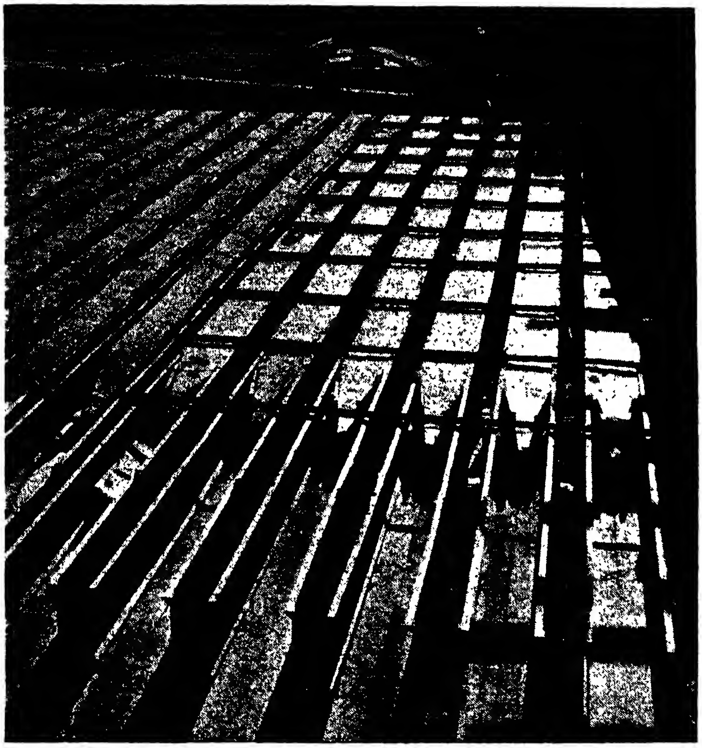


FIG. 219.—PILE MOULDS.

bedded to prevent distortion of the piles due to uneven settlement of the bottom boards. On soft ground heavy piles have been found to exhibit a bow of as much as 1 in. A substantial timber grid helps to spread the load over a larger area. In some cases where the ground is soft it may be necessary to drive pairs of 4-in. by 2-in. stakes at 2-ft. centres into the ground for a depth of 4 ft. or so to obtain a firm bearing. A crosshead carried by each pair of stakes supports the bottom board. A test load should be applied to the stakes ; if the calculated working load on each stake is, say, 200 lb., a test load of 400 lb. should show no settlement.

Reinforcement for piles is usually made up into skeletons and lifted into the moulds in this form. Methods of suspending the skeletons in the forms and of providing the requisite cover are shown in *Fig.* 218. In assembling the reinforcement, the longitudinal bars should bear on the top of the shoe while the tops of all bars should be level to prevent the pile head shattering due to a varying thickness of concrete over the end of each bar. When special hoops preventing twisting or distortion of the skeleton are not provided, it is usual to insert forks diagonally across the pile at about 5-ft. centres. It may be possible to reduce the number required by spacing them at much wider intervals, but it is best to determine a satisfactory number on the site. The forks may be cast iron, pressed steel, or lengths of gas pipe with slotted ends to fit the bars. Pressed steel forks can be obtained up to 20 in. (suitable for a 19-in. or 20-in. square pile). In large piles a link or tie is provided at each fork to prevent the bars spreading. Two forks at right angles should not be placed close together, as a possible plane of weakness may be developed.

The holes for toggles for guiding, and if necessary for slinging, can be provided by casting gas-pipe ferrules through the stem of the pile at suitable points as discussed later.

The concrete in piles is seldom leaner than $1 : 1\frac{2}{3} : 3\frac{1}{3}$. Alternatively, a $1 : 2 : 4$ mix could be used for the bulk of the pile with, say, $1 : 1\frac{1}{2} : 3$ for the topmost 3 ft. or 4 ft., where the driving stresses are most severe. In sea-water and in certain contaminated soils richer mixes may be specified with or without special admixtures, and with Portland cement or aluminous cement. On one important marine contract a mix of approximately $1 : 2 : 2\frac{3}{4}$ was used with an admixture of 2 per cent. of diatomaceous silica and a water-cement-ratio of 0·65. Piles driven into ground saturated with sea-water may be coated with tar ; due to abrasion from the ground while driving, there is a possibility of the tar being removed from proud faces of concrete, leaving hollows, possible airholes, and the like, filled ; the latter, being the surfaces in most need of protection are therefore covered. For exposed piles, as in jetty construction, the advantage of applying tar is not so great. If disintegration or spalling due to damage from vessels or otherwise occurs it is expensive to remove the tar before effecting repairs. If it is not removed the tar may impair the bond between old and new concrete.

The strength of the concrete controls the age at which the piles can be moved and driven, the important times being :

(i) When the side shutters can be struck ;

(ii) When the piles can be removed from the bottom boards and transported to the maturing stacks ; and

(iii) When the piles can be slung and driven.

In normal weather, with proper curing, reasonable times after casting for the performance of the various operations would be :

	Normal Portland cement	Rapid-hardening Portland cement
Removal of side shutters	3 days	24 hours
Removal of piles from bottom boards	2 weeks	3 to 7 days
Driving	4 to 6 weeks	7 to 14 days

The exposed surfaces of the pile should be covered with wet sacking after casting or kept moist by other means to obtain proper curing and adequate strength at the required age. It is advantageous to drive test piles as early as possible, and for this reason it sometimes pays to cast these in aluminous cement, in which case they can be driven in two or three days. When piles are stacked for maturing they should be so arranged that air can freely circulate around each.

The importance and the reward in speed and economy of thoughtfully

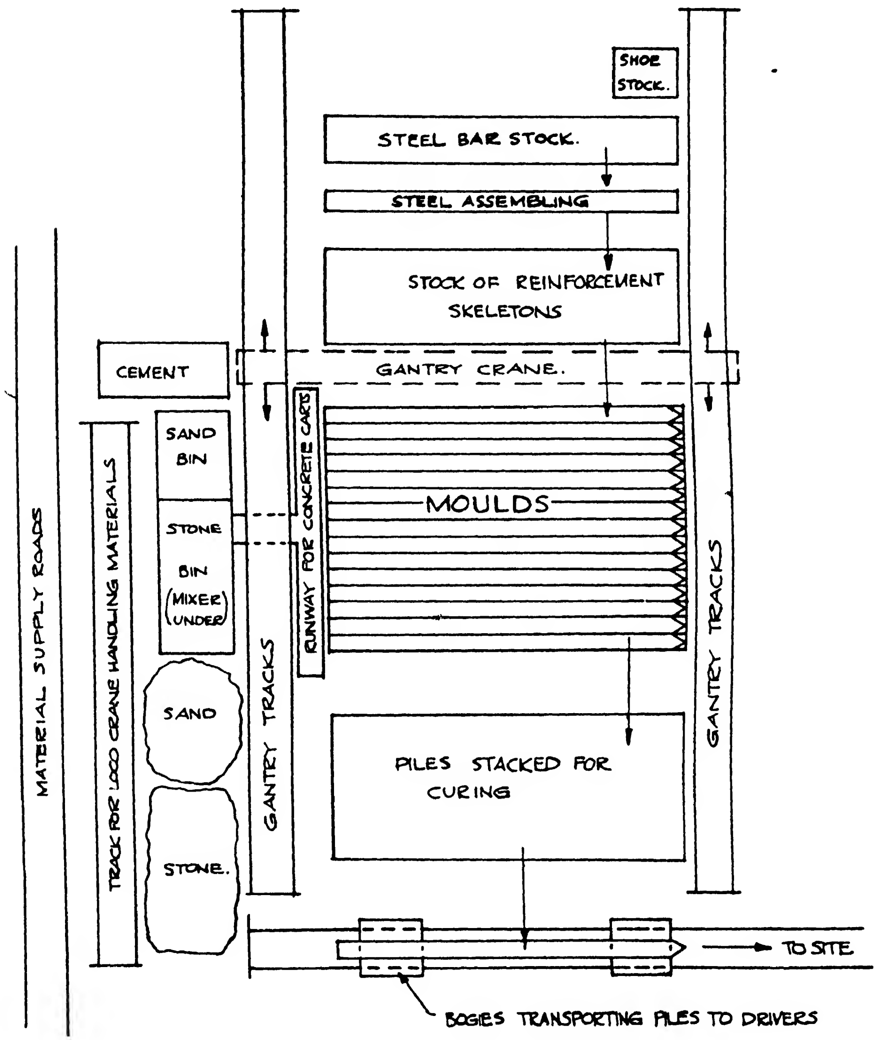


FIG. 220.—PILE-YARD FOR LARGE NUMBERS OF PILES.

arranging the plant for pile manufacture are even more marked than in ordinary construction. Examples of plant arrangements for contracts involving the provision of various numbers of piles are illustrated in *Figs.* 220, 222 and 223 as well as on *Figs.* 9 and 144 in previous chapters. These illustrations cover cases where from fifty to several thousand piles have to be manufactured. The installations in *Figs.* 9 and 144 are both arranged for the manufacture of large numbers, over 3,000 in the former and almost unlimited numbers in the latter example. Such contracts warrant the provision of expensive plant in the form of mobile mixers, cranes, and other handling equipment. The arrangement given in *Fig.* 220 is also applicable to large numbers of piles and involves a stationary mixing plant. This is located midway along one side of the pile-yard and is adjacent to the material supply roads. A locomotive crane running on a short length of track unloads the sand and stone wagons on to heaps alongside the supply roads and replenishes the aggregate bins. These bins feed directly into a batching and mixing plant from which the concrete is transported in concrete carts to the moulds. The area covered by the latter is spanned by a gantry crane which (i) lifts the steel skeletons from fabrication trestles on to stock heaps; (ii) transfers the skeletons from stock to moulds; (iii) transfers the hardened piles to the curing stack; and (iv) takes the matured piles from stock and loads them on bogies for transit to the pile drivers. The tracks on which

FIG. 221.—TRANSPORTING PILES ON BOGIES.

the gantry crane runs are supported on two parallel timber gantries that are built up high enough to allow the crane to clear matured piles stacked six high, and to give sufficient clearance below the tracks for the concrete carts to gain access to the moulds. Such an installation is suitable for dock or similar work where large areas of land are available for the pile-yard in the vicinity of the site.

Pile-yards for bridge construction require special treatment since the area available on the banks of rivers and alongside railways is usually very limited. The principal points to bear in mind are to avoid having to rotate in a horizontal plane either the steel skeletons or matured piles, in handling up to the moulds in the former case and from the moulds to the drivers in the case of the piles. The mixing plant should be as near the material heaps as possible and adjacent to the moulds. The distance from the moulds to the drivers should be as short as practicable; when this distance is too great to be covered by a crane, piles are often brought up to the drivers on bogies mounted on jubilee tracks as shown in *Fig.* 221. In other cases standard-gauge tracks and bogies are used.

Building sites do not often offer much difficulty in arranging the pile yard economically, except in closely built-up areas. The arrangement shown on *Fig.* 222 is suitable for an open building site involving rows or groups of piles. In this case, which represents conditions common in factory construction, the pile yard is located on vacant ground behind the building and pile driving proceeds from the front of the building. The sequence of operations from the supply of materials to the stacking of cured piles can be followed from the diagram. In this instance the piles are transferred to the frames by a mobile steam crane,

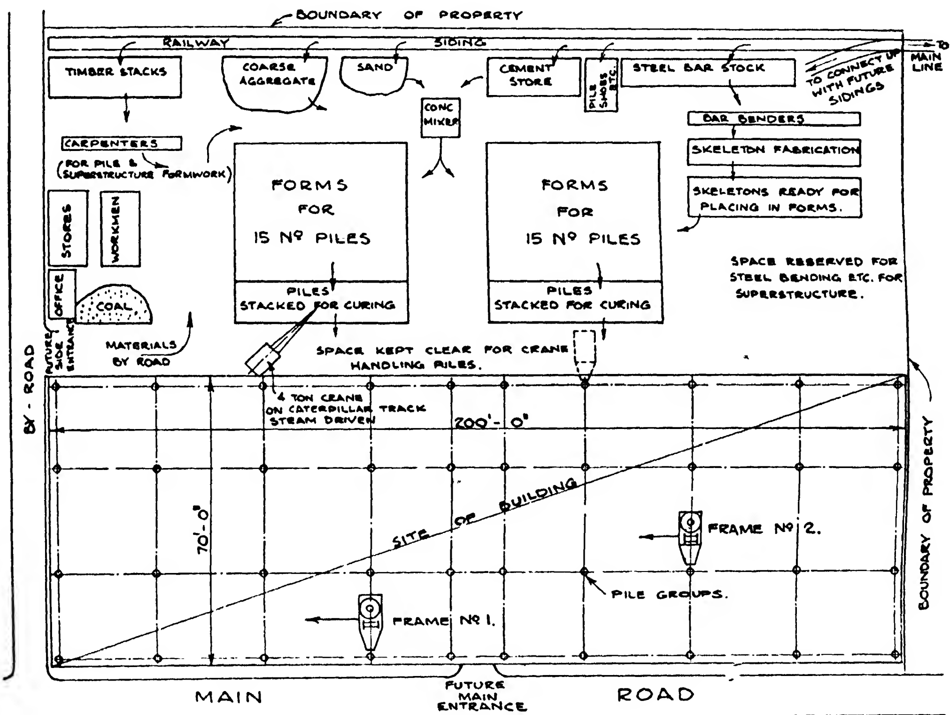


FIG. 222.—PILE-YARD ON OPEN BUILDING SITE.

but bogies running on a jubilee track laid and re-laid to the various piling positions can be used if a mobile crane is not available.

On a confined city site conditions may be very difficult, if as is usual, the piles have to be driven in a basement excavation. In such instances, especially in narrow excavations, a cast-in-situ pile is undoubtedly advantageous. An example of a pre-cast pile moulding yard within the confines of a city building is illustrated in *Fig.* 223. As constructional speed is of first importance, pile driving must commence as soon as sufficient excavation has been taken out to enable the frames to be put in position. Thus excavation and pile manufacture must proceed simultaneously. The following procedure meets these requirements. While the right-hand portion of the site is being excavated, pile making

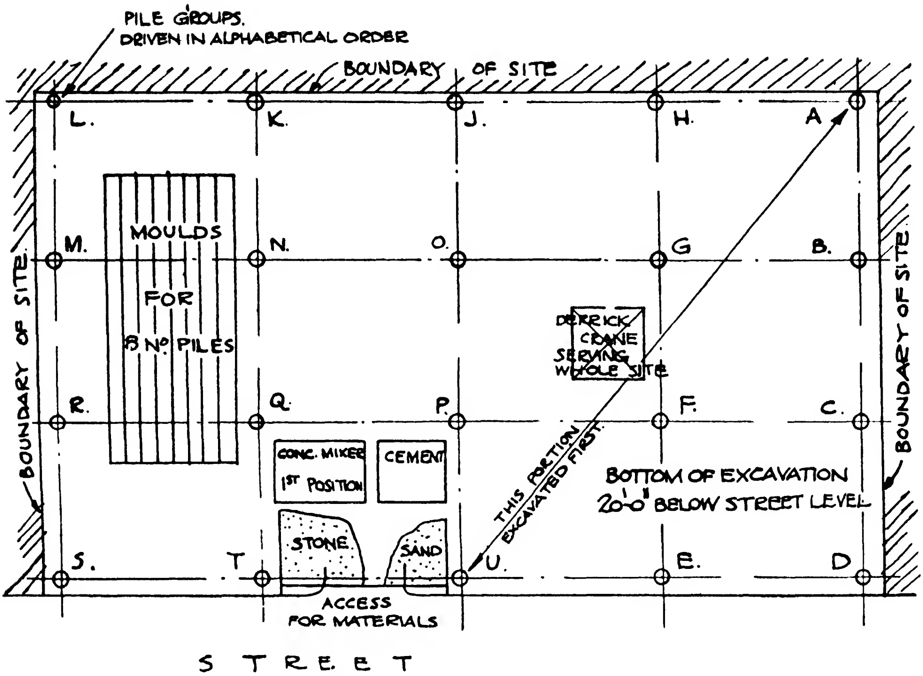


FIG. 223.—PILING ON RESTRICTED SITE.

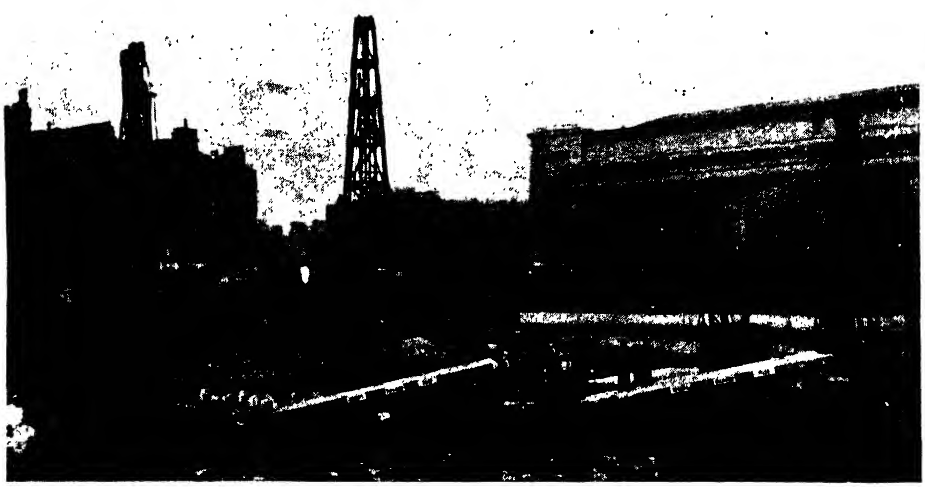


FIG. 224.—PILE FRAMES ON BUILDING SITE.

proceeds on the unexcavated left-hand portion. In the example illustrated, moulds for eight piles could be accommodated. The piles can be stacked for curing in any convenient position on the site, but should be kept horizontal. As soon as sufficient excavation on the right-hand side has been taken out pile driving can commence, pile groups being driven in the sequence A, B, C, etc. Before groups G and H are completed, it should be possible to cast the final piles and stack them for curing, thus enabling the moulds to be dismantled and allowing excavation to proceed unhindered from right to left. The concrete mixing plant can be conveniently situated at road level, about the centre of the frontage, and this would be the last portion to be excavated. When all the piles have been cast the mixing plant can be transferred to the bottom of the excavation and mixing concrete begun for the pile caps and basement floor, this operation proceeding from right to left in the wake of the driving. A derrick or a tower crane, installed at the beginning of the job and located to control the whole site, can handle the piles from the moulds to curing position and thence to the frames. It may also be used to lift the driving frame and mixer into and out of the excavation and to handle skips of excavated material. An example of piling on a city building site with three steam-operated piling frames is shown in *Fig.* 224. In this case the piles were cast off the site and transported to the site by motor-lorries.

Handling and Driving.

Pre-cast piles are normally subjected to higher stresses during handling and driving than when under their permanent bearing load. Therefore these operations must be carried out in a manner that will not overstress the concrete, and for this reason the points of attachment of crane slings or other supports during lifting must be carefully chosen, especially with long piles. The positions indicated in *Fig.* 225 for single-point and double-point lifting give the minimum range of bending moments; that is to say, if the pile is supported at any other points than those specified the stresses in some part of the pile due to bending under its own weight will be increased. There are two conditions to consider with piles suspended at more than one point: (i) when one end of the pile rests on the ground; (ii) when the pile is suspended free from all support from the ground. When lifting the pile off the ground bumping should be avoided, since the instant the toe touches or leaves the ground the bending moments are reversed and the compressive stresses change to tensile and vice versa. If repeated frequently the fluctuation of stress may be detrimental to the pile.

With short piles strict adherence to the points of support given in *Fig.* 225 is not essential as the bending stresses may be quite low. In such cases, say piles not more than 35 ft. long, the toggle hole provided near the top for the attachment to the frame guides may be used for single-point lifting. Longer and heavier piles need more consideration. If from 60 ft. to 80 ft. long they may require three-point suspension, although octagonal piles as long as 93 ft. have been handled with two-point suspension. Piles 75 ft. long have been brought up to the frame with a crane sling attached at two points near the head, the toe being dragged along the ground; while 24-in. square piles 105 ft. long have been lifted by four-point suspension. A pile, suspended at two points, is shown being brought up to the frame in *Fig.* 226 and is shown in position ready for driving in *Fig.* 227.

The slings may be attached by toggles passing through holes formed in the piles or by ropes or chains embracing the pile. If wire ropes or chains are used the pile should be protected from damage by inserting timber or similar packing between the sling and the concrete. In special cases novel devices for attaching slings have been introduced, such as a tee-bolt temporarily screwed into a plate buried in the body of the pile.

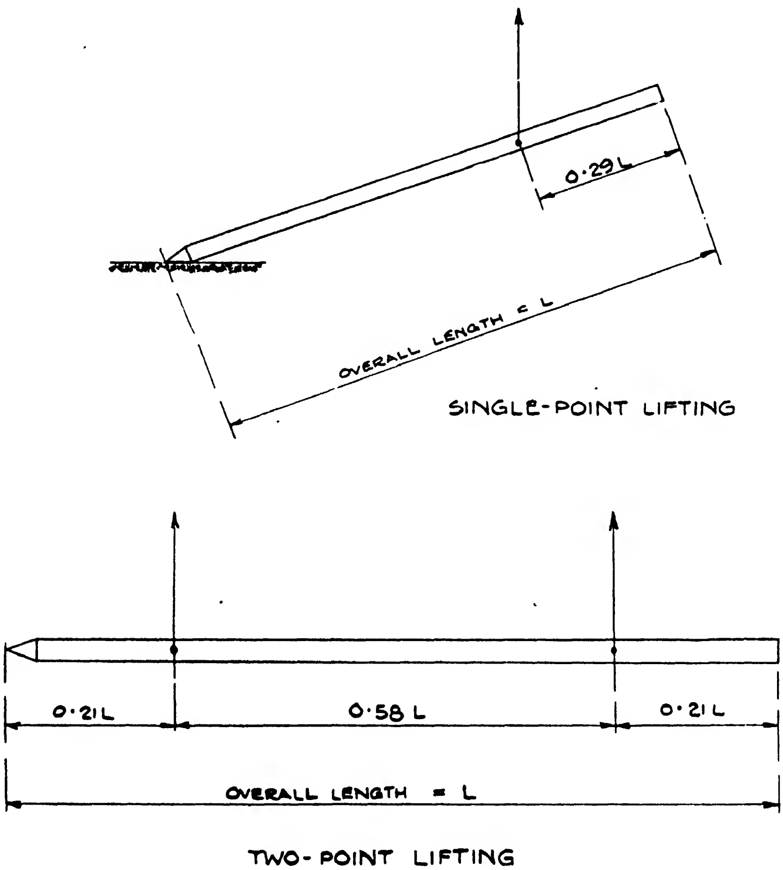


FIG. 225.—LIFTING PILES.

Pile-driving equipment consists essentially of a timber or steel frame with vertical or inclined guides against which the pile bears and upon which the hammer slides. The hammer may be a simple cast-iron monkey operated by a hand, steam, or electrically-operated winch, or it may be a steam-operated single-acting or double-acting hammer. For small light piles a portable double-acting hammer actuated by steam or compressed air as in the McKiernan-Terry type can be used suspended from a crane instead of a frame. During the driving of pre-cast concrete piles the pile-head should be protected by a helmet usually

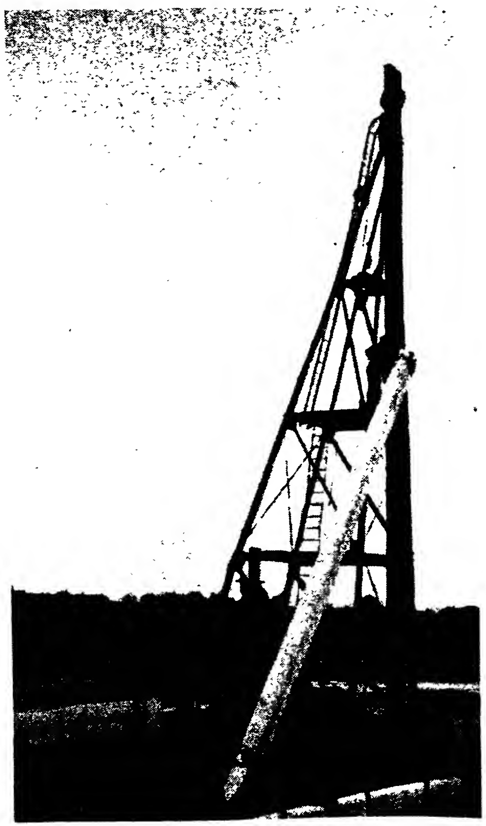


Fig. 226.—Bringing Pile up to Frame.

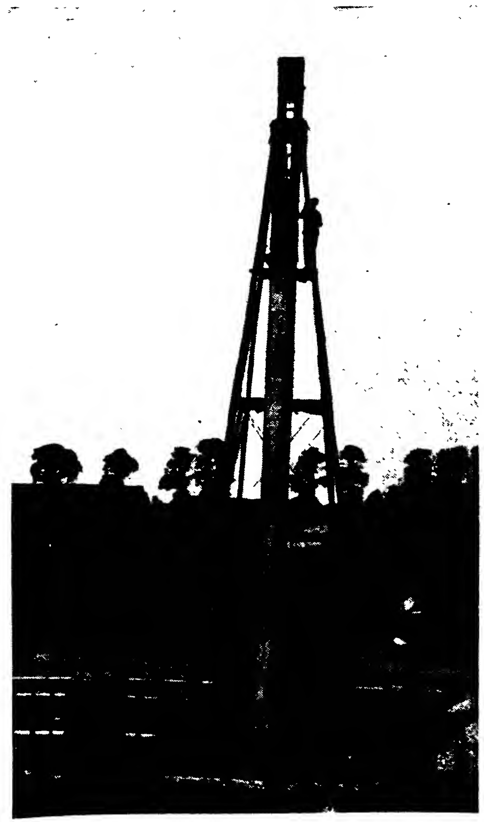


Fig. 227.—Pile ready for Driving.

made of cast steel and similar to that illustrated in *Fig.* 228, which shows a helmet for a 14-in. square pile. The hammer blow is taken on a timber stub-dolly fitted into the top of the helmet, a packing being placed between the latter and the pile-head. Elm is a common material for stub dollies, while the packing can be either sacking or sawdust in sacks for light driving. More effective, since resilience is retained under more severe driving, are such materials as coir mats, coiled hemp rope, felt, and asbestos fibre. When the head of the pile is to be driven below the bottom of the guides or leaders of the frame, a length of timber, 10 in. by 12 in. square in cross-section, is placed above it. This long dolly receives and transmits the hammer blow to the pile. *Fig.* 229 shows the principal components of pile-driving equipment. In this case a winch-operated drop hammer is employed with a timber frame. The hammer is secured to the guides by two bolt and block attachments. The dolly, fitted into a metal helmet, is also kept in position by a similar device, as is also the head of the pile. A timber frame similar to that in *Fig.* 229 can be constructed of timbers ordinarily

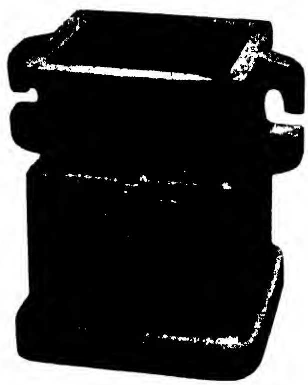


FIG. 228.—CAST STEEL HELMET FOR 14-IN. BY 14-IN. PILE.

available on the site, a suitable size being 6 in. by 3 in. Connections should be made with $\frac{3}{8}$-in. mild steel bolts. The construction of a pile frame from random lengths of timber requires about 100 man-hours, while the assembly and erection of a standard steel frame delivered on the site in transportable pieces requires about 50 man-hours.

The piles shown in *Fig.* 229 are being driven to a batter by packing up the front of the frame sufficiently to obtain the required inclination. When the inclination is considerable, as is often the case in arch abutments and wharf construction, a frame designed for the purpose is employed. A steel frame of this pattern is shown in operation in *Fig.* 230. In this instance a single-acting steam hammer is being used.

Piling-frames are moved from position to position by one of three methods. The frame may be mounted on a four-wheel bogie running on rail-tracks laid parallel to the line of piles to be driven, as shown in *Figs.* 226 and 227. Movement sideways is then obtained by passage along the track. Backward or forward motion, that is movement normal to the line of piles, is obtained by screw adjustment and is therefore limited to a few inches or the free length of the

FIG. 229.—PILE DRIVING.

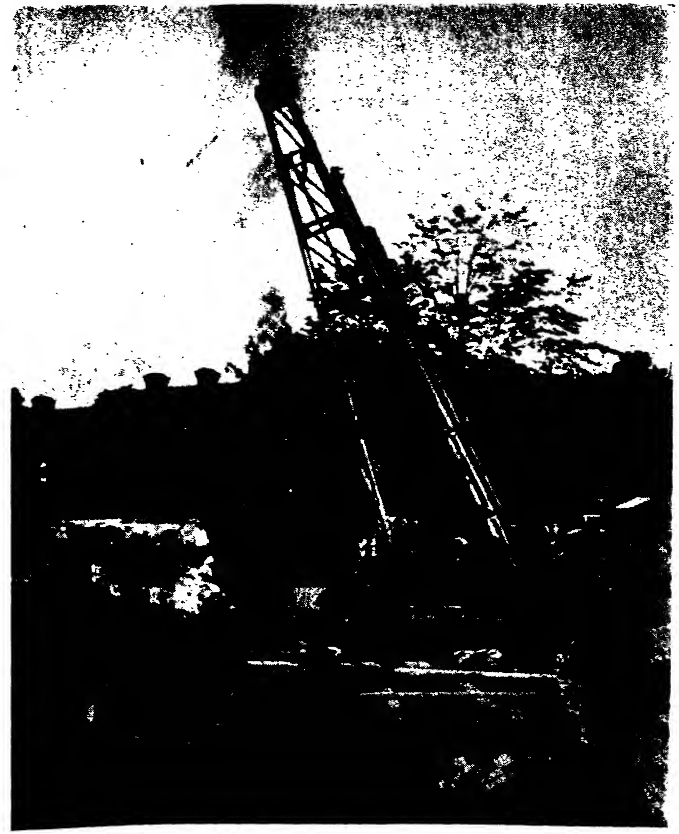


FIG. 230.—DRIVING INCLINED PILES.

screw-thread. Any greater backward and forward adjustments are obtained by moving the rail-tracks bodily. While such provision is suitable for heavy frames, light frames usually depend upon simple hand leverage for backward and forward movements and for small movements sideways, since they are usually mounted directly on a timber frame laid on the ground. Large movements sideways may be obtained by passing a rope from the winch, mounted on the frame, around a " dead man " and back to the frame, the " dead man " being situated in such a position that the rope from it back to the frame is parallel to the line of piles, thus ensuring parallel movement. A third device is adopted on modern equipment where the frame is mounted on two steel rollers. Manual rotation of the rollers by tommy bars through holes at each end of the roller provides backward and forward movement. Lateral movement is provided by power, through a rope fixed to a hook attached to the end of the roller and taken back to the winch mounted on the frame. This type of roller mounting is illustrated in *Fig.* 230.

The rate of driving piles depends on the organisation for handling and serving the frame with piles, the method of moving the frame and the amount of movement required, as well as upon the time taken in actually driving the piles. Records on actual contracts show that under normal conditions four to eight piles can be driven in an eight-hour shift. With severe driving the progress would be less ; in some cases several thousand blows are required to penetrate to the required depth. Hard driving may be very severe on the timber stub-dollies which may need to be replaced once or twice during the driving of a single pile. In normal conditions one dolly may be used for two or three piles. With easy driving and efficient organization the rate of driving can be increased and in some cases eight to twelve piles 30 ft. long may be driven in an eight-hour shift.

During driving, cracking and crumbling at the pile head may occur because (1) the concrete is immature ; (2) the helmet is not squarely seated, the packing having compressed unevenly ; (3) the pile is not plumb ; or (4) the blow from the hammer is too heavy, the fall being excessive. Usually a heavy hammer with a small drop is preferable for concrete piles. Commonly used hammers weigh from $\frac{3}{4}$ ton to 4 tons, although weights outside these limits are used in special cases of exceptionally light or heavy duty. The ratio between the weights of the hammer and pile should be as near unity as practicable and commonly lies within the range of one-third to unity.

Occasionally one pile in a group may penetrate to a much less depth than the others. This usually indicates that an obstruction has been met. The obstruction should be penetrated if it is possible to do so without punishing the pile too much. If the piles are being driven through filling and it is not practicable to drive the pile any deeper, another pile should be pitched and driven nearby, as the obstruction may be a piece of timber or other material offering only temporary support to the toe of the pile. If the obstruction is met in virgin ground where it may be a boulder, it is usually safe to assume a satisfactory permanent support. In all cases of irregular driving a report should be made to the engineer, who may consider bridging over the obstructed pile in such a way that should it settle at any time the load would be distributed among adjacent piles.

When driving piles in a group, it is often difficult to drive the last pile to the same depth as the earlier piles. A recommended procedure is first to drive all the piles to a depth short of that at which the specified set is obtained and then to drive one pile at a time to the specified set.

When piles are required to penetrate compact sand or silt, it is frequently necessary to assist the driving with a water jet. Jets are not effective in gravelly soils. The jet can be arranged by inserting a length of pipe (up to 4 in. in diameter) in the pile, extending from the top to the shoe, thus providing a boring jet. In some cases branches from the centre pipe at 3-ft. centres are provided, these ending in elbows turning upwards along the sides of the pile, thus providing streams of water flowing upwards along the faces of the pile. If the specification requires driving to a given set, jetting should be stopped a few feet short of the expected final depth and driving continued without jetting until the required set is obtained.

The effect of vibration on adjacent buildings due to driving pre-cast piles, especially when severe driving is anticipated, should be considered before a hammer-driven pile is selected. Generally the vibration is transmitted more readily through plastic soils and may be almost negligible in sands. Brick buildings are more likely to be detrimentally affected by small vibrations than are heavy masonry or frame structures. Brick retaining walls or similar structures, where shearing forces along the joints exist, are likely to be impaired by vibrations from pile hammers. Buildings founded on plastic soils may be subject to settlement, since the ordinary load of the building has already compressed the soil until a state of equilibrium between the load and the resistance is reached. Any increase in effective load (which is the effect of vibrations) will upset this balance and produce further settlement.

Cutting-off Pile-heads and Lengthening.

It is usually necessary to strip the concrete from the upper part of a pre-cast pile to expose the reinforcement so that it can be bonded into the reinforced concrete pile caps or the superstructure. The concrete can be broken away most effectively with a pneumatic hammer.

If piles are not driven to the depth anticipated, it may be necessary to cut off the surplus length. The concrete can be removed by shattering with a pneumatic hammer and cutting through the bars with a hacksaw. It is only necessary to remove the concrete immediately above the point where the cut is to be made; alternatively, if all the concrete is cut away, it may not be necessary to cut the bars, the surplus lengths of the latter extending into the superstructure. Another method is to remove by hammer and chisel the concrete cover only at the cutting-off level, the exposed bars being cut through with an oxy-acetylene torch and the remaining concrete core being broken off by four men pulling on a rope attached to the head of the pile; this operation would take about one hour per pile.

To avoid the cost of stripping pile-heads it is practicable on large contracts to cast the piles with the bars projecting a sufficient distance above the head to bond with the superstructure. A special cast-steel helmet is provided having slots to enable it to fit over the projecting bars. A timber cushion (sycamore is suitable) slotted and drilled to accommodate the bars is placed between the helmet and pile head and the hammer blow is transmitted to the helmet by a steel, or a stout timber, dolly which extends beyond the tops of the projecting bars. If a steel dolly is used, a piece of timber should be placed on top to cushion the blow. The dolly itself would consist of a cylindrical shaft with square ends, the lower end being slotted to suit the position of the bars.

When piles are driven deeper than was anticipated it may be necessary to lengthen them in situ and possibly resume driving. When driving is not to be continued, as when the original pile has been dollied down to the required depth, it is sufficient to strip the head to expose a length of steel of not less than twenty-four diameters. This allows the bars in the extension to be lapped with the steel projecting from the pile. In some cases the engineer may require more than twenty-four diameters projection, and when driving is to be continued after lengthening special precautions should be taken to make an efficient splice. A satisfactory method is to strip off the concrete for a distance of 2 ft. or 3 ft. Over the ends of the exposed bars a piece of gas-pipe, not less than 12 in. long, is fitted and into the upper end of this socket the vertical reinforcement is inserted. A bar, say 5 ft. long, is lashed to the old and new bars and the whole of the spliced section is well bound with ¼-in. binding at close centres.

Load-Carrying Capacity of Piles.

The load to be carried by a pile is usually decided by the engineer, who specifies the penetration, set, and other ultimate driving particulars having regard to the conditions under which the driving is conducted. It is useful, however, that the effect of variations in these conditions should be understood by those responsible for the driving, and, without giving any formulæ for the determination of the probable safe load on a pile, some of the factors affecting this load will be considered. In this connection it is convenient to distinguish between a "bearing pile" and a "friction pile," the former depending for its support upon the bearing resistance of the ground under the toe, which may be augmented by an appreciable amount of frictional resistance on the sides. This frictional resistance is not called into play until the pile begins to settle, and therefore not until the bearing resistance has been partly overcome; hence the safe load on a pre-cast bearing pile should be assessed from its bearing resistance. Except in certain dry soils, the frictional resistance during driving is usually considerably less than the frictional resistance that develops when the pile has been left undisturbed for even a few hours while the earth settles about the embedded length. Thus a calculation based on the driving resistance is principally a calculation of the bearing resistance.

Friction piles are usually piles driven into soft ground where little or no bearing resistance can be obtained, the pile being supported principally by the surface adhesion of the soil. The pile will also be afforded a certain amount of support from the ground under the toe and, although the inclusion of this bearing resistance is questionable, it is common practice to add the two resistances to obtain the load-carrying capacity of the pile. Friction piles may not be as reliable as piles driven down to a hard stratum, as the safe load is generally calculated on assumed values for the frictional resistance of the ground, and unless loading tests are made to prove that these resistances are obtained a wide margin of safety must be adopted.

The class into which any given pile falls can be judged from its driving record if a curve, similar to those shown in *Fig.* 231, is drawn to show the relation between the driving resistance and the depth penetrated into the ground. The curve usually follows one of three principal types, although minor variations such as are shown by the broken lines are likely.

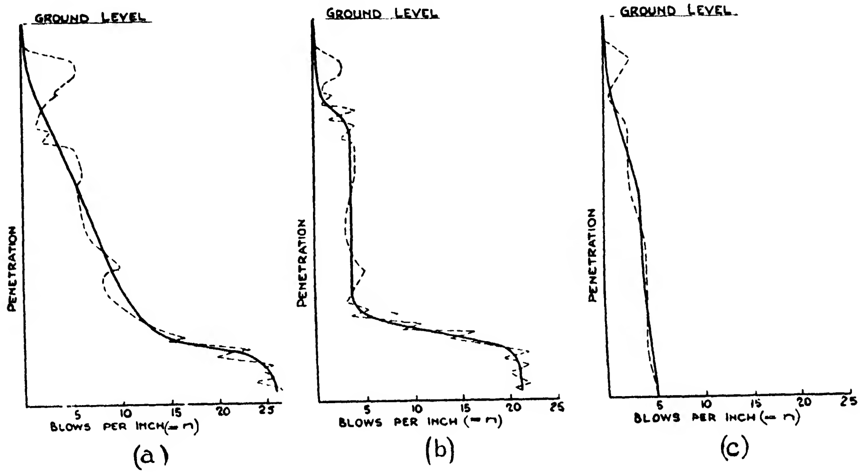


FIG. 231.—CHARACTERISTIC PILE-DRIVING CURVES.

The three characteristic curves are alike insofar that they begin almost vertically, this representing the initial penetration due to the weights of the hammer and pile only. Beyond this point the resistance represented by curve (*a*) increases at a more or less uniform rate until a hard layer is reached, when a definite and considerable increase in resistance is encountered. Curve (*b*) also shows this rapid increase upon reaching a firm layer, but before reaching this stage the resistance has become almost uniform and not very great. After initial variations curve (*c*) shows a steady increase in resistance that never reaches any appreciable figure and lacks a considerable ultimate improvement.

Curve (*a*) is characteristic of driving through firm loamy soil or a tenacious subsoil of poor bearing quality down to a firm stratum. Such soils are evidenced by lack of appreciable bouncing of the pile. When the pile has been allowed to remain undisturbed for a while there is usually little change in the driving resistance.

Curve (*b*) is characteristic of driving through saturated loamy soil, silt, loose filling, or peat, down to a stratum of considerable firmness such as soft rock, gravel, sandy gravel, or compact clay. Before reaching the firm stratum the driving is easy, as the bearing value of the soil is poor and the frictional resistance is negligible on account of the lubricating action of the water in the disturbed soil. After resting for a period the frictional resistance would probably increase severalfold, but the ultimate driving resistance is practically all bearing resistance.

The conditions represented by curve (*c*), usually accompanied by large sets at each blow, occur when driving a pile into wet soils of poor bearing value without reaching a firmer substratum. After a period of rest the resistance of the pile usually increases considerably owing to increase in the frictional resistance, though at the same time the slight bearing resistance encountered during the initial driving may decrease as the compression induced in the soil under the toe during driving relaxes in time. These piles should be considered as friction piles, and their supporting power determined by a loading test.

If a pile has the driving characteristics represented by curve (*c*) but has a small set in the early stages of the driving, and if this set is maintained or improved upon, it is indicated that a satisfactory foundation could probably be designed without using piles.

The load-carrying capacity of a bearing pile is determined primarily by the resistance offered by the ground to further penetration and is assessed from the weight and drop of the hammer and the penetration for the final blows. It is preferable to measure the total penetration or set for a number of blows, say not less than ten, at the end of the driving, and to convert this into the " equivalent " number of blows per final inch of penetration. That is, if the last ten blows produce a penetration of ½ in., the number of blows per final inch of penetration is twenty. It is important that driving should be continuous for some time before the final set is measured. If driving ceases for a period the set upon redriving may at first be much smaller, especially in clayey soils ; it may be about 33⅓ per cent. less after remaining undisturbed for twenty-four hours, and 50 per cent. after three days. The factors that enter into a consideration of the load-carrying capacity of a pile, include the type, condition, and the method of operating the hammer, the severity of the driving, the character of the ground passed through and upon which the pile bears, the inclination of the pile, the size and length of the pile, the shape of the toe, the presence or absence of a helmet, cushioning, stub-dolly or long dolly, and the condition of the dolly at the conclusion of the driving. Numerous pile-driving formulæ have been put forward by various investigators ; the popular idea that different pile driving formulæ, when applied to the same problem, give widely inconsistent results is due to losing sight of the fact that each formula has generally been evolved for a limited range of conditions, the limitations being usually precisely defined by the authors in the original texts.

There are two features to take into account in considering the effect of the type of hammer in an interpretation of pile-driving results, namely, (i) Rapidity of blows, and (ii) Efficiency of fall of hammer. A hammer that subjects the pile to a large number of blows per minute is more efficient than one that gives fewer, as the pile is kept more or less moving throughout the whole period of driving. Thus a simple freely-falling drop hammer would be less efficient in this respect than a single-acting steam hammer, and a hammer operated by a hand or power winch would be of intermediate efficiency.

The efficiency of the drop depends upon the method of operating the hammer ; any loss of efficiency is due to friction on the guides and, in the case of winch-operated hammers, to the retarding effect of unwinding the rope from the drum. Winch-operated hammers are not free from deception, as the winchman can retard the fall of the hammer by a gentle application of the brake. The degree of lubrication of the ram of semi-automatic or fully-automatic hammers determines their efficiency. With these types of hammers operators can also practice deception by steam-cushioning the blow.

The effect of very high hammer drops does not give results directly proportional to the drop when other conditions are constant, but this does not usually enter into a consideration of driving pre-cast piles since drops exceeding 6 ft. are not common. Generally a heavy hammer with a small drop is advocated, but in clayey soils a light hammer and high drop are preferable. High drops involve hammer bounce, probable damage to the pile head, and increased resistance to falling which absorbs energy that cannot therefore be used in overcoming ground resistance.

The effectiveness of the hammer blow is further dependent on the relative weight of the pile and hammer, and this undoubtedly is one of the most important factors in assessing driving resistances. The heavier the hammer the more effective the blow. A hammer equal in weight to the pile is three times as effective as a hammer of half the weight having the same drop, and five times as effective as one of a third the weight. Whereas in driving timber piles equality of hammer weight and pile weight are not uncommon, with pre-cast concrete piles, owing to the greater density of the latter, a hammer equal to half the weight of the pile is more common.

With automatic hammers it must be remembered that the stationary part of the hammer rests on the pile head throughout the operation, and has therefore the same effect as an increase in the ratio of the weight of the pile to the weight of hammer.

The effect of driving piles at an angle is equivalent to lowering the efficiency of the hammer. Since the fall of the hammer is retarded by increased friction on the inclined guides or spindle, the blow is less effective due to friction between the pile and the guides and there is a complex re-arrangement of the frictional resistances of the ground. The driving may be as much as 10 per cent. less effective with batters up to 4 to 1, 15 per cent. for 2 to 1, and 30 per cent. less effective with an inclination of 1 to 1.

The temporary compression in the pile is a function of the pile length and the driving resistance, and it is convenient to include the effect of helmets and dollies and also ground characteristics in considering the resilience losses. The provision of a helmet of normal design with a timber stub-dolly and packing protects the pile head by cushioning the blow, but reduces its effectiveness. This effectiveness is modified if the packing or dolly is damaged. If the stub-dolly brooms in such a way as to increase the cushioning action, the loss of effectiveness may be considerable; ½ in. of "brooming" of the dolly can reduce the effect of the blow to one-half or even one-quarter of its full value. If an oak or elm dolly is provided, brooming is rare and the damage to both the dolly and the packing during severe driving is more likely to be of the nature of compression and flattening, consequently reducing the cushioning effect until the stage is reached where the blow is received directly on the metal helmet without cushioning. In driving effectiveness this is tantamount to having no dolly or packing, but a new dolly is usually fitted in the helmet before this stage is reached in order to protect the concrete.

When the head of the pile is to be driven by a long dolly there is a considerable loss of energy due to temporary compression in the dolly and to bouncing, impact, eccentric blows, and similar effects.

For bearing piles driven in normal conditions the elastic compression of the ground under the toe of the pile is considerable compared with the set of the pile, and the work done in producing the temporary compression uses energy that would otherwise be available for overcoming ground resistance. The elastic compression is a minimum for compact sand, gravel, or rock. Pure sands give less and clays higher compressions, than sandy gravels. Ground that is subject to visibly excessive quake under the hammer blow, such as loamy mud (with a low sand content) and peaty soils, may show several times the elasticity of sandy gravels, whereas this factor is negligible for most rocks. When piles are driven into sand with the aid of a water-jet, the driving records are useless for computing the bearing capacity since temporary artificial conditions are produced during driving. For bearing piles, the length of pile projecting above the ground during driving or the tenacity of the soil driven through before reaching the bearing stratum can be neglected without serious error in calculating the safe load.

The shape of the toe of the pile may affect the severity of the driving. Variations within the range of overall tapers of 1 : 2 to 1 : 1 do not appreciably affect the observed sets, nor, when driving into muds, peat, soft clays, and similar strata is there much difference between blunt or pointed piles. Some effect is evident when driving into sands, gravels, and other compact soils, and when using blunt piles in such soils a reasonable method of adjustment is to consider the equivalent number of blows per inch of penetration as half the observed value. The shape of the shaft or stem of the pile also has an effect on the bearing resistance, and various trials have been made by introducing lugs, collars, conical sections, etc., to increase the resistance of piles in poor ground.

The load-carrying capacity of piles driven to refusal is outside the scope of this book, for when the ratio of hammer weight to pile weight is reasonable the criterion of the working load is the safe load on the pile considered as a column. This is a design problem.

Test Loads on Piles.

The necessity of assessing the load-carrying capacity of piles by actual test in all doubtful cases has already been observed. Various methods of conducting the test have been successfully employed. A common method is to cast monolithic with the pile head a concrete table on which pig iron, rails, bricks, or other available material of known weight is stacked. The magnitude of the test load should be not less than 50 per cent. in excess of the anticipated working load, and to obtain fair results the load should be applied without jarring or impact. A pile is shown under test in *Fig.* 232 where a reinforced concrete loading platform is constructed monolithic with the head of the pile. An alternative design of concrete platform is shown in *Fig.* 233 in which the test load consists of 40 tons of pig iron. Among other convenient methods are testing by a hydraulic jack (*Fig.* 234) or by constructing a wooden bin on the pile head or on a group of piles, and filling it with sand or gravel. The method of using a jack is to place it upon the head of the pile and lay a beam over the jack. The ends of

FIG. 232.—TEST LOAD ON PILE.

FIG. 233.—TEST LOAD ON PILE.

the beams should be anchored down to adjacent piles or other rigid supports to provide a reaction against the upward thrust of the jack.

When piles driven in deep water are to be tested, a successful method is to drive a number of timber piles in a circle around the driven test pile. A tank containing water is supported on this cluster, and during the test the full load of the tank can be transferred to the concrete pile by jacking. If a test pile projects far above the ground level it is necessary to guy the head and loading platform, since unbalanced loading may snap off the pile.

The test should continue over a number of days, measurements of the settlement being made during loading and unloading. The permanent settlement is that remaining after the load has been taken off. A typical record of a pile test showed 0·05 in. settlement at 20 tons, increasing in steps under increments of loading up to 0·20 in. at 80 tons. Progressive settlement, that is, subsidence without further increase in load, was experienced at this load, and at the end

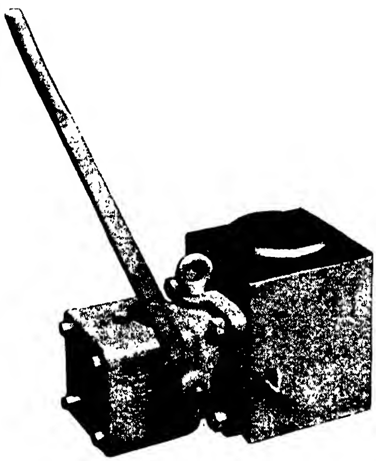


FIG. 234.—HYDRAULIC JACK FOR LOADS UP TO 100 TONS.

of three days a total settlement of 0·30 in. was recorded. For this pile, 80 tons would be considered the ultimate load upon which to base the safe working load of 40 tons. Another pile settled with increase in load up to 0·25 in. with 80 tons. The settlement was not progressive and the load was kept on the pile for some days before unloading commenced. The permanent settlement on removing the load was 0·10 in., indicating a safe working load of upwards of 40 tons. While a test is in progress it is necessary to ensure that the conditions are such that the results can be fairly interpreted. Thus pile driving in the vicinity of the loaded pile should cease, as the vibration might cause misleading settlement. Erratic behaviour of piles under test should be investigated as the cause may have an effect on the ultimate structure. On one contract it was found that under a constant load the piles were observed to rise and fall periodically, the variation being up to $\frac{1}{8}$ in. The piles were driven in wet soil in the vicinity of the coast, the terrain being level flats. The periods of rise and fall were found to occur an hour or so after the time of high and low tide respectively.

CHAPTER VIII

BUILDING CONSTRUCTION

SECTION I.—BASEMENTS.

Basement Retaining Walls.

ONE of the problems encountered in basement construction is the timbering in connection with retaining walls. In city buildings the walls bounding the basement generally extend to the permissible building line, beyond which excavation may be prohibited or at least severely limited. Thus the concrete, usually without shuttering on the earth face, is placed against the face of the excavation and the problem is to support the earth while the concrete is placed and matured. The usual procedure is to excavate a trench along the lines of the retaining walls leaving a dumpling of earth in the centre of the site. The width of the trench would be determined by the width of base required for stability, a dimension that would be specified by the engineer. In all cases, a dimensioned sketch of the proposed method of constructing the wall should be submitted to the engineer to obtain his assurance that at all stages the wall is secure against sliding and overturning and that the concrete is at no time subject to stresses that, due to either immaturity or design, it is unable to withstand.

Two practical methods adaptable to one or two-story basements are illustrated in *Figs.* 235 and 236. In the first case (*Fig.* 235) the wall has not been designed to stand as an independent structure, but requires to be supported until the basement and ground floors have been constructed. A wall of this kind, although requiring less concrete and reinforcement than an independent cantilever, is more expensive to construct. A timbered trench is excavated around the site leaving a dumpling in the middle, the trench being wide enough to provide ample working space and to suit the arrangement of the reinforcement. The base of the wall can be concreted as shown in the first stage. In the second stage successive lifts of the wall stem are concreted, the tiers of struts and walings being removed singly and replaced to provide a support for the shuttering to the face of the wall and the sheeting to the dumpling. In the third stage the stem of the wall is shown completed, with raking struts inserted, thus enabling the horizontal struts to be removed as the dumpling is excavated. The concreting

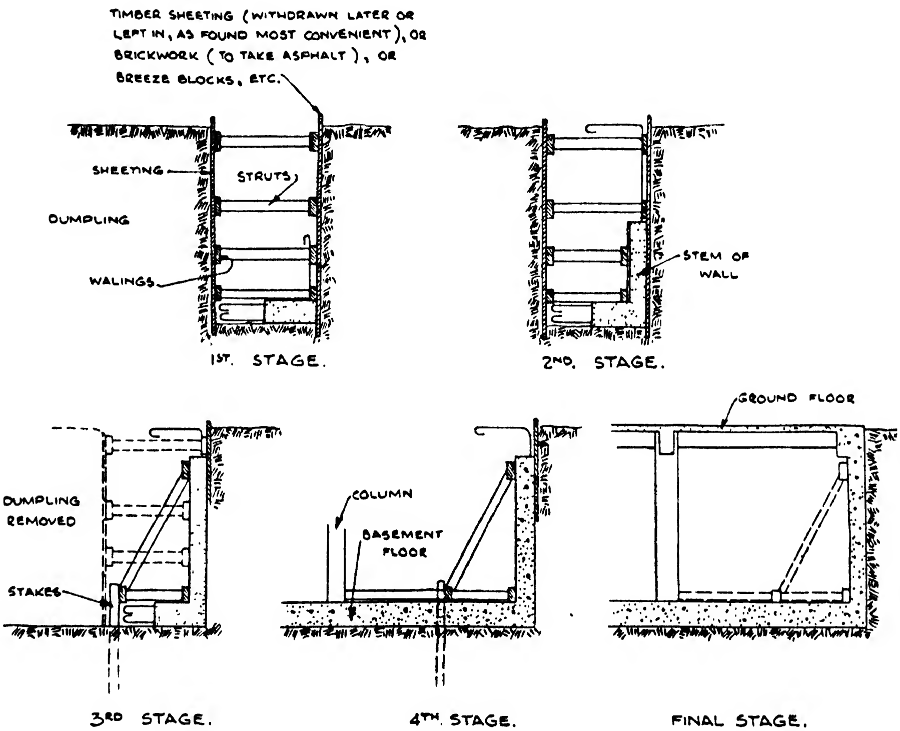


FIG. 235.—BASEMENT WALL CONSTRUCTION (i).
(STABILITY OF WALL DEPENDENT ON COMPLETED CONSTRUCTION.)

of the basement floor can now proceed, the stakes supporting the shoring timbers being removed singly in advance of the concreting and replaced by horizontal struts bearing against the feet of the columns or other convenient projections. The final stage is reached when the columns and ground floor are constructed, thus providing complete support to the wall and enabling the shoring to be removed. With this procedure the earth behind the wall is always supported and the wall itself, while being constructed, is propped against overturning and sliding caused by the lateral pressure of the ground.

The wall in *Fig.* 236 is designed to be self-supporting without assistance from the floors of the building, and so it is only necessary to relieve the concrete from earth pressure until the wall has attained sufficient strength to withstand the stresses. The excavation is taken out on the trench and dumpling system, and timbering, as shown in the first stage, is erected. As the wall stem is concreted, the walings on the right-hand side are removed one by one and the end of each strut is sawn off, the thrust from the dumpling being taken on the

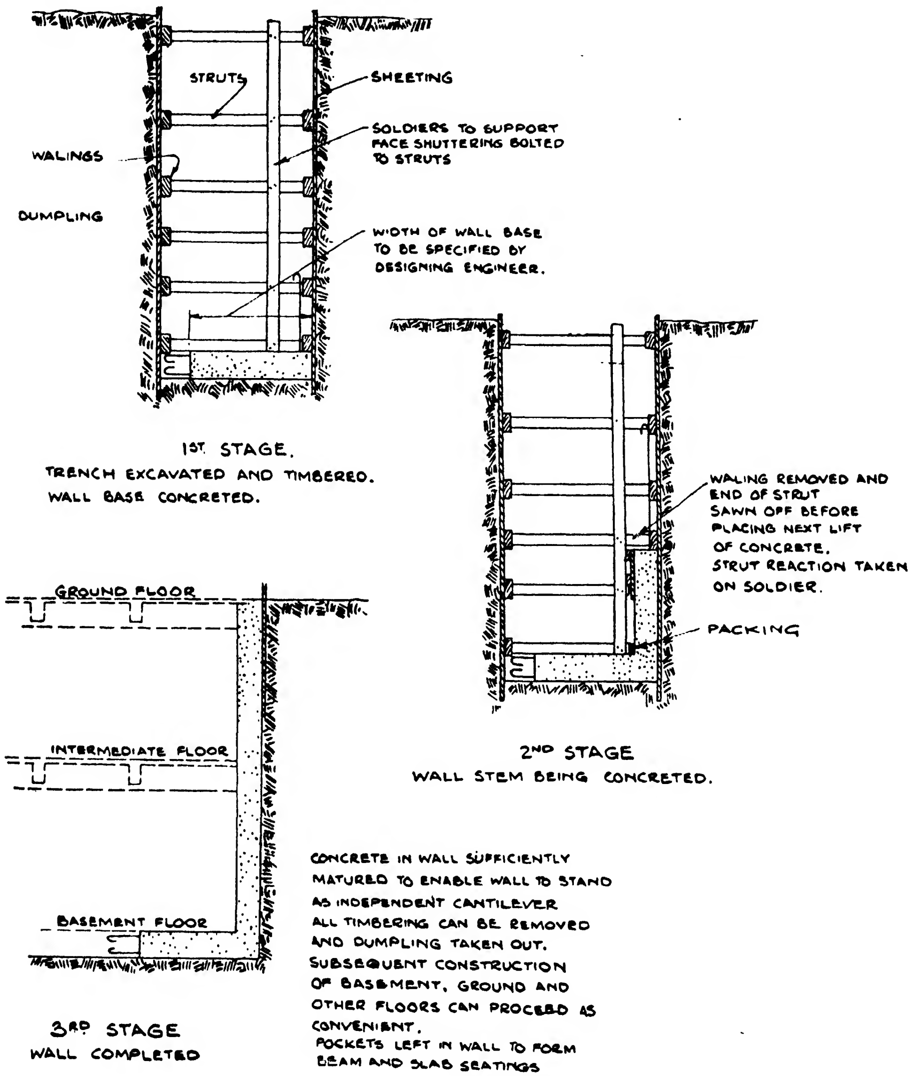


FIG. 236.—BASEMENT WALL CONSTRUCTION (ii).
(WALL CAPABLE OF ACTING AS INDEPENDENT CANTILEVER.)

vertical soldiers. From the latter members the shuttering for the inner face of the wall is wedged. As the concrete in the stem of the wall matures it becomes unnecessary to support the wall any longer, and as the dumpling is removed the timbering in the trench can be taken away. The engineer should be consulted concerning the age at which the concrete will be strong enough to resist the stresses due to earth pressure.

The timber struts across the trench in the schemes shown in *Fig.* 235 and 236 pass between the vertical reinforcing bars in the wall stem. Generally the spacing of the bars can be adjusted to allow the passage of the struts so long as the total number of bars is unaltered. If the proposed method of timbering is known before the design is prepared, the reinforcing bars can often be arranged

FIG. 237.—ARRANGEMENT OF REINFORCEMENT FOR BASEMENT WALLS.

in groups between the struts as shown in *Fig.* 237. Generally the struts would be 8 in. or 10 in. square. In a case where the spacing of the bars cannot be altered and the specified distance between adjacent bars is less than the normal size of a strut, a composite strut consisting of a pair of spaced timbers can be used. This leads to more expensive construction as the two timbers require to be bolted together at frequent intervals. If the spacing of $\frac{3}{4}$-in. bars is 6 in., two 10-in. by 5-in. timbers placed with the 10-in. dimension vertically would be suitable if a 1-in. packing piece is inserted between them.

Waterproofing Basements.

Basements extending below the level of permanent ground water or liable to flooding are usually lined with a waterproofing medium such as asphalt. A common method of constructing the floor of the basement is to cover the bottom of the excavation with a layer of lean concrete on which asphalt or other material is laid, and to build over the asphalt a concrete raft, suitably reinforced to resist the probable water pressure on the underside of the asphalt. Since the waterproofing layer may be damaged while placing reinforcement, etc., it is better to provide two layers of lean concrete. The bottom layer would be the normal blinding layer (see Chapter VII) upon which the waterproofing is spread. Immediately over the latter an additional 2 in. of lean concrete should be laid to receive the reinforced concrete floor as shown in *Fig.* 238.

If water is met during the excavation the opening should be maintained dry by pumping until the reinforced concrete raft has been constructed. Should it be impossible to keep the free-water level below the asphalt level due, say, to breakdown of the pumps or inability of the pumps to deal with exceptional inflow, the asphalt, where not weighted down by the reinforced concrete raft

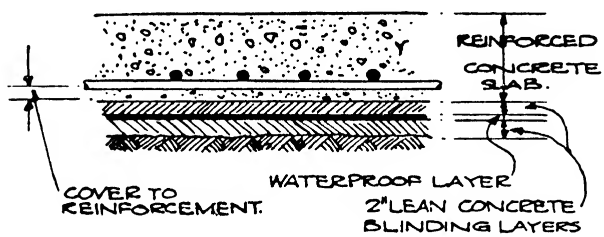


FIG. 238.—WATERPROOFING FOR FLOORS.

above, may blister and in severe cases fracture. It is essential to make good any defects so caused before proceeding with the concreting in order to maintain an impermeable membrane. If a few vent-holes can be cut through the floor slab when exceptional water pressures are encountered, it may be possible to avert damage to the work, as the water is then free to flow over the top of the floor and equalise the upward and downward pressures. When normal conditions again prevail the excavation can be pumped dry, the vent-holes sealed, and the remainder of the construction proceeded with.

Many basements are provided with a permanent drainage sump, the position of which is determined by the architect in relation to the drainage scheme. If a permanent sump is not provided, it may be necessary to construct a temporary sump from which the water can be pumped. The sump should be about 2-ft. square and 1 ft. 6 in. deep and should be placed at the lowest part of the basement excavation, preferably in a corner where the interruption in the floor slab will have least effect on the strength of the floor. Where possible the waterproofing layer should be carried around the sides and the bottom of the sump, as shown in *Fig.* 239, so that the waterproof membrane is unbroken. When no longer required the sump should be filled to the top of the floor with concrete of the same mixture as that in the floor, and embracing any reinforcement that projects from the walls and floor.

Waterproofing for concrete walls of basements is normally applied on the outside. If asphalt is provided, the excavation is usually lined with $4\frac{1}{2}$-in. brickwork or with pre-cast concrete blocks, to which the asphalt is applied in three coats, each $\frac{3}{8}$-in. thick, the joints in successive coats being staggered. The concrete is deposited against the asphalt. When the waterproofing consists of bituminised fabric or similar material, it is often applied directly to the timber sheeting in the excavation, the timber being left permanently in place.

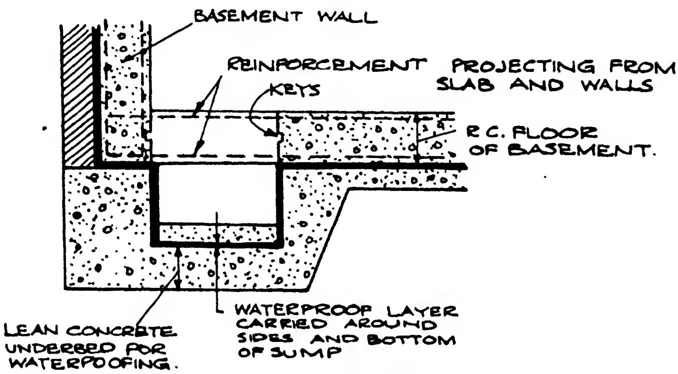


FIG. 239.—TEMPORARY SUMP.

Before ceasing pumping or other measures to keep down the level of the water around a completed basement, it is necessary to ensure that the uncontrolled water will not cause the basement to float, that is, there must be sufficient dead weight in the walls and floor of the basement to counteract the upward pressure of the water. A rough rule for this purpose is that 6-in. thickness of concrete in the floor of a basement will counteract 1-ft. head of water. If sufficient weight is not available, pumping must be continued until the floors, walls, or other construction over the basement are sufficiently advanced to provide the load required to weight the basement down. Alternatively the basement should be temporarily flooded to a depth corresponding to the outside water level. When sufficient superstructure weight has been added the basement can then be pumped dry.

SECTION II.—COLUMNS AND WALLS.

Column Boxes.

Shuttering for square and rectangular columns consists of panels of vertical boards held in position by yokes bolted together in pairs as shown in the details of the assembled box in *Fig.* 240(*a*). A method of framing a beam opening at the head of the column is also indicated, together with the arrangement of a temporary opening at the foot of the column to facilitate cleaning out the column box before concreting commences. It is commonly specified that if the column box is constructed to the full height one side of the box is to be omitted and built up in short lifts as concreting proceeds. The three-sided box is often assembled at the joiner's bench and taken as a unit to the position to be occupied by the column. To preserve the shape of the box during transit, two or three battens may be nailed across its open face and removed when the box is erected.

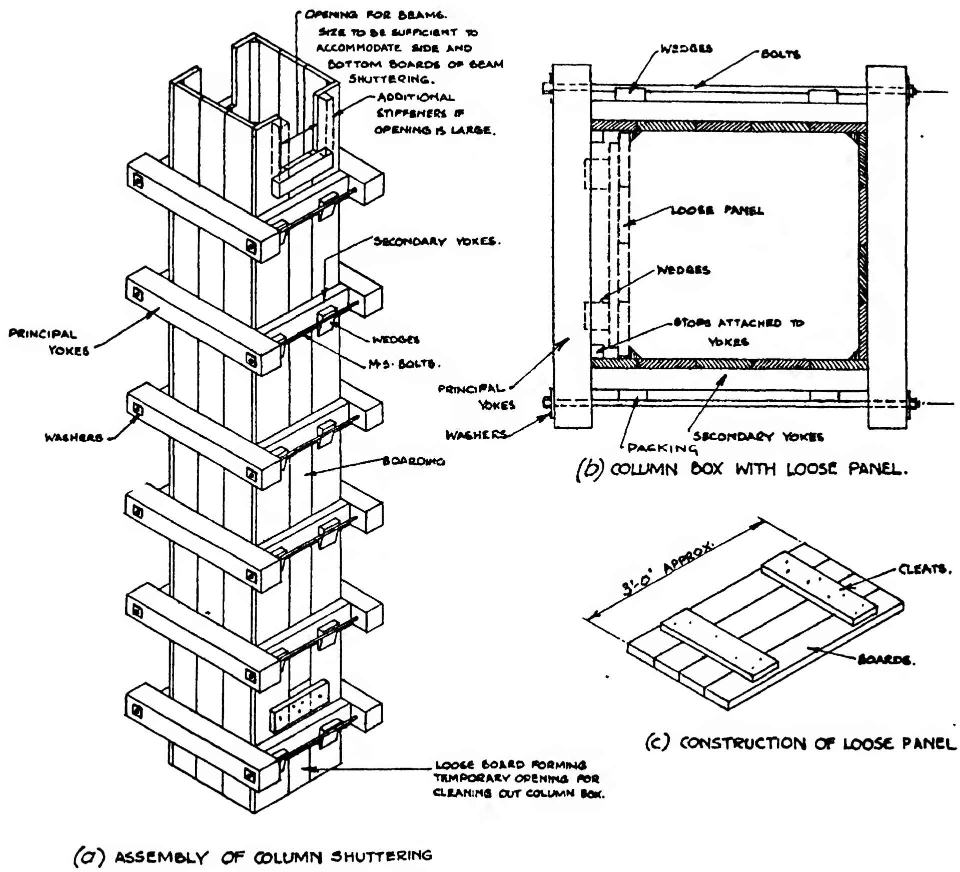


FIG. 240.—DETAILS OF COLUMN BOXES.

Three-sided boxes ready for erection are illustrated in *Fig.* 241. The remaining side is made up by attaching separate panels, as in *Fig.* 240(*b*) and (*c*), about 3 ft. long and equal in width to the column side. These panels are placed in position as concrete is deposited and can be taken down without dismantling the box, thus providing a convenient means of inspecting the face of the hardened concrete before striking the shuttering. Dismantling the box without shock is facilitated by the wedged and bolted connections (*Fig.* 240). The fillet pieces for the four chamfered corners are all attached to the three-sided box.

If concrete placing is limited to lifts not exceeding 3 ft., and if time is allowed

Fig. 241.—Column Boxes Prepared for Erection.

for the initial set to take place in each lift before placing the subsequent lift, the pressure on the shuttering will be maintained at a practical minimum, and the spacing of the yokes can be as given in *Table* XXI.

TABLE XXI.

Spacing of Column Yokes.

Width of column face	1 ft.	1 ft. 6 in.	2 ft.	2 ft. 6 in.	3 ft.
Principal yokes, 4 in. by 2 in. Boarding 1 in.	2 ft.	2 ft.	1 ft. 4 in.	—	—
Principal yokes, 4 in. by 4 in. Boarding 1¼ in.	2 ft. 3 in.	2 ft. 6 in.	2 ft.	1 ft. 9 in.	1 ft. 6 in.
Principal yokes, 6 in. by 4 in. Boarding 1½ in.	—	—	3 ft.	2 ft. 9 in.	2 ft. 6 in.

For such conditions ⅜ in. or ½ in. diameter mild steel bolts with 2-in. by 2-in. or 3-in. by 3-in. washers are satisfactory. For the thickness of the washer ⅛ in. is usually satisfactory, but for washers over 3 in. square, a thickness of ¼ in. is required. For all columns of exceptional size or for rates of filling that give at any one time more than 3-ft. head of wet concrete, equivalent to a pressure of say 400 lb. per square foot, the sizes of the bolts and yokes and the spacing of the latter should be calculated as described in Chapter IV.

Many contractors have their own methods of constructing column boxes, and special circumstances may require departure from conventional design. For example, in the shuttering of the large columns illustrated in *Fig.* 242, in place of the usual horizontal yokes the boarding is supported on vertical timber joists wedged against a surrounding frame of tubular steel scaffolding. When this arrangement is applied to large columns, the shuttering is easily dismantled and the waste due to cutting timber into the short lengths required for horizontal yokes is avoided.

Vertical stiffeners for a rectangular column box may also be adopted in the manner shown in *Fig.* 243, where the 5-in. by $2\frac{1}{2}$-in. verticals are supported by wire ties connecting opposite timbers. For large columns, the wire ties could be replaced by bolts. The use of wire ties in the design of a small column box is shown in *Fig.* 244. In this case a three-sided box is erected complete with fillet

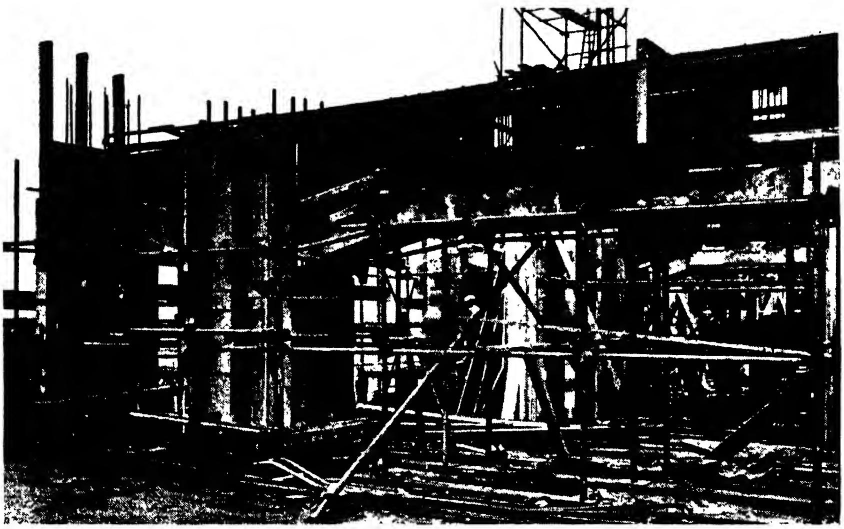


FIG. 242.—SHUTTERING FOR LARGE COLUMNS.

pieces and wire ties across the two opposite faces of the three sides. At intervals boards are nailed across the open side to preserve the shape of the box and to provide anchorage for the ties attached to the intermediate face. The ties pass through holes in the boarding and are maintained taut by attaching them to nails hammered over to secure their ends. There are many occasions in industrial construction and similar work when there is no objection to the wires passing through the columns; column boxes such as are illustrated in *Figs.* 243 and 244 are then economical to erect and dismantle. After the shuttering has been removed, it is essential to cut off the projecting ends of the wire ties close to the surface of the concrete. It is preferable to clip the wire off about an $\frac{1}{8}$-in. below the concrete face, making good the small cavity that is left by mortar, which will be covered by subsequent grout washing or other surface finish. Alternatively the wire can be cut off flush with the concrete face and tapped into the latter by the ball end of a hammer; subsequent grout washing will conceal the slight depression. The column box detailed in *Fig.* 243 is suitable for a column 4 ft. by 2 ft. in cross-section, while that in *Fig.* 244 is suitable for columns up to

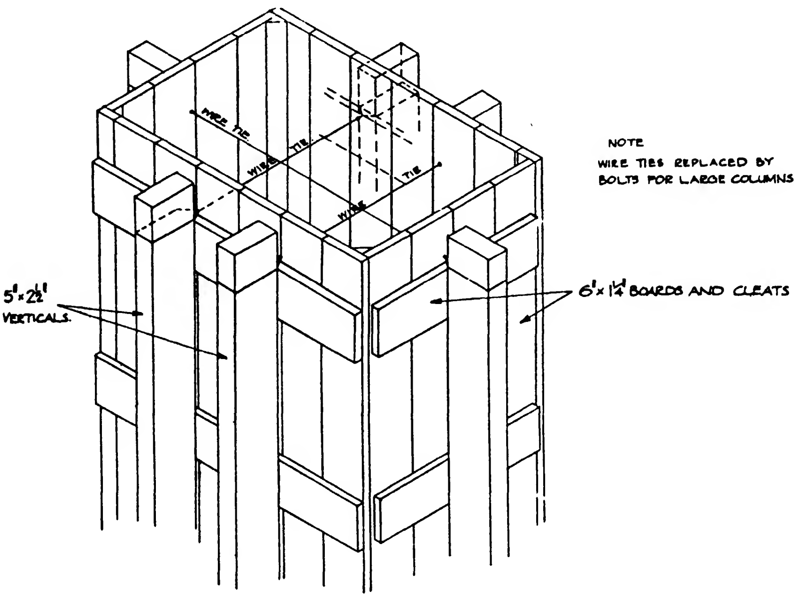


FIG 243.—COLUMN BOX WITH VERTICAL STIFFENERS.

18 in. square. In the latter design the open face is closed by nailing boards across it as concreting proceeds.

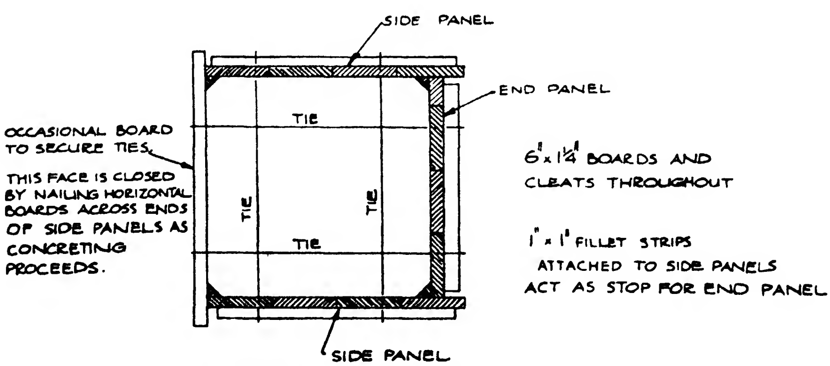


FIG. 244.—SMALL COLUMN BOXES WITH WIRE TIES.

Circular columns, if not cast in metal moulds, may be shuttered as shown in *Fig.* 245. A series of circular collars, placed at 2-ft. to 3-ft. centres, forms a template for the longitudinal lagging which is formed out of 2-in. by ½-in. (or smaller) laths.

In place of bolts or wire ties, one of the numerous designs of patented metal column clamps may be used. The advantages of these are that they can be fixed and removed easily without shock. Offset against these advantages is the first cost, but if numerous reuses of the clamps are possible money is saved. If care is exercised in handling and loading the clamps for transit they can be used an almost unlimited number of times without replacement. As a joiner and a labourer can make and fit in one hour four or five frames composed of timber yokes with bolts, and a joiner can fix twenty clamps or more in an hour, a clamp used a dozen times and costing less than £1 will pay for itself. In *Fig.* 246 an uncommon but useful combination of bolted and clamped connections is illustrated. The screw clamps in this case cost only a few pence each. The collars embracing the four sides of the column box are made from boarding and are

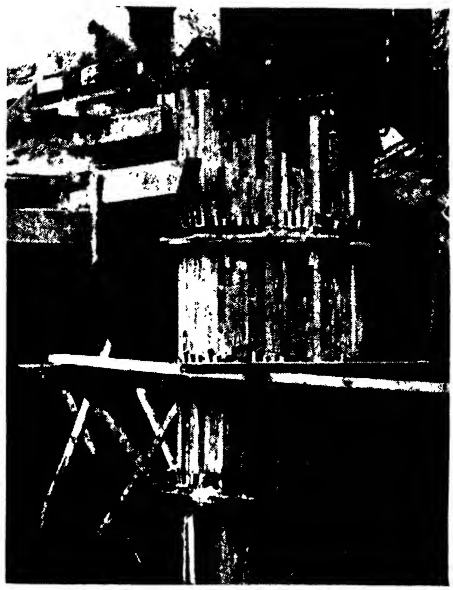


FIG. 245.—SHUTTERING FOR CIRCULAR COLUMN.

secured at each corner by one or two clamps. Outward movement is resisted by the bolts passing between the short vertical members bearing against the collars.

Tall exposed shuttering must be guyed against wind and column boxes should be well braced to prevent movement of the head or foot which may result in distorting the shuttering above. The beam boxes usually effectively tie together the heads of a series of columns, and diagonal braces extending from the top of one column to the base of an adjacent column prevent general racking. Such a system of bracing is seen in *Fig.* 246, while the bracing necessary for a single row of exposed column boxes before the beam shuttering is erected as indicated in *Fig.* 247.

In multi-story buildings the column sizes decrease in each story, and much costly remaking of shuttering for columns, and to a less degree for beams and slabs,

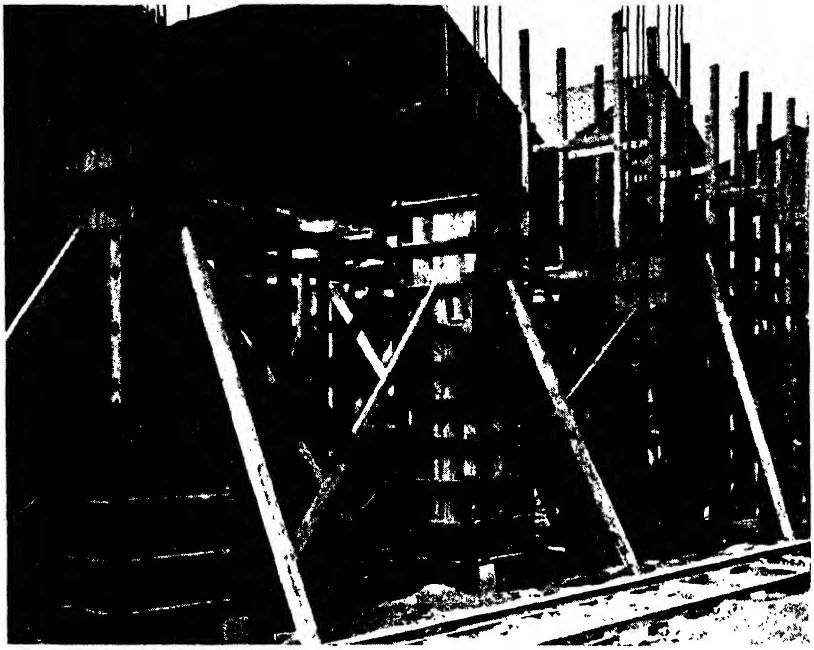


FIG. 246.—CLAMPED COLUMN BOXES.

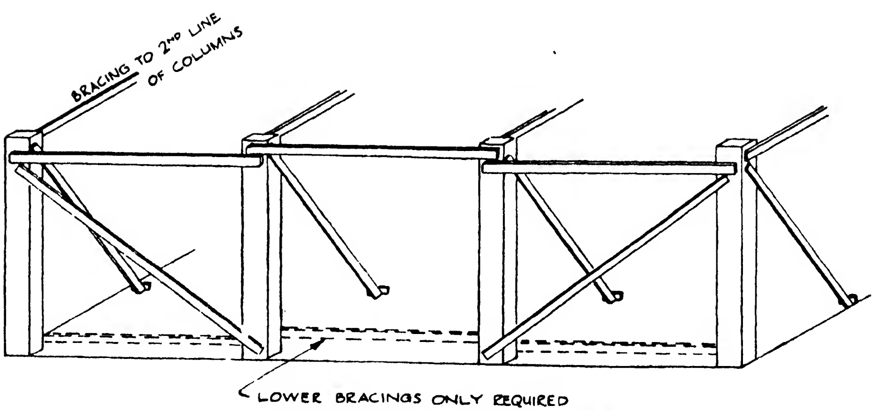


FIG. 247.—BRACING COLUMN BOXES.

can be avoided if the shuttering for the lowest story is designed so that it may be simply adjusted for use in upper stories. *Fig.* 249 indicates a method of making up the deficiency in the slab and beam shutters as the column boxes decrease in size, and *Fig.* 248 shows the construction of the first-story column box to

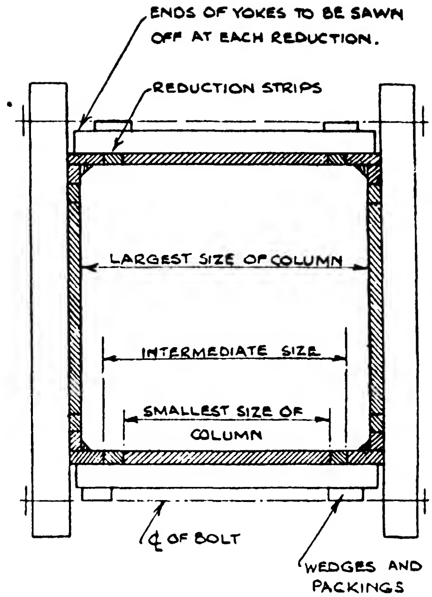


FIG. 248.—ADJUSTMENT FOR DECREASING COLUMN SIZES.
(i) COLUMN BOXES.

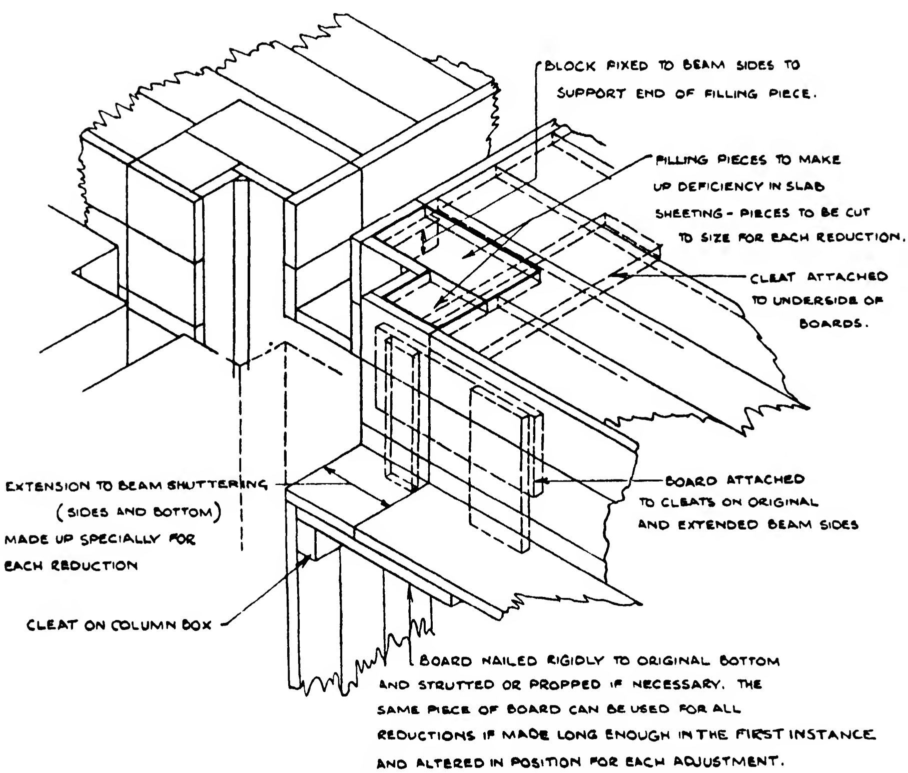


FIG. 249.—ADJUSTMENT FOR DECREASING COLUMN SIZES.
(ii) BEAM AND SLAB SHUTTERING.

allow for successive reductions. It is not always possible to make a box that is adaptable for more than four different sizes, but when the reducing strips have been removed the remaining timbers are generally unmutilated and may be used for remaking into smaller boxes. When the column sizes are reduced it becomes necessary to provide packings behind the wedges as the positions of the bolts are unaltered, although it may be worth while boring new holes in the yokes if the reduction exceeds 6 in. Unless the bolts have a generous length of thread it will be necessary to provide packing pieces between the nut and yoke. As a rule the additional timber required to extend the beam and slab shutters up to the receding column faces is usually discarded after each use in order to avoid joining several pieces together, which is not only costly but leads to weakness.

Shuttering for Panel Walls.

The primary force to which wall sheeting is subjected is the outward pressure of the contained wet concrete. In thin walls sheeted on both sides the shuttering on opposite faces is connected by bolts or wire ties as described in Chapter IV, so that the outward pressures react against each other, making it unnecessary to provide external props or struts to obtain stability, although occasional props may be required to maintain alignment.

There are various ways of constructing wall shuttering. If the panel is only one story high a double line of vertical soldiers is erected from ground-level or from floor-level, and horizontal boarding is nailed to these as shown in *Fig.* 250. The soldiers are plumbed vertically or, if the wall is inclined or tapered, follow the rake of the adjacent face. The ties between opposite faces are placed at the vertical joists. It is usually most convenient to attach the sheeting for one face for the full height of the wall before starting to place the concrete, and to board up the other face, two or three boards at a time, as concreting proceeds. The open face would naturally be the one that is most easily approached from the concrete mixer. Fixing the wall and wall-column reinforcement, which is seldom heavy or intricate, takes place unhindered before the erection of the shuttering.

Soldiers should not be less than 4 in. by 2 in. in section ; depending on the height of the wall, the spacing of the soldiers, and the number and position of the rakers, any section up to, say, 9 in. by 3 in. may be used. Their spacing depends on the thickness of the boarding and the rate of placing the concrete if this is continuous. For normal conditions when the concrete is placed in daily vertical lifts of 3 ft. or 4 ft., the spacing of the soldiers would be as given on *Table* XXII.

TABLE XXII.

SPACING OF WALL SOLDIERS.

Boarding	Spacing of verticals	
	More than two continuous spans	Single span or two continuous spans
	ft. in.	ft. in.
1-in. wrought boarding	1 8	1 5
1¼-in. ,, ,, ...	2 0	1 9
1½-in. ,, ,, ...	2 4	2 0

If the concrete is to be consolidated by vibration the verticals should be spaced at not more than 18-in. centres and the boarding, preferably tongued-and-grooved, should be screwed instead of nailed to the soldiers.

The economical spacing for any case can be calculated by using the design table in Chapter IV.

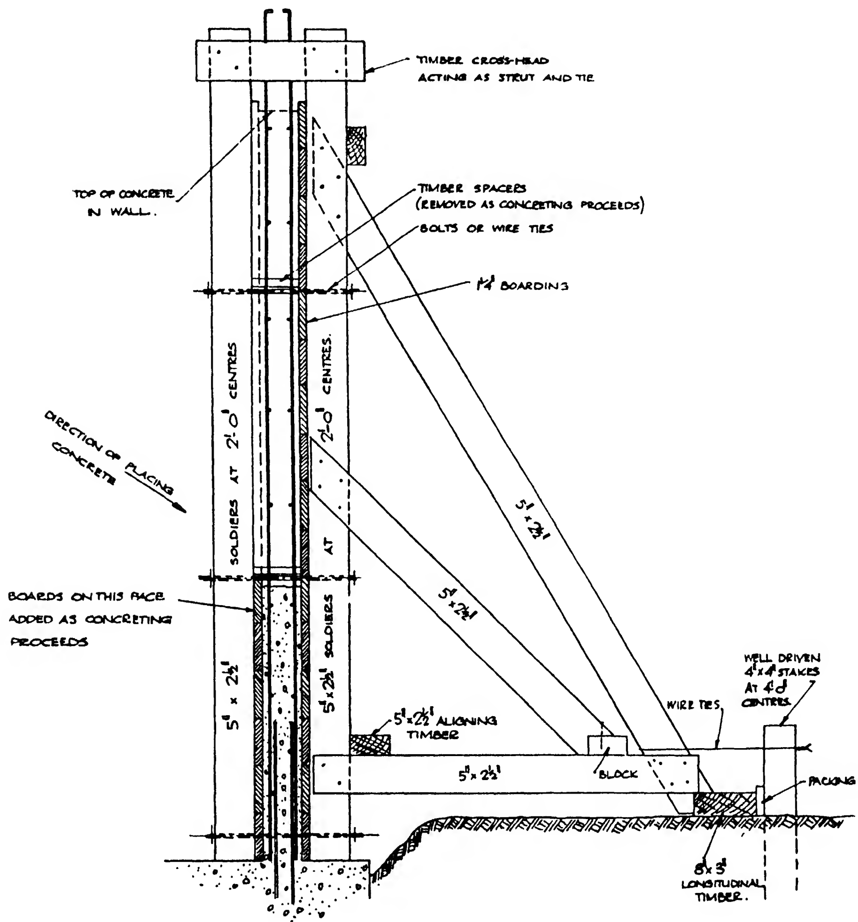


FIG. 250.—SINGLE-STORY WALLS.

Timber boarding which is to come in contact with the concrete should be wrought on two edges and at least one face and should be close jointed, but, where insulating or other linings are to be fixed before concreting, rough boarding with open joints is permissible. Notes on the use of tongued-and-grooved boarding and steel shuttering for walls are given in Chapter IV. Plywood or other linings for producing a fair face are described later and also in Chapter IV.

In *Fig.* 251 a number of the foregoing remarks are illustrated. An insulating lining is provided on the inside face of the wall, and the sheeting behind the lining consists of rough timbers. Nails driven through the lining and projecting into the concrete secure the lining permanently in position. A fair surface on the exposed face is produced by using steel shuttering stiffened and aligned by horizontal timbers.

Where a wall extends through several stories with an unbroken face, a convenient method of constructing the shuttering is to make a series of easily-handled

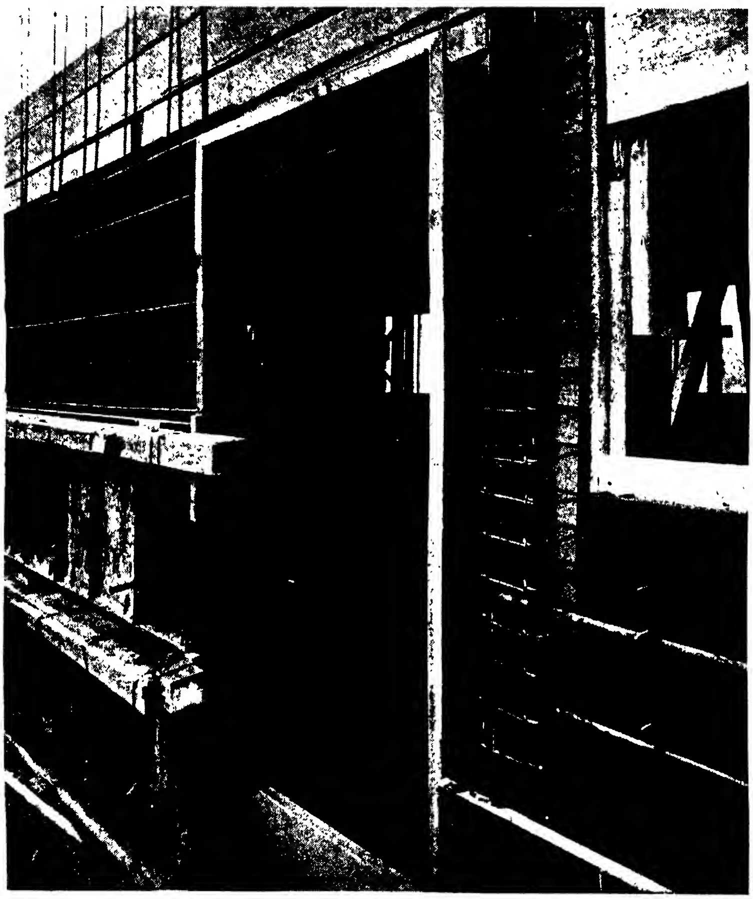


FIG. 251.—PANEL WALL CONSTRUCTION FOR RESIDENTIAL BUILDING.

panels, say, 3 ft. deep by 6 ft. long, or of such other lengths as are determined by vertical features. Each panel may consist of horizontal boarding attached to 4-in. by 2-in. vertical stiffeners kept in position by 4-in. by 3-in. horizontal walings (*Fig.* 252). A substantial unit like this can be erected and struck many times without damage and is suitable for concrete placed with external vibration, in which case tongued-and-grooved boarding screwed to the verticals would be necessary. Panels on opposite faces are connected by wire ties or bolts at the top and bottom to resist the outward pressure of the concrete. Before concrete is placed a timber spreader is fixed at the top to keep the two panels apart and the tie taut. At the bottom, the hardened concrete in the portion of the wall already concreted acts as a spreader. The top spreader must be removed when the level of the concrete approaches the top of the shuttering. When commencing to place the bottom lift of the wall a spreader is needed at the bottom; a mortar block, say, 1 in. square in section and in length equal to the specified wall thickness, is very suitable for this purpose, as it can be left in the finished work. If a timber spreader is used it must be removed before it is buried in the concrete. This is often troublesome in narrow walls.

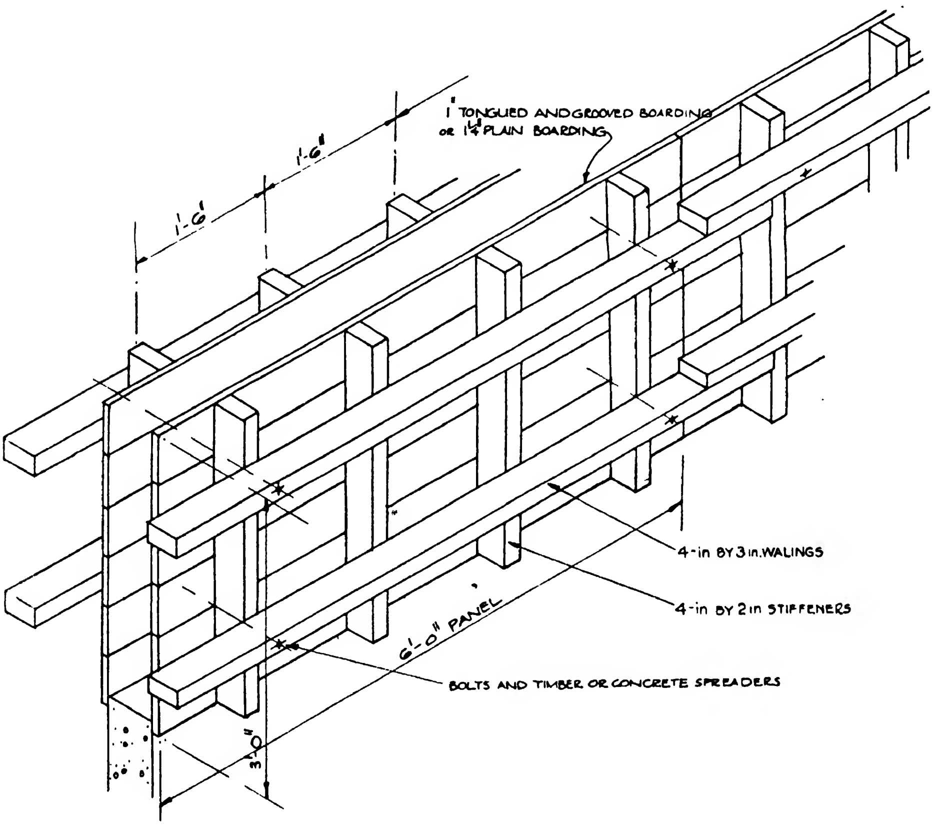


FIG. 252.—CONSTRUCTION OF HEAVY PANELS.

The bolts connecting opposite panels should pass through the vertical members, but where this is impossible owing to the position of the reinforcement or fixtures the bolts should be placed as near the verticals as possible. If bolts are used the weight of the timber panels is carried by the lower bolts resting on the completed portions of the wall, but if wire ties are used an independent support is necessary. This can be arranged by propping from the scaffolding or from lower floors. The panel units for the walls of the structure illustrated in *Fig.* 23 (Chapter I) are supported on putlogs passing through the walls, but these supports need not be provided at every lift. In the example referred to, they are provided for alternate lifts. The construction of a light panel of wall shuttering suitable for normal 4-in. or 5-in. walls is illustrated in *Fig.* 253. This panel is made completely from boarding connected by wire ties used in conjunction with timber or concrete spreaders.

Openings in walls for windows, doors, etc., are easily formed if there are no sills, heads, or other projections around the opening. If the openings are small in comparison with the area of the wall, or are irregular in position or few in number, the wall should be sheeted all over on both faces and a framework, made from timbers of a width equal to the wall thickness, inserted between the shuttering panels. Where the openings are placed regularly or are large in area the wall sheeting can be constructed to cover the net area of the wall only.

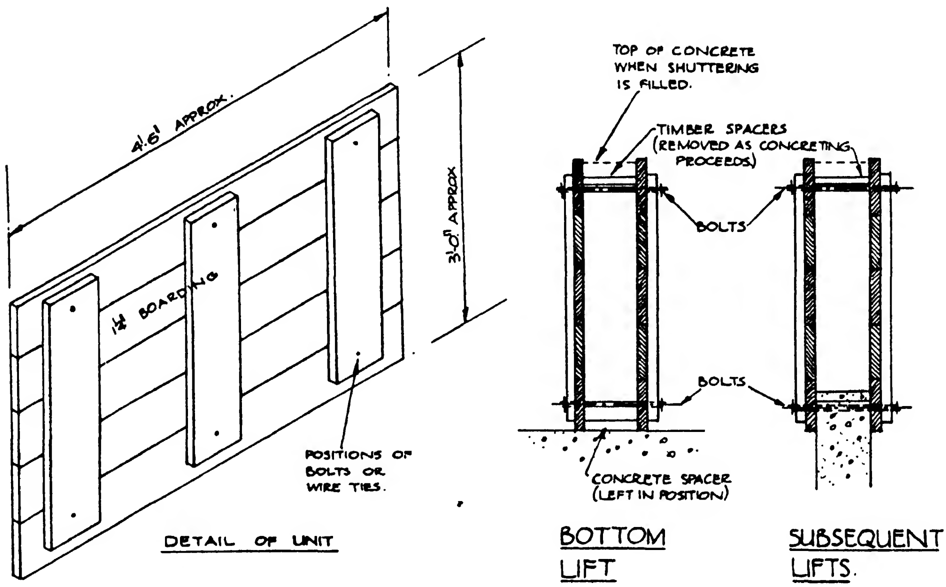


Fig. 253.—Construction of Light Panels.

Fig. 254 shows alternative arrangements for forming openings in walls and indicates the stiffening required at the corners of the frames. Fixing blocks for windows and doors must usually be built into the concrete walls ; *Fig.* 254 gives a detail of a hardwood block suitable for this purpose. The blocks, shown in position in (*b*), should be lightly attached to the framework by nails so that when the frame is removed the block remains in the concrete without undue strain on the material in which it is embedded. In using fixing blocks care should be taken to ensure that the cover of concrete on the adjacent reinforcing bars is not reduced below what is specified.

Instead of using blocks for fixing door-frames, bolts may be built into the concrete while it is being placed. Holes drilled through the temporary framework forming the opening keep the bolts in position until the concrete hardens. For heavy doors, or doors in industrial buildings where rough usage is anticipated, building in bolts is undoubtedly better than attaching them to fixing-blocks. If bolts are left projecting the threads should be greased freely before placing the concrete so as to prevent spilled mortar adhering to the metal, as this is extremely difficult to remove.

Tongues attached to the frames are used to fix some types of pressed-steel door frames (*Fig.* 251). Since frames of this kind are erected at the same time as the shuttering, they should be on the site in ample time to avoid delaying the progress of the work. Attachments for fixing door frames in external walls should, unless otherwise specified, be placed so that the door-frame is flush with the inside face of the wall.

Window-frames are sometimes erected as the concrete work advances, but where openings are to be left for doors or windows to be fixed later a clearance is

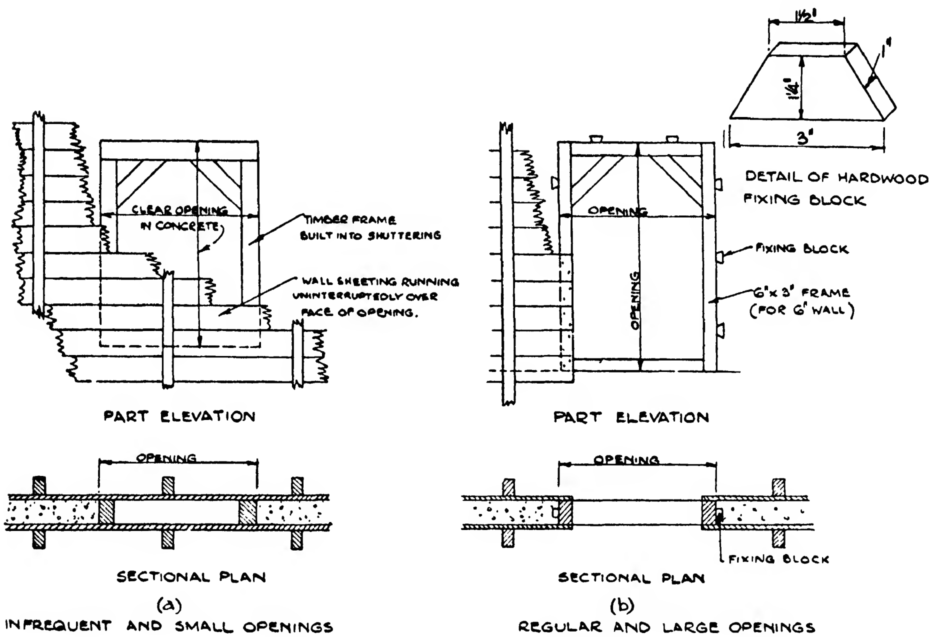


FIG. 254.—OPENINGS IN WALLS.

required over the net size of the frame for fixing. With steel window-frames, a clearance of $\frac{1}{8}$ in. at the top, bottom, and sides is advisable over and above the manufacturer's specified size, with an additional $\frac{1}{4}$ in. for each transom or mullion. The most common methods of fixing steel window-sashes in concrete walls are shown in *Fig.* 255. That shown at (1) is usual in industrial buildings. A wood or lead plug or a steel lug is let into the concrete opposite the screw-holes in the frames. If provision for a plug has not been made before concreting the wall, a Rawlplug is the most satisfactory connection. The space between the frame and the face of the concrete should be packed with mortar, and the gap between the outside edge of the frame and the concrete should be pointed with cement or mastic. If this method is adopted the opening in the concrete must be truly rectangular.

The alternative method shown in *Fig.* 255 (2) allows some tolerance in the size of the opening and is suitable when the exact size of the window frame is not known when the wall is concreted. A rebate is cast in the sides of the window opening. The frame is secured by steel lugs attached to the frame and buried in the mortar filling the rebate. The latter is readily formed by attaching a shaped strip to the shuttering of the opening. Most metal window frames are attached at the heads and sills as well as the sides, the detail at the head and sill being as shown in *Fig.* 255 (3).

Cast-in-situ concrete walls for balconies commonly occur in the construction of modern residential buildings; the illustrations in *Figs.* 256 and 257 show a successful method of shuttering for these walls. The sheeting for the external face (*Fig.* 256) and the internal face (*Fig.* 257) consists of horizontal boarding attached to closely-spaced vertical battens. The boarding is not close-jointed in this case as a lining is provided to produce a face free from board marks. The

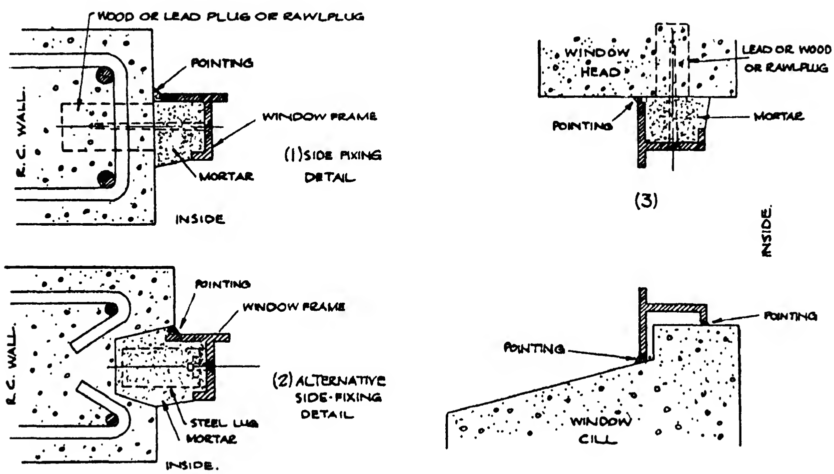


FIG. 255.—FIXING STEEL WINDOW FRAMES.

vertical members of the shuttering are wedged against horizontal walings. The shuttering is prevented from bulging by connecting the corresponding walings on opposite faces by bolts passing through the wall at frequent intervals. At the bottom a lateral bearing for the outside shuttering is obtained against the edge of the concrete forming the balcony slab, while timber spreaders would be provided at the top. At the bottom a spreader is needed to hold the foot of the inside shuttering against the pull of the bolts. The outside shuttering is carried on timbers supported on the external steel scaffolding; on the inside the completed balcony slab affords a general support for the shuttering. Lateral stability of the complete shuttering is provided by the raking struts bearing against the lower waling on the outer face and, on the inside, by the raking struts from the top waling to a small upstanding portion of the main wall cast integrally with the floor slab. This waling acts in combination with the wire guy on the inner face. The method of securing the feet of the rakers and of anchoring the end of the guy is clearly indicated in the illustrations, which also show an elevated

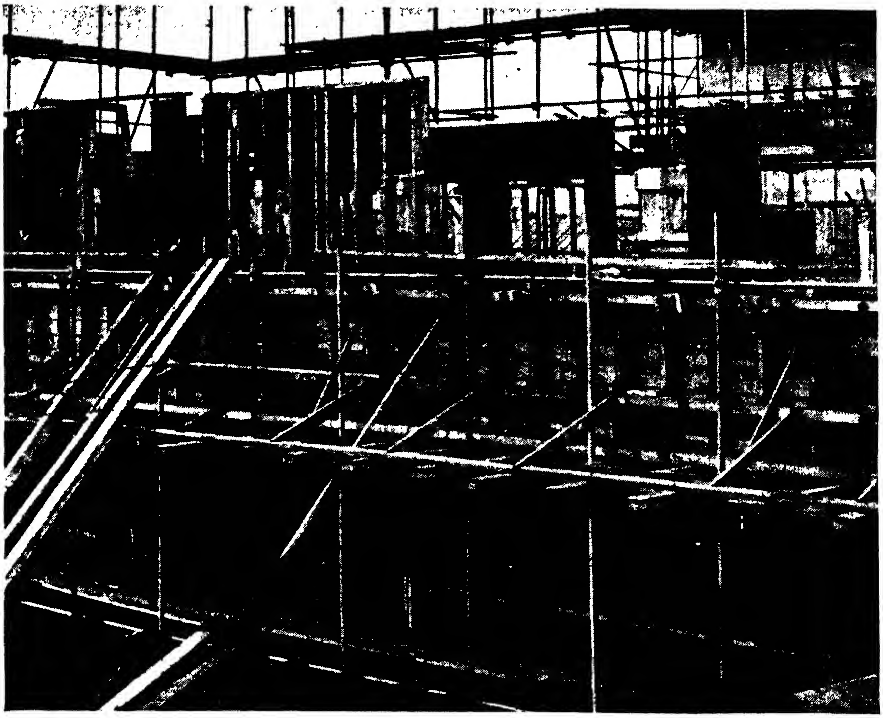


FIG. 256.—EXTERNAL FACE OF BALCONY WALL.

FIG. 257.—INTERNAL FACE OF BALCONY WALL.

working platform from which concrete is placed in the walls. This platform is carried on transverse timber joists supported at one end on the outer steel scaffolding and on the inner side on patented adjustable metal stands.

Concreting Columns and Walls.

The general principles of placing concrete laid down in Chapter VI should be applied to the construction of columns and walls. In the case of columns, when the reinforcement has been fixed the three sides of the column box to the full height of the column and the first lift of the fourth side are erected. The column box is cleaned out, the cleaning-out opening (if provided) is closed (see *Fig.* 240), and the reinforcement is inspected and packed out from the shuttering with cover blocks where necessary. The first lift of concrete is then deposited, and a further panel is placed in the open side of the box before placing the second lift. The second and subsequent panels should be ready for erection, and should not require to be cut or trimmed as this would cause sawdust or shavings to fall on to the concrete previously placed.

The operation of placing concrete in each lift should be as continuous as possible, but the rate of vertical progress should not exceed about 3 ft. in half an hour. The depth of concrete placed in a single batch is frequently specified not to exceed 3 ft. The rate of filling the box, within these limitations, is controlled by the strength of the shuttering. If each lift attains its initial set before the next lift is placed, the risk of the aggregate dropping towards the bottom of the shutters and leaving a layer of mortar at the top of the lift, which is characteristic of badly-concreted columns, is minimised, and the formation of cavities in the body of the finished work due to unequal settlement of the wet concrete is prevented. Settlement is accelerated by ramming and tamping or by the use of vibrators. In the case of deep, single deposits of concrete, hammering the outside of the shuttering assists to consolidate the concrete if it is combined with internal tamping. The ramming should be done with care, especially in the corners, and it is preferable for the men ramming to be stationed over the top of the column. Thus the rammers can be worked vertically up and down in the " cover " and in the column core.

The concrete should be deposited centrally in the column, and if, as in the case of tall columns, it is not practicable to drop the concrete vertically from the top of the box, it should be fed in at convenient intermediate heights through a short length of chute extending to the centre of the column. Concrete should not be discharged directly from long chutes into the column boxes else honeycombing due to separation may result.

Concreting a column should be stopped 3 in. to 6 in. below the level of the beams running into the column head, the portion of the column between the stopping-off level and the top of the slab being concreted with the beams. The latter operation should not be commenced within two hours of placing the concrete in the column, this interval allowing the concrete in the column shaft to settle. Usually this interval is several days, and these are occupied in erecting the shuttering for the beam and slab and fixing their reinforcement. The top of each lift of concrete or any other construction joints in vertical columns should be truly horizontal. If the column is inclined to the vertical, the joints should be at right angles to the column axis.

The consistency of the concrete in walls and columns is sometimes varied, a drier mix being used for the upper portions of the work to counteract the effect of water welling up from the lower lifts. This method is advantageous where placing is practically continuous, but where the initial set of each lift is allowed to take place before the next lift is deposited a concrete of uniform quality is obtained if the consistency is constant. The latter should be as dry as can practically be worked around the reinforcement. Ramming, tamping, or vibrating should be stopped immediately excess water appears on the surface of the concrete. If these operations are carried beyond this stage, weak layers will be formed in the column.

Walls should be concreted in vertical lifts of about 3 ft., similarly to columns, any wall columns or pilasters being concreted at the same time as the intermediate panels. The concrete should be deposited in layers, say, 12 in. deep uniformly along the length of the wall or section of wall under construction. Placing concrete at one point and working the concrete along the shuttering by rods or tampers should not be permitted.

All stoppages between successive lifts or for meal-times or day's work joints should be left with truly horizontal and vertical faces. Horizontal joints should be made along the line of external features where practicable, in order to conceal the joint. Care should be taken to obtain sharp unbroken edges true to line, and in important situations the position of the joints should be determined in consultation with the architect and engineer. Since the corners of windows and door openings are potential positions for cracks in wall panels, neither horizontal nor vertical construction joints should be made in line with the tops, bottoms, or sides of openings. Vertical joints at considerable changes of section or at reduced sections, as at openings, should also be avoided ; preferably they should be placed where the panel extends for its full height without interruption. A construction joint in a wall at the level of the tops of adjoining floor slabs, although better avoided, is generally provided because it leads to easier construction. If a wall extends above the slab, acting in conjunction with the slab and with or without a portion of wall below the slab, and has been designed to act as an " upstand " beam, as is commonly the case in residential buildings, it is imperative that a portion of the upper wall panel, together with any columns monolithic with the wall, should be cast simultaneously with the slab. The engineer should always be consulted on this point. The illustration in *Fig.* 257 shows an example where a joint at the level of the top of the slab has been avoided and a portion of the wall above has been concreted. In this case, as already stated, the small upstand was helpful in supporting the balcony wall shuttering as well as maintaining the vertical alignment of the main walls through successive stories.

The shuttering on the faces of a wall prevents the concrete drying out rapidly during the early period while it is still in position. Further curing is not usual in normal conditions, but where the face of the wall is exposed to the direct rays of a hot sun or to drying winds it is necessary to keep it damp in order to prevent shrinkage cracks. This is done either by spraying the face with water or, in extreme cases, by hanging over the exposed face saturated sacking, muslin, or other suitable material.

SECTION III.—FLOOR AND ROOF CONSTRUCTION.

Supports for Floor Shuttering.

The primary support for shuttering for beams and slabs may be timber or metal props or timber or steel scaffolding. Timber props may be either lengths of scaffold poles or sawn timbers not less than 4 in. square; smaller sizes can carry only very low loads or require very closely-spaced bracing.

The working load on a prop depends not only on its cross-sectional area but very considerably on its length. To sustain the maximum load derived from the compressive strength of the timber, the lateral unsupported height of the member must be limited by bracing. Values of the safe stress on timber props for various ratios of unbraced height to least width of prop are given in the table for designing shuttering in Chapter IV. In ordinary building construction common practice favours unbraced props, as the working load rarely approaches the full strength of the member. *Table* XXIII can be used as a guide to the spacing, or alternatively the size, of props required for normal floors.

TABLE XXIII.

AREAS OF FLOOR SHUTTERING SUPPORTED ON PROPS.
(Inclusive load = 150 lb. per square foot.)
Areas (sq. ft.).

Size of prop	Unbraced height of prop (ft.)				
	10	12	15	20	25
4 in. by 4 in. . .	66	58	46	26	—
5 in. by 2½ in. .	33	23	8	—	—
6 in. by 3 in. . .	60	48	30	—	—
7 in. by 2 in. . .	23	9	—	—	—
9 in. by 3 in. . .	90	72	45	—	—
6 in. by 6 in. . .	180	168	150	120	90

The tabulated areas represent the area which each unbraced prop can support, assuming that the sum of the superimposed load during construction and the weight of the shuttering does not exceed 75 lb. per square foot, and that the average load due to the concrete in the slab and beams is less than 75 lb. per square foot. If the total load varies from 150 lb. per square foot, the area supported can be taken as inversely proportional to the figures in the table. Thus in heavy floor construction, where the combined loading may be 200 lb. per square foot, the areas should be reduced to three-quarters of the tabulated values, while for light floors and roofs where an inclusive load of 100 lb. per square foot would be sufficient the tabulated areas should be multiplied by 1½. Where the heights exceed 25 ft. or the sizes of the prop are different from those tabulated, the case should be investigated by applying the data in Chapter IV.

EXAMPLE.—For supporting the shuttering for a heavy floor 6-in. by 3-in. timbers are available, the clear propping height being 15 ft. The spacing of the beams is 7 ft. 6 in. At what intervals should the props be placed if a single row is provided under each line of beams?

From the table, a 6-in. by 3-in. prop 15 ft. high will support 30 sq. ft. of normal floor, which can be considered as $\frac{3}{4} \times 30 = 22\frac{1}{2}$ sq. ft. of heavy floor. The spacing of the props will therefore be $\frac{22\frac{1}{2}}{7\frac{1}{2}} = 3$ ft.

When the load on a prop is such that an unbraced prop is inadequate, the bracing may be as shown in the typical arrangement in *Fig.* 258 (*a*). The diagonal bracing adds to the general rigidity of the group of props and stiffens the separate props. A line of props should preferably bear upon a continuous sill timber, and, to allow of adjustment in tightening up the prop and facilitate easy removal without jarring the superstructure or suddenly releasing the support, timber folding wedges should be provided at the foot of the prop as shown in *Fig.* 258 (*b*)

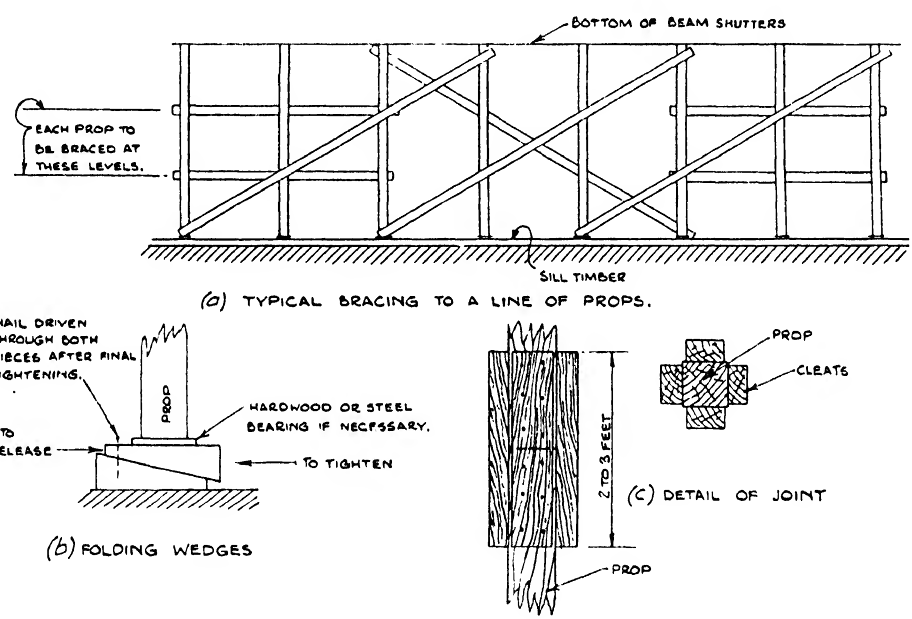


Fig. 258.—Props for Beams.

and *Fig.* 259. When the wedges have been finally tightened, a nail is driven through the two blocks to prevent relative movement. In normal work, with flat-angled wedges, security in this manner is not always essential, but if the slab or beam shuttering is to be subjected to external vibration a fixing nail is necessary. In place of wedges, sand boxes or jacks are used for exceptional structures such as the falsework for arches.

When the head or foot of the prop bears on soft timber and the average bearing stress, determined by dividing the load on the prop by its cross-sectional area, exceeds 400 lb. per square inch, a hardwood or metal bearing plate must be placed between the end of the prop and the soft timber to prevent the prop settling by compressing the softer material. The area of the bearing plate in square inches should not be less than the load on the prop in pounds divided by 400. Thus if a 4-in. square prop supports 60 sq. ft. of normal floor construction, the load on the prop may be $60 \times 150 = 9{,}000$ lb. and the end bearing stress is therefore $\frac{9{,}000}{4 \times 4} = 563$ lb. per square inch. A bearing plate with an area of not less than $\frac{9{,}000}{400} = 22\frac{1}{2}$ sq. in. is required.

In building work the props in any story should, where practicable, be directly above those in the story below, and the series of props should continue down to a firm support on the ground or to a floor or beams that have attained sufficient strength to support safely the superimposed load from the floor above. When

Fig. 259.—Timber Props for Floor Shuttering.

possible, each prop should be a single piece of sound timber free from knots or other defects. Where a sufficient number of long timbers is not available some props may be jointed, but the number of spliced props should not exceed half the number of props supporting slabs or one-third the number under beams. Only one joint should be allowed in each strut, and the ends of each piece should be square and butt closely. The joint should be covered by four timber cover-plates 2 ft. to 3 ft. long securely nailed to the prop and arranged as shown in *Fig.* 258 (*c*). If the prop is of circular section three cover-plates should be provided, unless the props are closely spaced and the load on each is small, where two cover-plates may be sufficient as in the case of the jointed scaffold-pole props shown in *Fig.* 259.

Lengths of timber scaffold pole are also used as props in the example shown in *Fig.* 260. In both this illustration and in *Fig.* 259 attention is drawn to the method of securing the crossheads carried on the top of the props and supporting the beam bottoms. The crosshead remains attached to the prop, and ready for re-erection on subsequent floors, when the shuttering is dismantled. An alternative detail at the heads of props is given in *Fig.* 261, together with the corre-

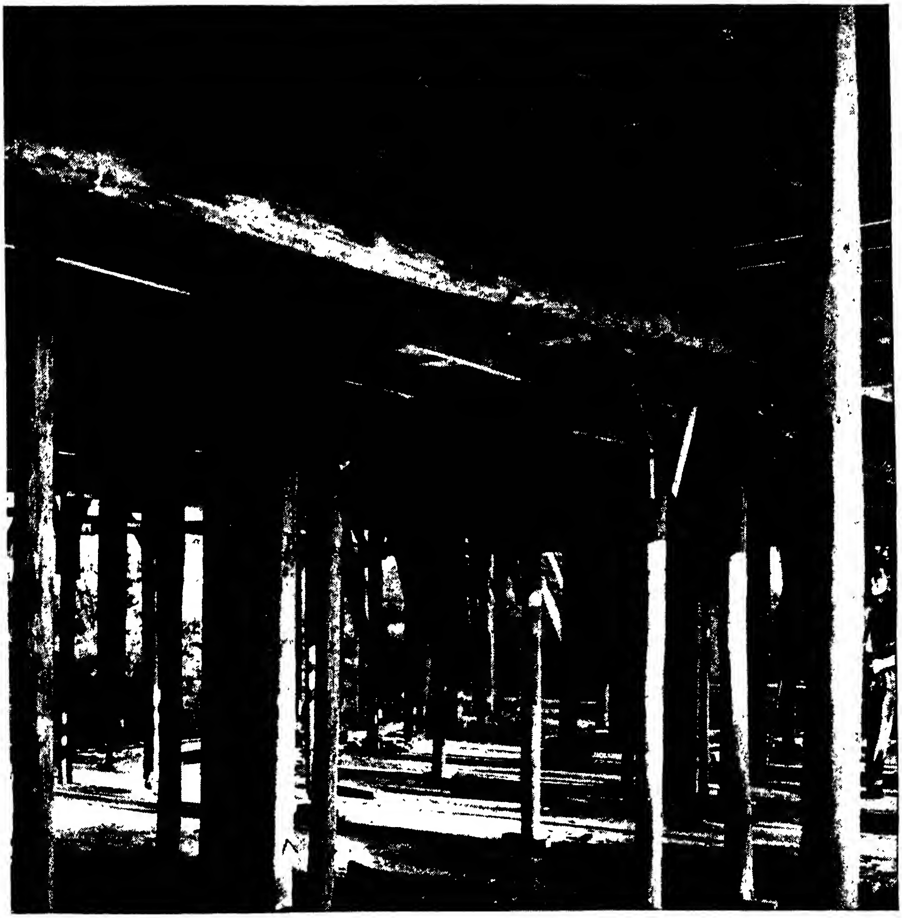


FIG. 260.—SCAFFOLD POLES AS PROPS.

sponding detail when props are used to support slab joists directly. The cleat shown in the latter detail prevents the joint between the joist and the head of the prop moving and is attached after the prop has been wedged in position.

There are a number of proprietary adjustable metal props on the market which are used extensively by contractors who have a continuity of contracts ensuring numerous re-uses. In such conditions these props are economical, but a contractor who only has occasional reinforced concrete flooring contracts would no doubt find timber props more advantageous, owing to the variety of odd sizes and rough timbers that can serve this purpose. A timber prop may therefore be of negligible cost in material, other than the cost of conveyance from one site to another. If new timber is used, a 6-in. by 3-in. prop, 12 ft. long, may cost 3*s*., and if used a dozen times before scrapping, the material would cost 3*d*. for each position. Assuming that a joiner and a labourer can cut, erect, and adjust six props in an hour, the net labour costs would be 5*d*. per prop. Including reasonable on-costs on these charges, a timber prop may cost 10*d*. each time it is erected.

Steel props in the form of built-up lattice stanchions were used successfully for the contract illustrated in *Fig.* 262, where considerable strength was required owing to the long span of the beams and the heavy floor construction. The

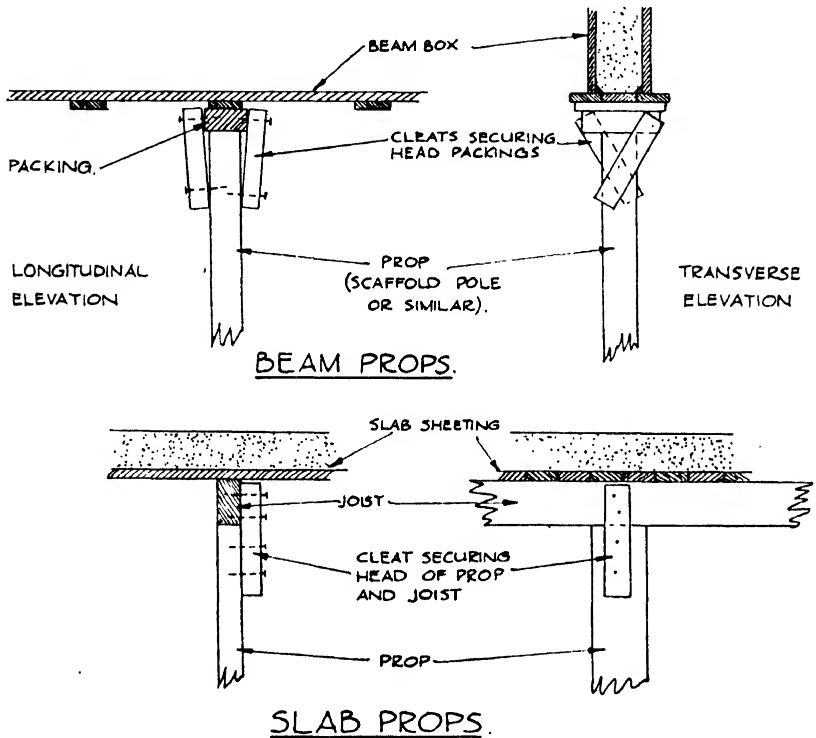


FIG. 261.—HEADS OF PROPS.

props were adjusted by screw-jacks placed under the base plates. Such an elaborate type of prop would not generally be economical if specially-fabricated steel columns are required, but if suitable stanchions are available on the site for future structural use or from the demolition of existing buildings their temporary use as beam props would be convenient.

Tubular steel scaffolding is also employed in place of timber props as the principal support for beam shuttering. An example is illustrated in *Fig.* 242, where the vertical tubes are provided with square metal sole-plates nailed to a 9-in. by 2-in. sill timber. This type of scaffolding can also be used to support the complete shuttering for light floors, its use being particularly favourable for high stories and where columns are widely spaced. *Fig.* 263 illustrates the

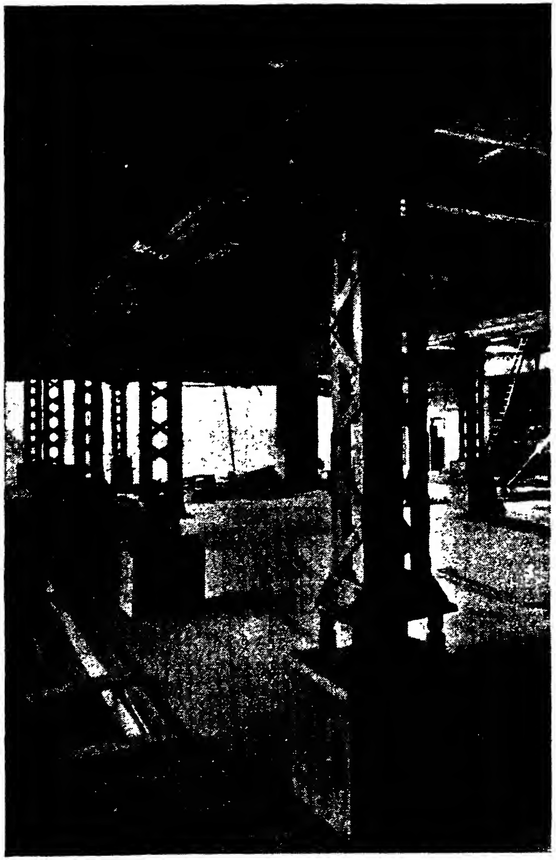


Fig 262.—Steel Columns as Props.

Fig. 263.—Steel Scaffolding Supporting Floor Shuttering.

application of steel scaffolding for this purpose. Since high-level floors and roofs make it necessary to provide a working platform immediately below the soffit for the men stripping the shuttering, the scaffolding serves the double purpose of a support for the platform and for the shuttering. If the floor under construction is heavy, due either to wide column spacing or to high superimposed loads, the shuttering may be supported on independent timber props and an internal steel scaffolding used to provide a high-level working platform. This apparent duplication is warranted economically by the fact that the vertical steel tubes forming the scaffolding require excessive horizontal bracing if high loads are to be carried. The tall timber props employed in this double construction have almost their original value even after a few re-uses, owing to the small mutilation they suffer.

The safe concentric working load in tons on 2-in. steel scaffold tubes (No. 6 gauge), allowing a factor of safety of about 4, can be taken as follows:

Unsupported height of tube (ft.).	Safe concentric load (tons).
4	3[illegible]
5	3
6	$2\frac{1}{4}$
8	$1\frac{1}{2}$
10	1
12	$\frac{5}{8}$
14	$\frac{1}{4}$
16	nil

The safe load on a putlog coupling is about 1 ton, on a double coupling $1\frac{1}{4}$ tons, and on a universal coupling $1\frac{1}{2}$ tons. A coupling is the weakest part of the construction, and since the grip attained varies considerably with the experience of the erector a maximum working load of $\frac{1}{2}$ ton should be taken in general cases. The safe working load on a 2-in. (No. 6 gauge) ledger acting as a beam supported at 6-ft. centres with a central point load is about $2\frac{1}{2}$ cwt., or 5 cwt. if the load is uniformly distributed along the 6-ft. span.

For beams with long span the number of props increases, and the practical difficulties and cost of a large number of separate supports may make it advisable to use a temporary supporting girder. In cases where no support for intermediate props is available, some form of girder is required beneath the beam soffit. A timber truss (*Fig.* 264) can be used. It is essential that the ends should be adequately supported by timber trestles, heavy column boxes, walls, or otherwise. The connections between the various members of the timber truss can be made with dog-spikes if vertical loading only is anticipated. If side-racking is experienced, bolted and plated connections similar to that shown in *Fig.* 264 (*b*) must be provided together with lateral bracing between adjacent girders. The boom timbers should be in a single length if possible; otherwise they should be joined over the head of one of the posts as in *Fig.* 264 (*c*). With the diagonals placed as indicated in the sketch, all members of the truss, except the bottom boom, are in compression. The beam shuttering is supported by packing and wedging off the top boom of the truss as illustrated at (*d*).

A steel truss could also be used, but the fabrication of special steel-plate girders or trusses in building work is seldom warranted. Timber trusses of the form previously described have the advantage that they are readily made with the labour and material available on a building contract. Available steel joists may be economically used without intermediate props over suitable spans. In any proposal to use a steel joist for supporting floor shuttering, the available

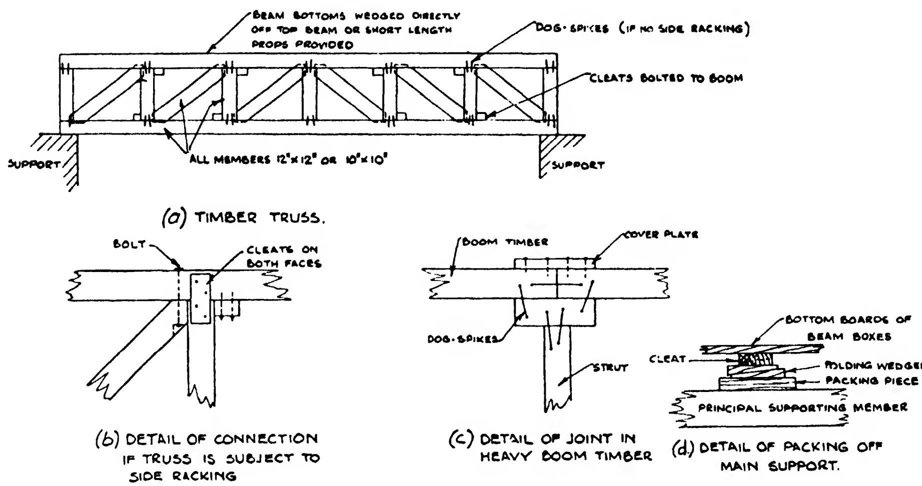


FIG. 264.—TIMBER TRUSS.

joist, before proceeding, should be checked for deflection by applying the data given in structural steel handbooks.

Alternatively *Table* XXIV can be referred to for the safe distributed load on steel beams of British Standard sections.

TABLE XXIV.

SAFE LOAD (IN TONS) ON STEEL BEAMS.

Section of steel beam	Free span of beam (ft.)										
	10	16	20	22	24	26	28	30	32	36	40
6 in. by 3 in. by 12 lb.	$3\frac{3}{4}$	$2\frac{1}{3}$									
8 in. by 4 in. by 18 lb.	$7\frac{1}{2}$	$4\frac{1}{2}$	$3\frac{3}{4}$	$3\frac{5}{8}$							
9 in. by 7 in. by 50 lb.	$24\frac{3}{4}$	$15\frac{1}{2}$	12	11	10						
10 in. by $4\frac{1}{2}$ in. by 25 lb.	13	8	$6\frac{1}{2}$	6	$5\frac{1}{2}$	5					
12 in. by 5 in. by 30 lb.	18	$11\frac{1}{2}$	9	8	$7\frac{1}{2}$	7	$6\frac{1}{2}$				
12 in. by 8 in. by 65 lb.	43	27	21	19	18	16	15				
14 in. by $5\frac{1}{2}$ in. by 40 lb.	28	18	14	13	12	11	10	$9\frac{1}{2}$			
15 in. by 6 in. by 45 lb.	35	22	17	16	14	13	$12\frac{1}{2}$	12	11		
16 in. by 6 in. by 50 lb.	41	25	20	18	17	15	$14\frac{1}{2}$	$13\frac{1}{2}$	13	11	
18 in. by 6 in. by 55 lb.	50	31	25	22	20	19	17	16	15	14	12
20 in. by $6\frac{1}{2}$ in. by 65 lb.	57	40	32	29	27	25	23	21	20	18	27
22 in. by 7 in. by 75 lb.	70	50	40	37	33	31	29	27	25	22	20
24 in. by $7\frac{1}{2}$ in. by 90 lb.	—	67	54	49	45	41	38	36	33	30	16

Where no load is specified, deflection is excessive.

EXAMPLE.—What size steel beam is required to support the shuttering for a normal floor if beams are placed at 10-ft. centres and are supported on column boxes placed 24 ft. apart? If we take the inclusive load of shuttering, concrete, etc., as 150 lb. per square foot the total load on each steel beam is

150 lb. × 10 ft. × 24 ft. = 36,000 lb. = 16 tons. Assuming the load is transferred to the beam through the reaction of several close-spaced joists, the load is practically uniformly distributed, therefore, from the table, a 16-in. by 6-in. by 50-lb. steel beam is required to carry a load of 16 tons over a span of 24 ft.

Timber and steel used for supporting falsework have high salvage values, while timber has the added advantage of allowing blocks and other attachments to be nailed on in any position.

For moderate spans, a built-up truss or a line of props may be replaced by a single heavy timber, say, 10 in. or 12 in. square, or by pairs of 10-in. by 4-in. timbers placed on edge and bolted together, depending upon the material the contractor has on hand or can obtain most cheaply, and on the span to be covered and the load to be carried. A few rough calculations of strength and quantities will indicate the cheapest arrangement.

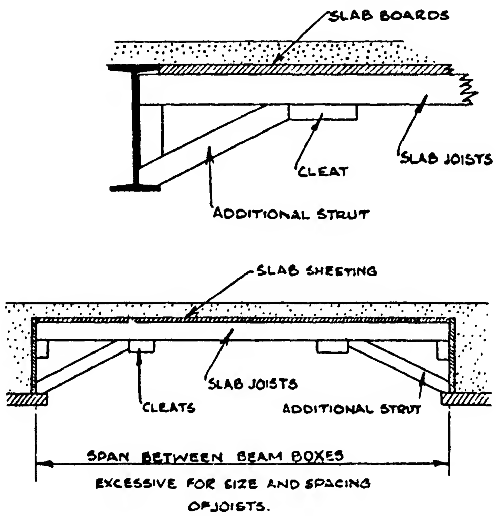


FIG. 265.—REDUCING EFFECTIVE SPAN OF SLAB JOISTS.

The shuttering for slabs up to 7 ft. span is usually supported on the beam boxes, but for larger spans one or more lines of intermediate props from the floor below may be necessary unless the slab joists are sufficiently heavy to span between the beams. The intermed'ate props can be avoided without increasing the joist section if the construction shown in *Fig.* 265 is used. The object is to decrease the effective span of the joists by providing struts whose lower ends bear on cleats fixed to the sides of the beam shuttering or column boxes. In the case of concrete slabs supported on structural steelwork, the bearing can be taken on the bottom flanges of the steel joists.

The shuttering for floors, balconies, or other work projecting from the main building at some distance above the ground can be supported as indicated in *Fig.* 266. This type of construction occurs frequently in modern buildings and industrial structures, and the elevation of the work usually makes direct vertical propping impracticable. The horizontal member immediately below the soffit of the work must be well tied back to the main shuttering and deflection of the overhanging portion must be limited by one or more inclined struts as shown.

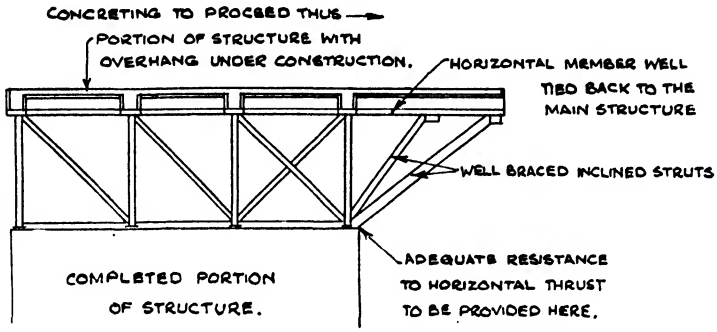


FIG. 266.—OVERHUNG WORK.

Details of the attachment of the feet of these struts and the connections of the top horizontal member depend on the design of the main structure.

Slab and Beam Shuttering.

Typical arrangements of shuttering for floor and slab construction are shown in *Figs.* 267 to 278. Slab sheeting is formed by boards at least 1 in. thick laid and carried on joists not less than 4 in. by 2 in. in section. The ends of the joists are supported on cleats or continuous bearers attached to the sides of the beam-boxes. The spacing of the joists will depend on the load to be carried by the sheeting (see Chapter IV). For ordinary work, the spacing of the joists, when the boarding is continuous over more than two spans, can be obtained from *Table* XXV.

TABLE XXV.

SPACING OF JOISTS SUPPORTING SLAB SHEETING.

Thickness of boarding	Thickness of solid concrete slab		
	4 in.	6 in.	8 in.
	ft. in.	ft. in.	ft. in.
$\frac{7}{8}$ in. (1 in. wrought board)	2 6	2 3	2 0
$1\frac{3}{8}$ in. ($1\frac{1}{2}$ in. ,, ,,)	3 6	3 3	3 0
$1\frac{7}{8}$ in. (2 in. ,, ,,)	4 6	4 0	3 9

These spacings limit the deflection to a maximum of $\frac{1}{16}$ in. without over-stressing the boards in bending. If the boards span over a single span only or over two continuous spans only, the tabulated values should be reduced by one-eighth.

If the boards forming the slab sheeting are not warped it is possible to lay them on the joists without nailing, but if nailing is necessary to retain the boards in position as few nails as possible should be used so that the shuttering may be struck easily. To avoid cutting many boards, and at the same time to keep the span of the joists to a minimum and enable them to be as widely spaced as deflection permits, the boards should be laid the long way of the panel formed by the surrounding beams or other supports. To make it easy to start stripping the boarding, one or more of the boards should be severed, as at *A* and *B*, *Fig*. 267 (*b*). When the joists have been removed from below the sheeting, the short pieces of board *A* and *B* can be prised out of position in turn. The remaining boards are then readily removed.

When the floor construction is identical on a number of floors, it is best to

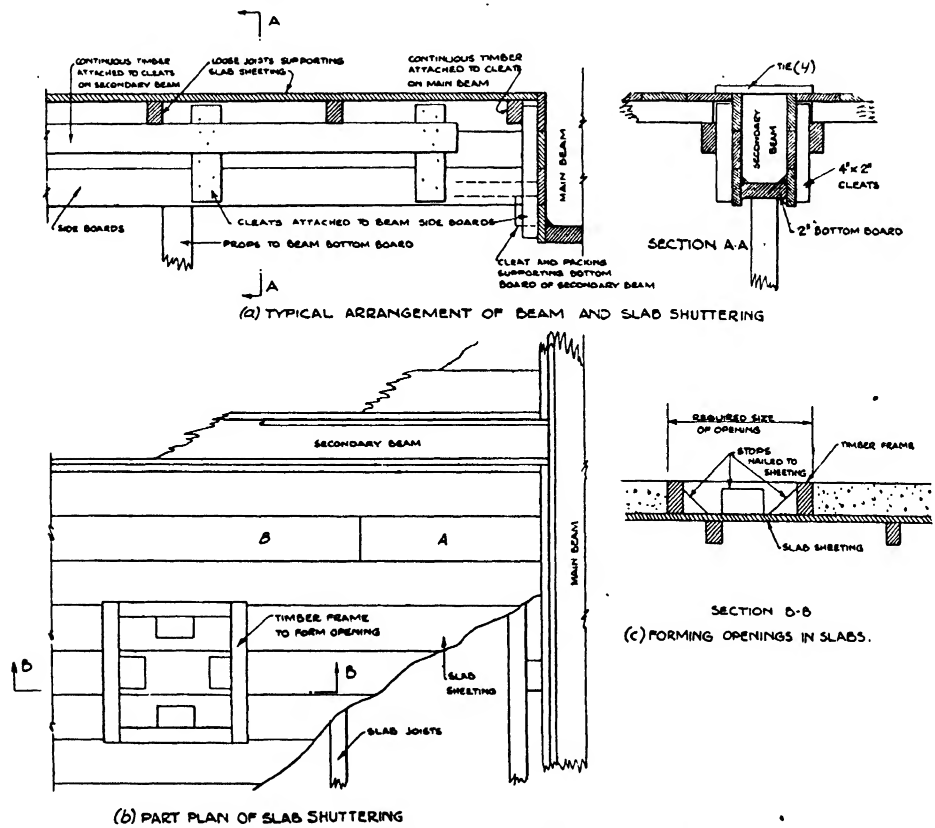


FIG. 267.—SHUTTERING FOR SLABS AND BEAMS.

build the slab shuttering in units that can be erected and dismantled in one piece. A method of doing this is illustrated in *Fig*. 268, where each unit consists of an inverted tray, the four sides of which are formed by boards standing on edge. Three intermediate transverse boards on edge help to support the sheeting parallel to the longer side of the panel. The whole of the members are securely nailed together. When the unit is erected the space along the longitudinal edges of the panel is closed by loose boards extending over the tops of the beam sides as illustrated. These boards enable the panels to fit loosely between the beam sides, thus ensuring easy removal. The units are supported on cleats or continuous runners attached at a convenient level on the beam sides, these supports being removed before taking down the panels. To facilitate striking the slab panels, a 2-in. space is left between the ends of adjacent units. After erection this space is filled with a 2-in. by 1-in. strip supported on cleats attached to the vertical boards along the shorter edge of the panel.

The shuttering for flat slabs (beamless floors) follows the principles of normal floor shuttering, props and additional joists taking the place of the support afforded normally to the slab sheeting by the beam boxes. The method of cutting the boards to form the column head and the drop around the latter is illustrated in *Fig.* 269. An additional fillet strip is provided to form a splay along the edges of the drop.

The method adopted for forming openings in slabs depends on the size of the opening. If they are between 1 ft. and 4 ft. square they can be made without interrupting the run of the sheeting by providing boards on edge

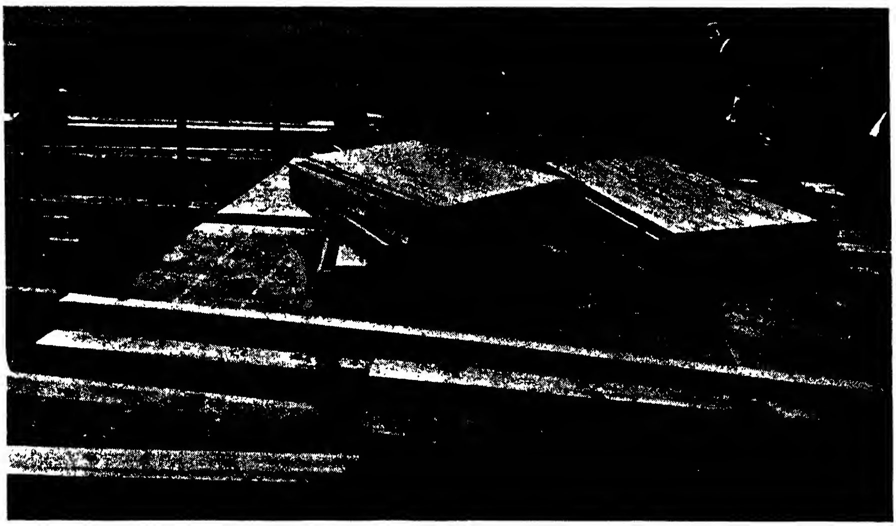


FIG. 268.—PANEL UNITS FOR SLAB SHUTTERING.

attached to stops nailed to the sheeting as in *Fig.* 267 (*b*) and (*c*). For larger openings, which would usually be trimmed with beams, the sheeting would be discontinued and one side of the box for the trimmer beam would be extended to form the shuttering for the edge of the opening. For small tapered holes such as are left for bolts or other fixtures, solid pyramidical plugs lightly attached to the sheeting are provided. When the sheeting is removed the plugs remain in position and can be easily tapped out from above or below, depending on whether the smaller end of the hole is uppermost or reversed. This method is practicable for holes up to about 4 in. square, but if a solid plug is used for larger holes difficulty is often experienced in removing the plug. A construction sometimes adopted for holes with either tapered or uniform section, is to split the plug vertically and to insert a slightly tapered piece of board between the two portions. By arranging this board to project it can be extracted without much difficulty and the half-plugs can be eased inwards and removed. An alternative is' to make a hollow box that projects well above the level of the concrete. The sides and their fixings are so arranged that when removal is necessary the nails can be extracted; gentle tapping on two opposite sides then

FIG. 269.—CENTERING FOR FLAT-SLAB COLUMN HEAD.

causes the box to collapse. Grooves in the top faces of slabs can be formed by fixing a strip of timber, mounted on two or more blocks, flush with the top level of the slab. If the grooves are curved in plan the timber can be bent to the required radius by making a few saw-cuts in one side as shown in *Fig.* 270.

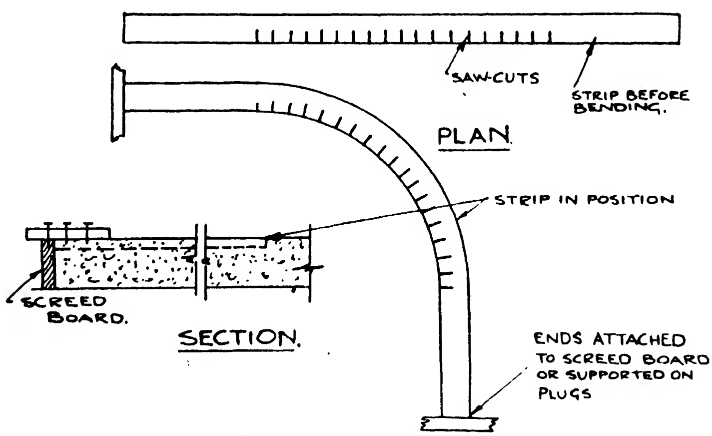


FIG. 270.—FORMING CURVED GROOVES IN SLABS.

A beam mould consists primarily of two parts—the bottom board and the sides. Methods of supporting the bottom board by propping or otherwise have already been discussed, and a common construction for the sides and bottom is indicated in *Fig.* 267 (*a*). This method is suitable for narrow beams of widths corresponding to commercial timber sizes, but for wider beams or for beams of such width that it would be necessary to cut a board to obtain the exact width, one of the alternative arrangements given in *Fig.* 271 is preferable. In adopting either of these methods it is necessary to cut the side boards to make up the exact dimension of the beam depth unless the latter happens to be a multiple of the board width. With the arrangement in *Fig.* 267 (*a*), however, a number of whole boards can be used, and this method is therefore more adaptable for a beam of constant width but varying in depth, or for successive uses on beams of various depths but of the same width. This method also offers some advantage when striking the shuttering, as the sides can be readily removed without disturbing the bottom boards and props. In all the methods illustrated the separate

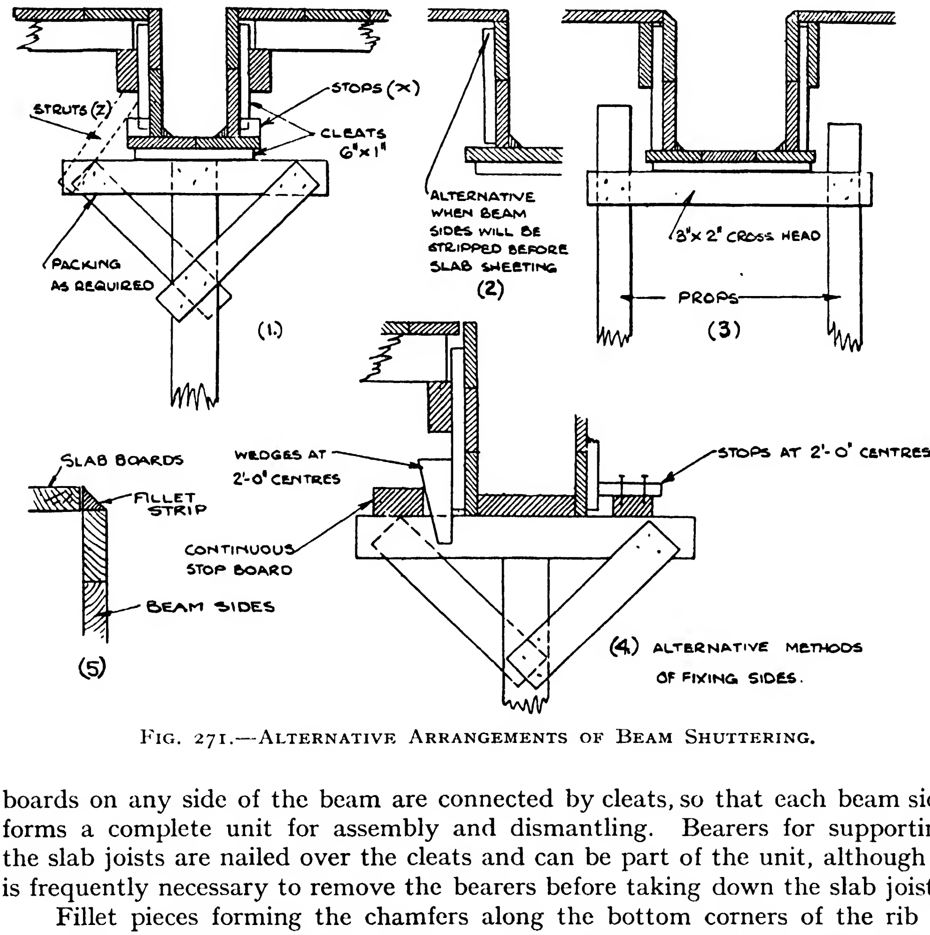


FIG. 271.—ALTERNATIVE ARRANGEMENTS OF BEAM SHUTTERING.

boards on any side of the beam are connected by cleats, so that each beam side forms a complete unit for assembly and dismantling. Bearers for supporting the slab joists are nailed over the cleats and can be part of the unit, although it is frequently necessary to remove the bearers before taking down the slab joists.

Fillet pieces forming the chamfers along the bottom corners of the rib of the beam are attached to the bottom boards. Splays between the slab and the beam stem can be provided by suitably cutting the top edge of the side shutters as at (3), *Fig.* 271, or more conveniently by attaching a fillet strip along the top edge of the side boards as at (5), *Fig.* 271.

The slab sheeting should only be carried over the top of the side boards, as in *Fig.* 271 (2), if the beam sides are to be stripped before the slab-boards.

The beam sides are prevented from spreading at the bottom by nailing them directly to the bottom board if the method in *Fig.* 267 is employed or by the stops marked x if the method (1) in *Fig.* 271 is followed. Other methods are given at (4), where removable stops and wedges are indicated. For easy removal the slab-joists should not bear tightly against the beam-sides, and so these joists cannot offer any lateral support to the tops of the beam-boxes. The slab sheeting may, however, provide support of this kind by preventing outward movement at the top of the box. A spreader between the side boards prevents inward movement, but each spreader must be removed as the concrete approaches it; the pressure of the wet concrete then prevents the sides from inclining inwards. A short timber tie *y* (*Fig.* 267) prevents both outward and inward move-

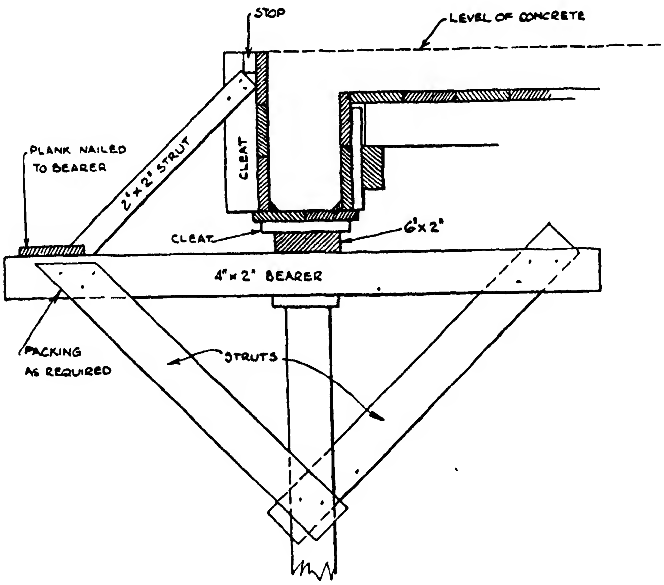
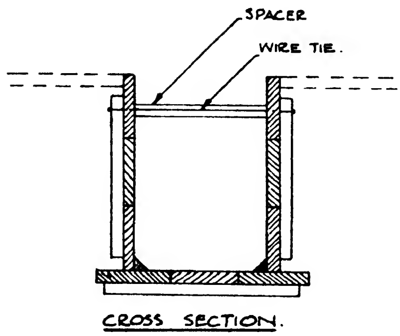


FIG. 272.—SHUTTERING FOR EXTERNAL BEAM.

ment of the sides of the beam box; these ties can be left in while the beam-stem is being concreted but must be removed before casting the slab. An alternative is to provide inclined struts extending from the crosshead to the sides of the cleats as at z (*Fig.* 271). A similar method is employed for beams at the edges of floors, a common form of construction being that shown in *Fig.* 272.

A further method of securing the sides of the boxes for light beams is by a wire tie and timber stretcher arranged as in *Fig.* 273. This construction can be used for tee-beams or for beams of rectangular section without a slab, although a more common arrangement for rectangular beams is that shown in *Fig.* 274. Fillet strips attached to the bottom boards prevent inward movement of the side shutters, while stops nailed to the bottom boards on the outside after the sides have been erected prevent outward movement. A timber tie nailed across the side shutters retains the tops of the latter in the correct position.



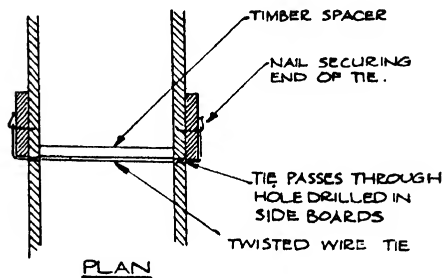


FIG. 273.—BEAM SHUTTERING SECURED BY WIRE TIES.

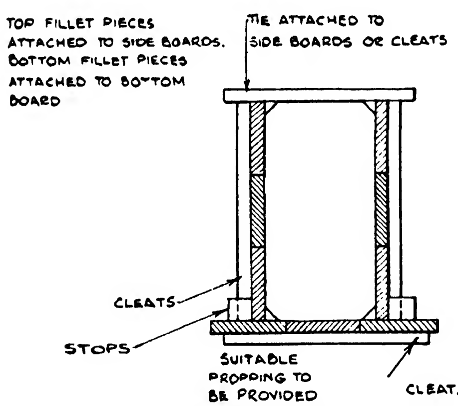


FIG. 274.—RECTANGULAR BEAM SHUTTERING.

A constructional detail for external or fascia beams involving simple architectural features is illustrated in *Fig.* 275. Features of interest in this example include the small amount of cutting of the longitudinal boards forming the face of the moulding, the method of supporting the shutter board for the upper kerb, and the wire ties passing through the beam to secure the shuttering on opposite faces. The provision of diagonal struts and the method of propping and wedging off the bottom board are clearly shown.

Haunched beams present complications in shuttering, since, if the beam is haunched at both supports, the bottom board has to be in three sections, and it

FIG. 275.—CONSTRUCTION OF FASCIA BEAM.

may be necessary to mitre the ends of each section to ensure a tight joint. The side boards (*Fig.* 276) are generally in odd lengths to avoid cutting. An alter-

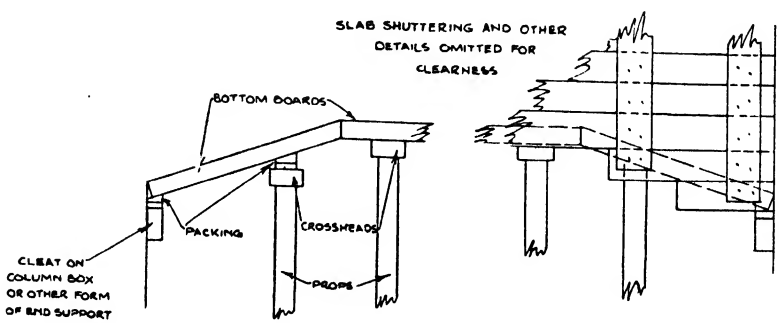


FIG. 276.—SHUTTERING FOR HAUNCHED BEAMS.

native method applicable to heavy beams is illustrated in *Fig.* 242, where, to prevent cutting to length and shape, the bearers below the haunches are in short lengths and pass inside the longer horizontal bearers under the central portion. In *Fig.* 277, the bottom boards for a series of haunched beams are shown propped

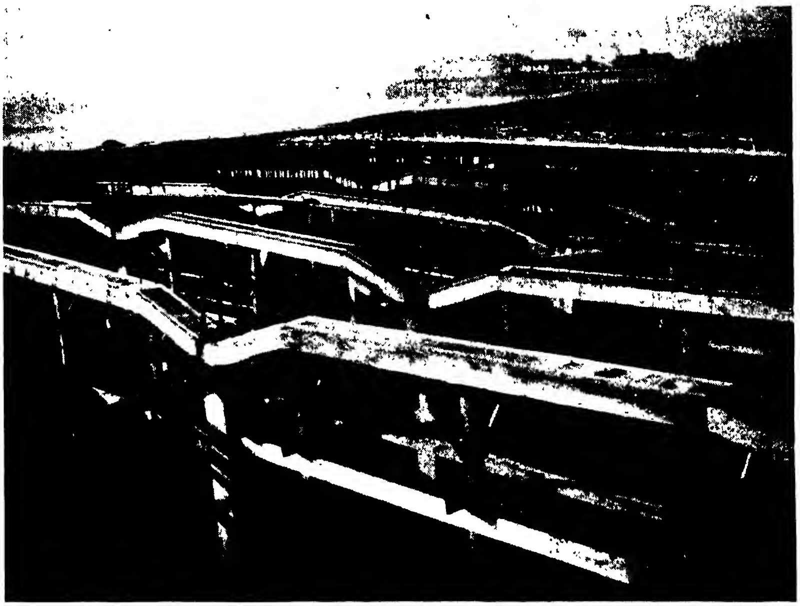


Fig. 277.—Haunched Beams.

in position ready for the reinforcement to be assembled before erecting the side boards.

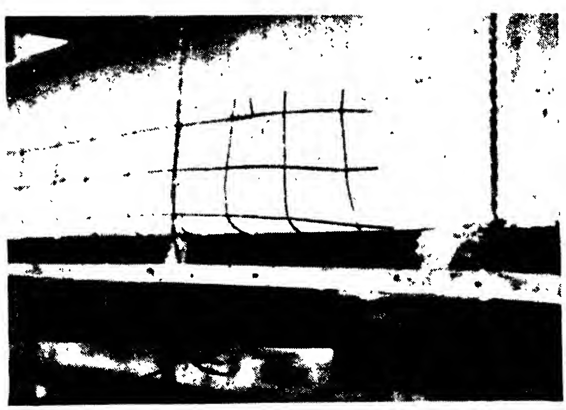


Fig. 278.—Shuttering for Steel Beams.

When steel beams are to be encased in concrete, the casing being monolithic with the floor slab, it is usual for the beam soffit board to be suspended from the beam by a twin wire tie, No. 9 gauge, as shown in *Fig.* 278. The beam sides, on which the slab joists are supported, are erected on the soffit board. Thus beam props are eliminated.

The order of erecting slab and beam shuttering to allow striking to proceed in a convenient sequence is:

(1) Attach cleats to column boxes or other end supports to provide a bearing for the beam bottom board;

(2) Lay beam bottom board with fillet strips attached and erect intermediate props (adjusted to give camber if necessary);

(3) Erect side boards as complete units and secure them top and bottom; attach slab joist bearer if not already on side boards;

(4) Lay slab joists in position with additional props, if required;

(5) Lay slab sheeting on joists; and

(6) True-up props.

This sequence enables the slab shuttering and beam sides to be stripped without disturbing the beam bottom board and its props. In accordance with some specifications it is often necessary to replace one or more props under each beam after the bottom board has been removed.

Concreting Slabs and Beams.

When placing concrete in floors and roofs attention should be given to the points mentioned in Chapter VI. The beam shuttering should be cleaned out, the reinforcement inspected, and the slab and beam shuttering saturated with water. Where practicable the ribs of small tee-beams should be concreted at the same time as the slab forming the flange. For large tee-beams it is better to allow an interval of not less than an hour so that the concrete in the rib may be allowed to attain initial settlement before the concrete slab is placed. When the rib is concreted separately, the concrete in the rib should be stopped 1 in. or 2 in. below the soffit of the slab and the remainder concreted with the slab.

Concreting the rib and slab simultaneously ensures monolithic construction but this practice is open to the objection that the slab reinforcement is liable to be displaced. This can be avoided by raising the barrow runs and gangways a few inches above the level of the finished floor. When the slab and rib are concreted separately, the top surface of the rib should be roughened and well wetted to ensure a good joint between the two units, special care being taken to remove laitance.

Slabs should be concreted in one operation to the full depth, except in two-course work. The correct top level and thickness of the slab are obtained by a screeding board worked off side-boards of the correct height, or off adjacent completed slabs. This operation is shown in *Fig.* 279. To obtain level floors, or more especially to guard against depressions in the middle of panels intended to be level, a camber of ¼ in. to ½ in. can be formed in the screed board. A convenient and simple form of intermediate level guide for use where large areas of slab are concreted in one operation is shown in *Fig.* 280. Where the slab is suspended, and therefore requires shuttering on the underside, timber blocks attached to the sheeting can be used to obtain the correct thickness. The height of each block should correspond to the slab thickness at the point at which the block is placed. Before concreting proceeds beyond the position of any block, the latter should be removed.

FIG. 279.—SCREEDING SLABS.

Ribs standing above slab level and forming beams should be concreted with the slab or not more than two hours later. Where this is not possible,

FIG. 280.—LEVEL GUIDE FOR SLAB.

and a definite break in the concreting must be made between placing the concrete in the slab and the rib, a height of 2 in. or 3 in. of the rib should be placed simultaneously with the slab to prevent a construction joint occurring immediately at the change of section at the top of the slab.

Day's work joints in floors should be made in accordance with the recommendations in Chapter VI ; in general, they should occur in the middle of the span of beams and be parallel to the main reinforcement in the slabs.

Owing to the large area of concrete exposed floor slabs require careful curing, and the top surface should be maintained damp for a few days. The most convenient method is to lay canvas on the surface as soon as the latter has hardened sufficiently to prevent the concrete from being marked by the covering. The latter should be kept damp by watering at intervals, as required, during dry weather. Canvas for this purpose can be obtained in rolls of convenient size which are easily spread and rolled back and can be used many times. Sawdust, which is often available in sufficient quantities on a large contract where power-driven saws are installed, is also suitable for curing purposes, as it will retain moisture for considerable periods. The bulk of the sawdust can be swept and shovelled away at the end of the curing period, and the remainder can be removed with a hose.

If a special covering is not used, frequent watering will produce the same result. Slabs should be watered once or twice daily, including Sundays and holidays, the number of times depending on the rate of drying. Whatever method is used it is necessary to prevent the excess water running away and damaging other parts of the work.

Hollow-tile Slabs.

The popularity of hollow clay tile and other forms of ribbed floors and roof slabs, due to their economy and light weight, has led to the development of various proprietary systems, but the designs of the commonest type of hollow clay tile slabs are not covered by any patent, although in London and for most public bodies, such as H.M. Office of Works, this class of work is controlled by regulations. There are a number of practical points deserving consideration in their construction. A general view of the construction of a ribbed floor is illustrated in *Fig.* 281, while details of hollow tile slabs are given on *Fig.* 282.

The tiles should be cleanly made, true to shape, out of winding, free from cracks and lime, of uniform density, sound, and burnt well and evenly. When specified, one or more of the outer faces should be roughened or scored to form a key for ceiling plaster or other finish. The size of tile should be one that is readily obtainable. All tiles should be well soaked in water immediately before concreting commences as their absorption is surprisingly great.

It is not usually necessary to place mortar in the joints between the tiles, although this is essential on contracts in London where the ends of the blocks must be solidly jointed with a 1 : 2 cement-sand mortar. When blocks with mortared joints are laid in position it is necessary to prevent workmen and others from walking over them until the mortar has set, otherwise the joint may be broken. In first-class work, in addition to mortaring the ends of the blocks when they are laid, the top and side joints are pointed with a thick cement paste or a rich mortar (say, 1 part of cement to 1 part of sand), immediately before placing the concrete. At the first and last tile in each row a stop-end must be provided. This can be done either by purchasing the required number of special tiles with a stop-end inserted or by using a piece of sheet metal to cover the cavity at one end of the tile. Alternatively, a concrete stop can be inserted by standing the tile on edge and dropping in a wad of concrete, which should be about 1 in. to 2 in. thick and well tamped against the side of the tile as shown in *Fig.* 283. The tile should not be moved until the concrete wad has hardened. About twelve tiles per man-hour can be stopped by this method, including taking them from the stock pile, upending, mixing, placing and tamping the concrete, and restacking to await use.

With the object of preventing pattern staining of ceilings, a tile slip about ½ in. thick is inserted in the bottom of the rib, while in other cases the whole

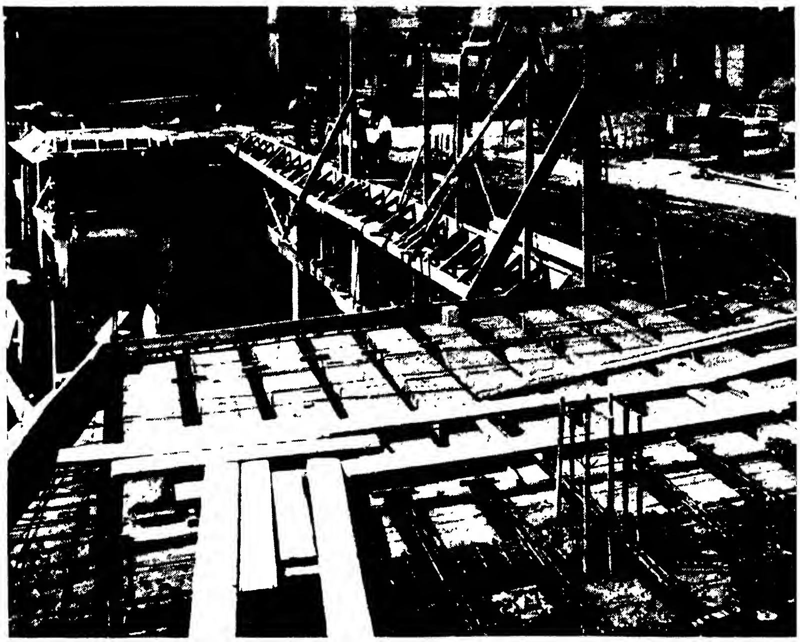


FIG. 281.—RIBBED FLOOR CONSTRUCTION.

soffit of the slab is covered by ceiling boarding. These alternatives are shown in *Fig.* 282 with acceptable relative sizes of concrete and tile, and these proportions are recommended as good practice when no regulations govern the design and construction of the work. In London, however, it is necessary to provide a cover of not less than ½ in. of concrete where a clay tile slip is inserted and at least ¾ in. when no slip is provided.

The top slab is not usually less than 1½ in. thick, but this thickness only occurs in the lightest floors or roof-slabs. A convenient thickness for the rib is about one-fifth of the clear distance between adjacent ribs but not less than 2 in., otherwise difficulty will be experienced in getting the concrete into the rib and around the reinforcement. The London regulations for fire-resisting construction require the thickness of material above the void to be not less than

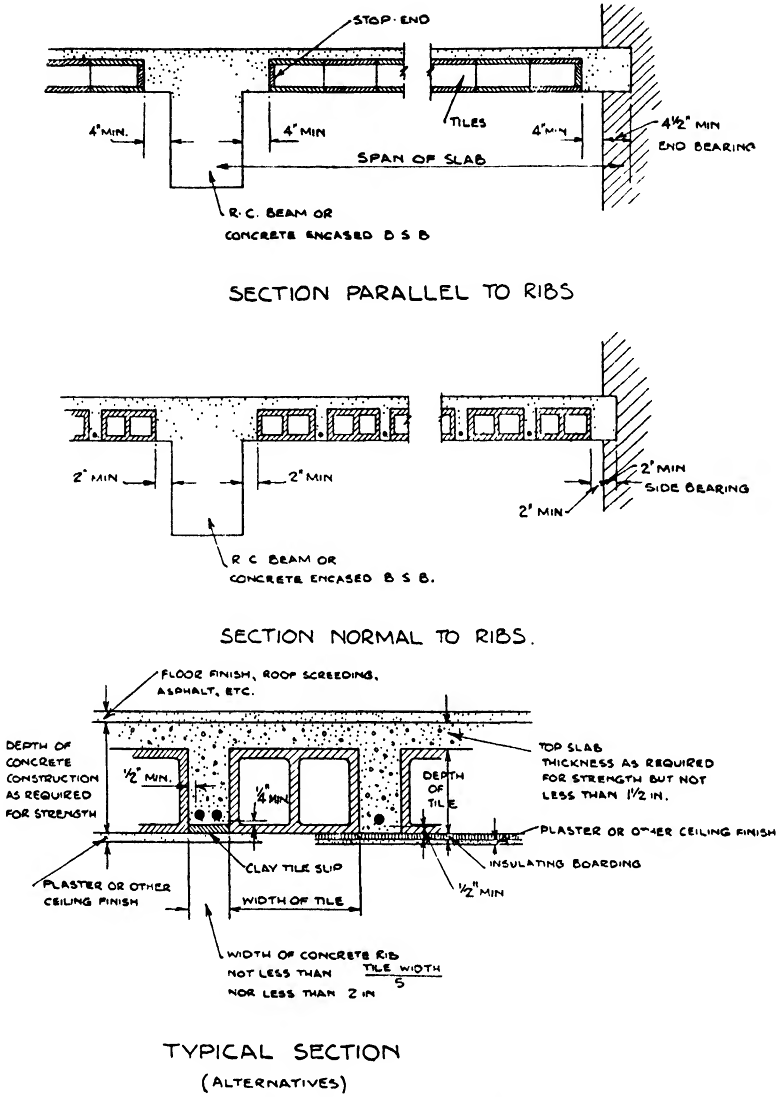


Fig. 282.—Details of Hollow Clay Tile Slabs.

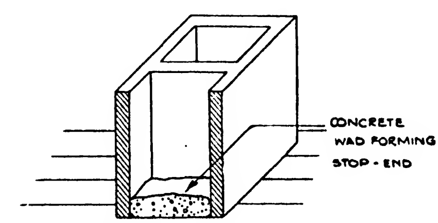


Fig. 283.—Stop-end in Hollow Clay Tile.

2 in. when the width of the void is not more than 4 in., and not less than 3 in. when the width is not more than 6 in. Further, the thickness of the soffit of the cavity must not be less than $\frac{3}{4}$ in. and the total thickness of the material below and above the cavity should not be less than 3 in These thicknesses are irrespective of plaster or other finishes. The limitations are important to a contractor ordering tiles, since he must determine before placing an order that the thicknesses of the webs and walls of the tiles, when used in combination with the rib and slab specified by the engineer, comply with the regulations.

Between the faces of supporting beams or walls and the first and last tile in a row, likewise between the outer rows of tiles and the side walls or beams, there should be a portion of solid slab. The minimum extent of this solid portion should be about 4 in. at the ends and 2 in. at the sides as indicated in *Fig.* 282, but these dimensions may be exceeded to comply with the shear and beam-flange requirements or the aggregate size of a row of tiles.

Owing to the narrowness of the ribs and the thinness of the top slab, a concrete mix not leaner than 1 : $1\frac{1}{2}$: 3 with coarse aggregate of $\frac{3}{8}$-in. maximum gauge is recommended. In London the regulations require the concrete mix to be 1 part of Portland cement to 3 parts of a fully graded aggregate $\frac{3}{8}$ in. maximum gauge. If the aggregate is not fully graded the mix must be increased in richness to 1 part of cement to 2 parts of aggregate. The ribs and the top slab must be concreted simultaneously.

Roof Construction.

Flat Roofs.—The construction of flat beam-and-slab roofs is similar to that of floors, but special problems arise in the case of high-level roofs, frames for pitched roofs, vaults, and domes.

The top surface of a flat roof-slab is generally laid to falls to facilitate drainage, the amount of fall varying from 1 in. to 3 in. in 10 ft. If the area of the roof-slab is small and the surface falls in one direction only, it is usually most convenient to retain a constant thickness of slab with a soffit parallel to the sloping surface. If the falls lie in more than one direction this method may lead to complicated sheeting for the underside of the slab, and a level soffit with a varying slab thickness may be more economical even though the quantity of concrete is greater. It is better to concrete the roof slab in a single operation ; by cutting the screed guide boards to the fall of the top surface, the latter can be readily screeded to the required slope. If falls have to be added after the slab has been concreted, they should be formed in a 1 to 3 mortar or fine concrete, depending on the added thickness. The surface of the old concrete should be hacked, well-cleaned, and saturated before laying the material forming the falls. Feather edges should be avoided.

Vertical props must usually be dispensed with in the construction of high-level roofs where the height between the floor and the underside of the roof is considerable and when intermediate structural columns are omitted as in power houses, swimming baths, cinemas, and other halls. When the roof is of heavy design, an internal scaffolding is erected to support a working platform placed directly below the roof, and the shuttering for the roof slab and beams is strutted from this scaffolding. For light roofs, where the distances between opposite walls are small, the cost of scaffolding would be high compared with the amount of work to be done; the method illustrated in *Fig.* 284 conveniently provides two intermediate points of support for the roof shuttering and decreases the span of the principal horizontal supporting member. The inclined struts are supported on cleats attached to completed walls or columns. A tie should be inserted between the walls or columns to prevent them being subjected to a thrust for which they may not have been designed.

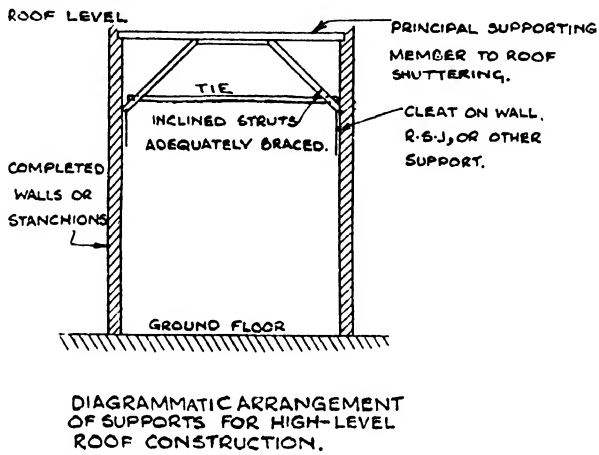


FIG. 284.—SUPPORTING HIGH-LEVEL ROOF SHUTTERING.

Framed Roofs.—Roofs without intermediate supporting columns are constructed with the main frames only in concrete or with a reinforced concrete vault slab in addition to the main frames, in which case the slab may be placed either above or at the soffit of the frames. Examples of the construction of both types are shown in *Figs.* 285 and 286. The details of the construction depend upon the type that is being erected.

In all cases the temporary centering consists of three principal parts: (1) the main scaffolding; (2) templates or profiles; (3) bottom and side boxing or sheeting for the frames and for the slab, if any. Timber or steel scaffolding is erected to form the working platform and support the shuttering. Details of the shuttering and profiles of a roof frame are given in *Fig.* 287, in which the template boards, cut to the required profile of the rib, are well braced and strutted from the scaffolding. All connections in the templates and their supports should be bolted where practicable, and the curved templates must be made with a suitable radius to allow space for the thickness of the bearers and the bottom board. Wedges are placed under the soles of the props to allow the whole of the centering to be eased before general dismantling takes place. Transverse bearers are nailed to the templates, and the bottom boarding is nailed to the bearers. If the boarding is 1 in. thick it will usually spring sufficiently to conform to practical radii for this type of work.

When the soffit of the frame consists of a series of straight lines instead of a continuous curve, template boards may not be required and a system of direct propping of the bottom boards from the scaffolding, as used in the example in

Fig. 285.—Roof Frame Construction.

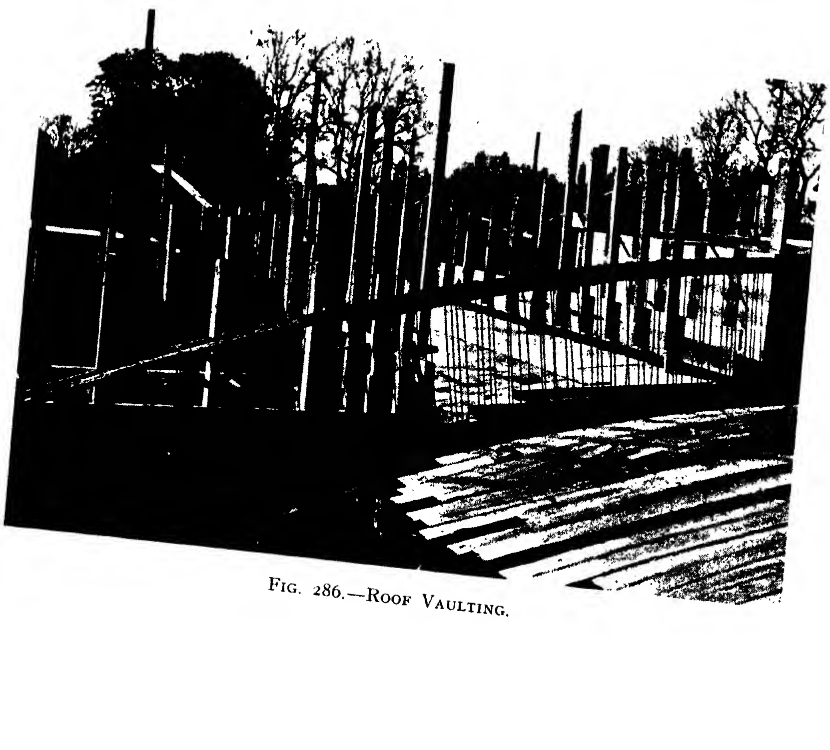


Fig. 286.—Roof Vaulting.

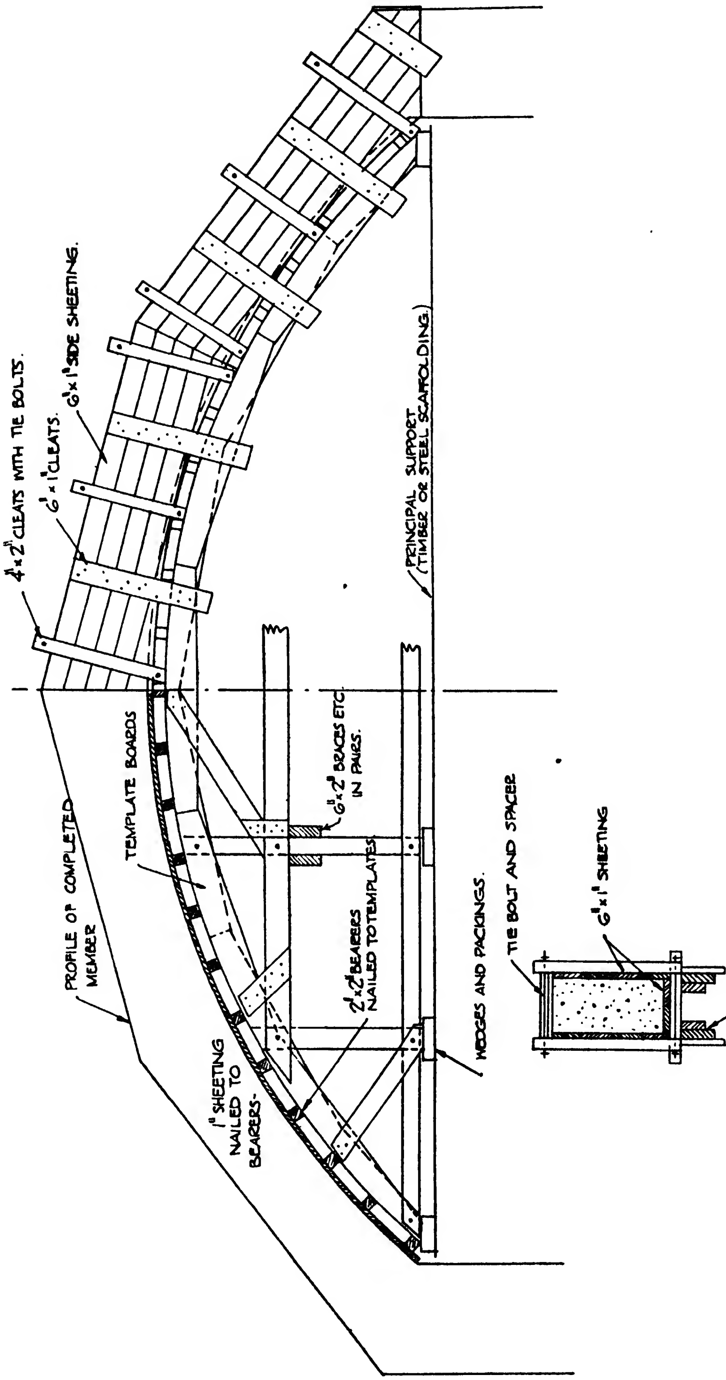


FIG. 287.—ROOF FRAME SHUTTERING

Fig. 285, can be used. A similar method is illustrated in the detail of the apex of a roof frame shown in *Fig.* 288. When a vault slab at the level of the frame soffit is included in the design, longitudinal bearers are fixed between the main supports of the ribs and the sheeting is nailed directly to these bearers as shown in *Fig.* 286. The reinforcement for the frames or ribs is erected after the bottom boards are in position (*Figs.* 286 and 288). The bars should be accurately bent and rigidly fixed in their final position, complete with binders and cover blocks, before the side shuttering for the frames is erected.

The side shuttering should be made in panels that can be removed without damage and re-used on subsequent frames. If the boards run roughly parallel to the soffit of the frame, a minimum amount of cutting is required and only

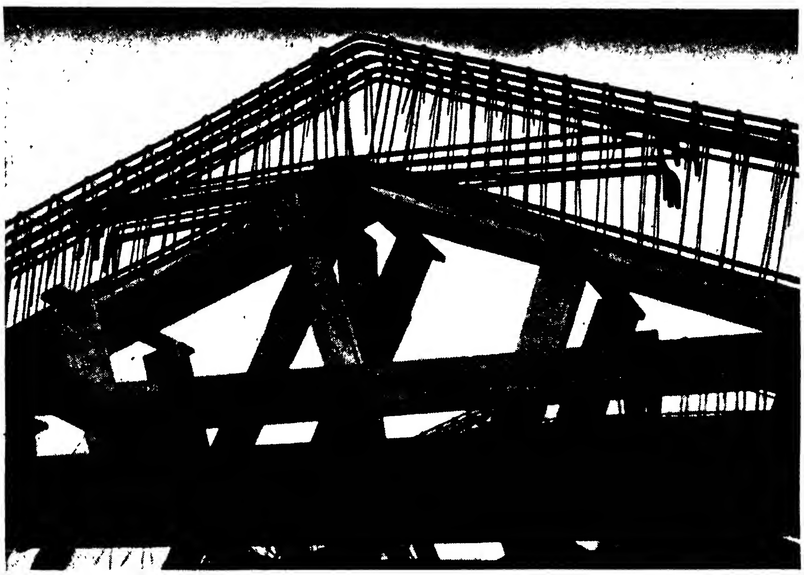


FIG. 288.—APEX OF ROOF FRAME.

the lower board requires to be shaped. Two series of cleats are provided. The first series carries the tie bolts and spreaders as illustrated, and intermediate cleats extend over the face of the templates to assist in preserving the alignment. The sloping members of the frames usually require to be shuttered on the top face; this generally consists of short lengths of board nailed across the tops of the side boards.

The construction of the shuttering for roof frames requires careful work on the part of the joiners. When the contract is not large enough to warrant the provision of power-driven plant on the site, it is often advantageous to make the rib shuttering in a joiner's shop and transport it to the site in sections. This method was adopted for the units shown in *Fig.* 289, where the box-shape of the units is preserved by the battens secured to alternate pairs of cleats.

Framed trusses should be concreted from each end simultaneously, working upwards and inwards. The position of day's work joints should be selected in consultation with the engineer, but should generally lie at right angles to the centre line of the frame. If the frame has a definite apex point, as in *Fig.* 288, there should not be a joint at the apex. The centre section should be concreted last, and if an interval of a day is allowed before doing this the bulk of the concrete in the frame will have had an opportunity to take up its initial shrinkage. For construction of long-span frames reference should be made to the observations on arch construction given in Chapter IX.

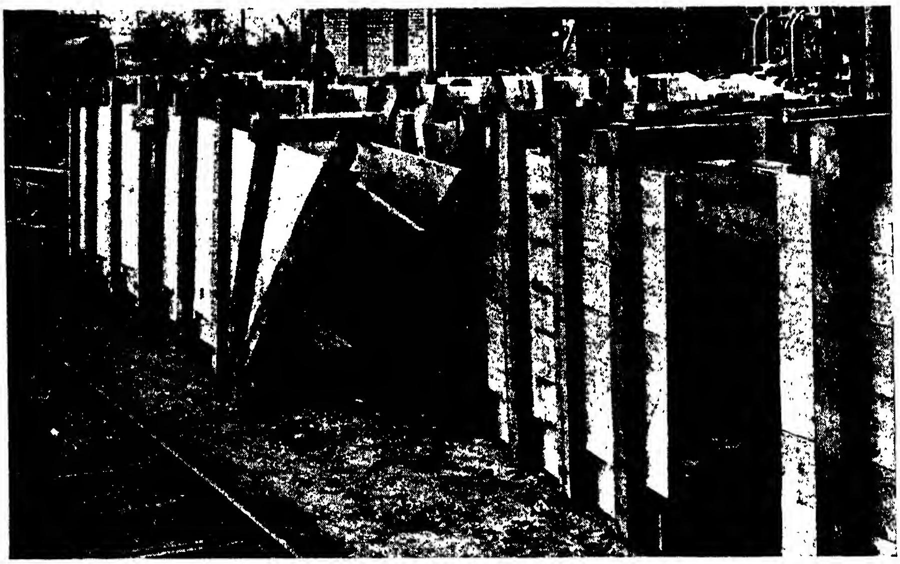


FIG. 289.—SECTIONAL CONSTRUCTION OF FRAME SHUTTERING.

Domes.—Reinforced concrete domes present a problem in shuttering similar to that of roof vaults, but with the additional complexity that the sheeting is curved in two directions. For hemispherical domes, the common procedure, as illustrated in *Fig.* 290, is to erect a series of ribs at close centres following the vertical profile of the dome and acting as templates for the sheeting. The latter is nailed to the ribs and may consist of 2-in. by ½-in. strips arranged circumferentially, the small size of the section enabling the strips to be bent to the curvature of the dome. Towards the top of the dome irregularities may occur due to the more rapid change of shape, but a smooth surface can be obtained by planing the sheeting when it is in position.

The shuttering for the outer surface of the lower part of a hemispherical dome consists of radial ribs similar to those used for the soffit, the sheeting being attached on the inside of the ribs as concreting proceeds.

An example of sheeting for the underside of a large diameter domed surface is illustrated in *Fig.* 291. In this case, owing to the easy curvature, boards of normal widths were used.

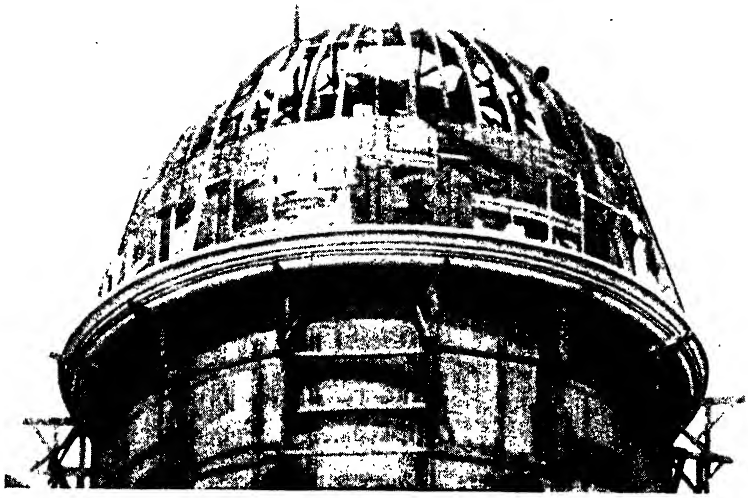


FIG. 290.—DOME CENTERING.

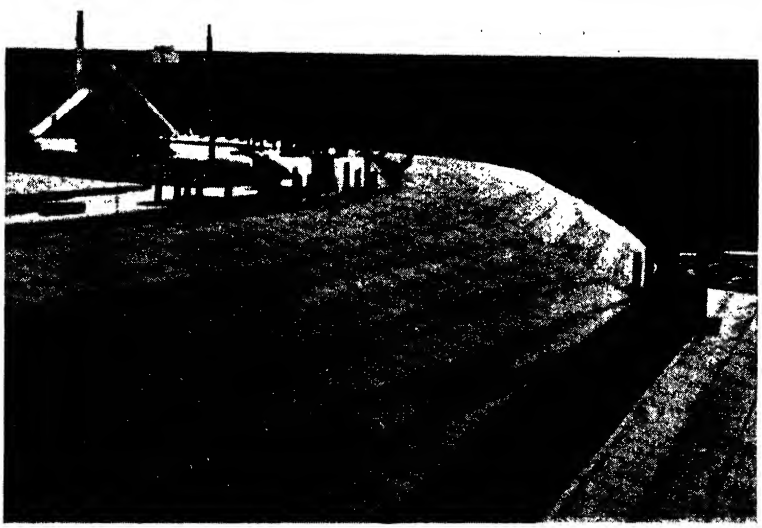


FIG. 291.—SHEETING FOR LARGE DOMED SURFACE.

Provision for Services.

The erection of buildings of almost any class involves the lighting, heating, water, drainage, or ventilation services, and may include arrangements for fixing machinery or plant. In reinforced concrete structures it is clearly an advantage that these installations should be planned before the construction of any particular part of the work concerned, as neglect of this precaution will make it necessary to cut holes in the concrete which may sometimes be detrimental to the strength of the work and is always an expensive operation.

For lighting conduits and other pipes on the underside of floors, circular openings of the required diameter are often left through the webs of tee beams. In the case of water services the holes may be up to 6 in. in diameter, and thus must be taken into account in fixing the inclined bars and bindings in the beams. Holes of this size are most readily formed by building into the concrete a short length of metal or glazed fireclay pipe of sufficient internal diameter to allow the permanent piping to be inserted. The length of pipe can be held in position

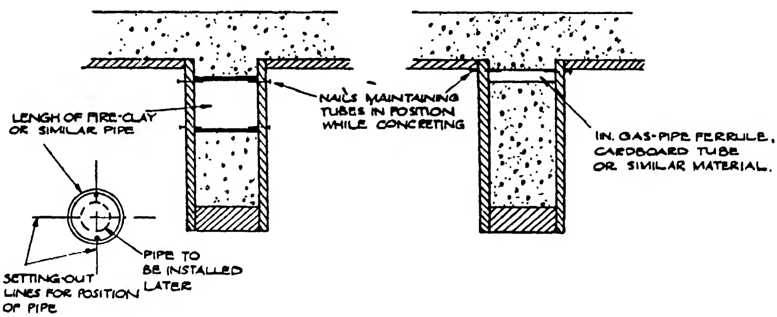


FIG. 292.—FORMING HOLES THROUGH BEAMS.

by two pairs of nails driven through the side boards of the beams as shown in *Fig.* 292. Alternatively, a cylinder rolled from sheet metal and maintained in shape by two or more circumferential wires provides an easily made former for the hole. In either method the former is left permanently embedded in the concrete.

Lighting conduits, cables, gas pipes, or small water-pipes, seldom require a hole greater than 1 in. in diameter, and if the arrangement of the system is not known at the time when the floor is constructed it is good practice to leave a 1-in. diameter hole through the web of each beam. If this hole is placed at midspan and directly below the soffit of the slab it is in a position that is convenient for normal installations, while having negligible effect on the strength of the member. The hole can be formed by a length of gas pipe or cardboard tube inserted between the sides of the beam box as shown in *Fig.* 292.

Lighting conduits, and sometimes gas-pipes, are buried in the thickness of solid concrete floor slabs in offices and residential buildings. This necessitates planning the piping before concreting commences. Cutting into the concrete to bury these services should not be allowed, as much structural damage may be caused, especially in hollow-tile floors. Where cutting chases is unavoidable in the tops of floor slabs, they should run parallel to the main reinforcement, and before cutting chases in any other position the engineer should be consulted.

Coils in panel heating systems are also buried in concrete floor slabs, a typical arrangement being that illustrated in *Fig.* 293. The coils are laid on packings,

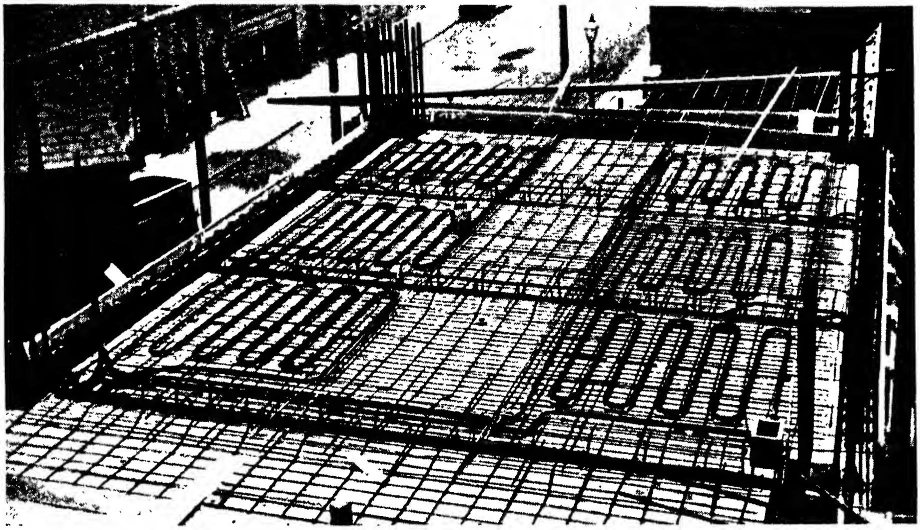


FIG. 293.—HEATING COILS IN SOLID SLAB CONSTRUCTION.

say, small pre-cast concrete blocks, that raise the piping above and clear of the main reinforcement. Lighting conduits would occupy a similar position.

When round holes are to be left through floor slabs for bolts, either of the methods shown in *Fig.* 294 can be adopted. In (*a*) a 1-in. diameter gas-pipe

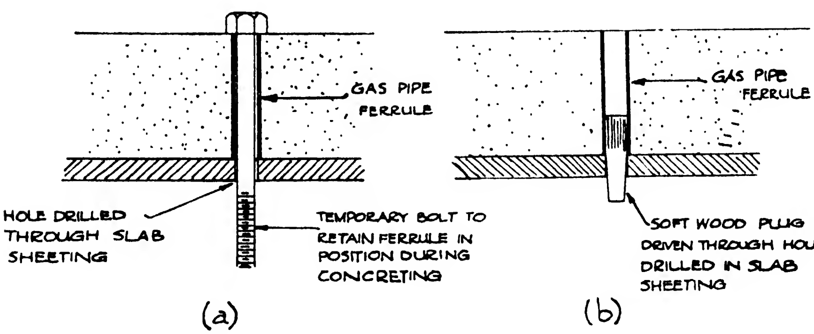


FIG. 294.—FORMING BOLT HOLES.

ferrule is placed vertically on the slab shuttering above a 1-in. hole drilled in the latter. A long $\frac{7}{8}$-in. bolt is inserted temporarily through the ferrule and the opening in the boards; due to its weight, the bolt retains the ferrule in position before and during concreting. The position of the ferrule can be conveniently set out on the slab sheeting by scribed marks or in pencil or chalk. The alternative method, *Fig.* 294 (*b*), is similar to that at (*a*), except that the ferrule is held erect by a softwood tapered plug forced into the lower end of the ferrule and passing through the hole in the sheeting. The plug can be driven out when the concrete has hardened. When it is necessary that bolt holes in a group, as under a stanchion base, should be accurately spaced in relation to each other, the ferrules should be placed in their correct positions by method (*b*). As an additional assurance against tilting, a plug should be inserted in the top of each ferrule to which a timber template frame can be nailed.

The methods shown in *Fig.* 294 are applicable to through bolts, but for bolts that do not extend through the concrete other means must be used. If the bolt can be built into the member before concreting it is only necessary to support the bolt in its correct position in such a way that the specified projection above the finished level of the concrete is obtained. If the bolts are not available when the member is being concreted, holes must be left in the top surface

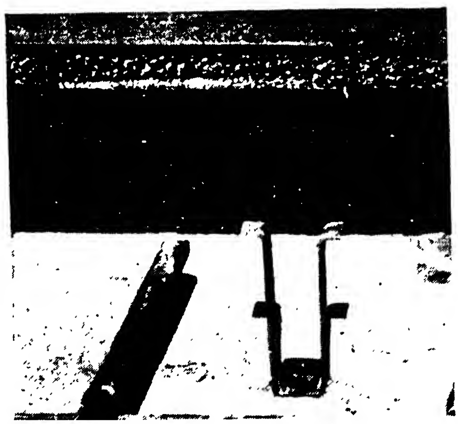


FIG. 295.—ANCHORS FOR BOLTS.

of the concrete if the bolts are to project upwards. For bolts of normal sizes a hole 2 in. to 3 in. square and deep enough to accommodate the buried portion of the bolt is left in the concrete. If washers of abnormal size are specified, the hole must be large enough to allow of their subsequent insertion. If bolts are required to project below the soffit of a slab or beam, rag-bolts or hook-bolts built into the work as concreting proceeds are preferable. The threaded end of the bolt pierces the slab sheeting or beam-bottom board to provide the required projection. Alternatively, special metal anchors, such as those illustrated in *Fig.* 295, can be fixed to the shuttering in the required position before concreting. Other designs of built-in metal hangers or anchors allow the bolt to be laterally adjusted in position. Anchors allow the bolt to be inserted at a later date, thus preventing delays due to non-delivery of the bolts and avoiding the risk of the threads being damaged or filled with mortar.

If provision has not been made for bolts during construction, the necessary holes must be formed after the concrete has hardened. These holes are usually jumped by hand, using a hammer and chisel, but if a power-hammer is available a large number of holes can be cut economically and quickly. Holes formed by a power-hammer may do less damage to the concrete by disintegration than hand-cut holes, and a hole with a smooth bore very little larger in diameter than is required for the bolt can be obtained.

Provision is not usually made during construction for bolts for such small fixtures as rainwater pipes, light fittings, small water-pipes, lightning conductors, etc. Where these must be attached to the concrete the common method is to drill a small hole in the concrete into which a wooden plug, say, 1 in. diameter and 2 in. or 3 in. long, is pushed. The screws or nails attaching the fittings are then driven into the plug. A more efficient and positive attachment results from using fixings of the Rawlplug type. In either method large numbers of holes can be economically cut by a power-hammer with special drills attached. A suitable length of Rawlplug for most purposes is 2 in.

SECTION IV.—FINISHES.

Floor and Stair Finishes.

Special finishes are usually applied to concrete floors in buildings, either to increase the resistance to wear or to improve the appearance. In industrial structures, however, the structural concrete is often left to act as the finished surface. In such cases the concrete should be fairly dry (having, say, a slump not exceeding 2 in.) and should be dense. The floor should be finished with a wooden float, taking care not to trowel the surface to such an extent that water and an excess of fine material are brought to the surface, otherwise the finished floor will eventually become dusty. Excess water should be removed and attempts to absorb this water by sprinkling with dry cement should be prohibited, otherwise dusting of the finished surface will also occur. The slab should be kept damp for a week after depositing the concrete, one or other of the methods already described being adopted. This ensures proper curing and hardening.

Coarse sand should be used for concrete which will be subject to abrasion, and the provision of a coarse aggregate that is in itself resistant to abrasion is self-evident. Hard broken stone is preferable to rounded gravels for wearing surfaces.

Sodium Silicate Treatment.—As an additional precaution against dusting, and consequent rapid wear, the hardened surface can be treated with sodium silicate or a proprietary floor hardener. In the common sodium silicate treatment a high-silica material should be used and the solution should be applied by brush, the first coat lavishly and succeeding coats more sparingly. The number of coats required will depend on the amount of solution absorbed by the concrete surface. Usually two or three coats are sufficient to ensure complete hardening of the surface by conversion of the free lime in the set concrete to silicate of lime. A solution of 1 part sodium silicate to 4 parts water is usual.

Granolithic.—Granolithic is composed of one part of Portland cement to three parts of washed granite chippings from $\frac{3}{8}$-in. or $\frac{1}{4}$-in. gauge down, but excluding dust. Alternatively, the mixture can be two parts of Portland cement to one part of a suitable fine aggregate and four parts of chippings. The consistency of the mixture should be such that it is not so wet that water flushes to the surface when it is being laid nor so dry that a harsh coarse surface is produced. The correct amount of mixing water is best judged by making trial mixes with the proportions of materials it is proposed to use.

The granolithic surfacing may be $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. thick, depending on the anticipated severity of the abrasion and the method of application. The topping is preferably laid within two hours of concreting the main slab, although some specifications require the surfacing to be applied within half an hour of the main slab. The top of the latter should be spade finished, and the surfacing should be tamped into intimate contact with the lower slab and screeded to the required levels. Trowelling the surface should be delayed for two to three hours after placing the granolithic, but not beyond three hours. This interval enables surfacing operations to be carried out before the cement takes its final set and while the topping can still be worked without bringing fine material and excess water to the surface.

If the base slab is not hard when the topping is laid, planks must be supported clear of the concrete as a platform for the workmen laying the granolithic. An objection to the practice of laying the finished surface almost simultaneously with the main slab is that the surfacing is liable to damage during subsequent construction and that precautions must be taken to protect the finish both from incidental damage and from the effects of sun, wind, and rain. The principal causes of damage are pitting by rain, walking over the unhardened surface, and laying timber or other materials on it. Damage due to the last can only be avoided by care on the part of the workmen and others working near the surfacing. Defects due to rain can only be prevented by covering the surface with tarpaulins or other suitable material should a shower threaten.

A topping $1\frac{1}{2}$ in. thick or more is frequently laid as a separate operation, but the simultaneous method is essential when the topping is 1 in. thick or less, since the adhesion between the two materials will prevent "crowning" and curling.

An alternative method that gives a satisfactory monolithic surface for light wear, and that is sometimes adopted for factory floors, consists of working a rich granolithic mortar into the top surface of the wet concrete while screeding the latter in the normal way.

Cases may arise when it is inconvenient or impracticable to lay the topping within two or three hours of the base slab. In such cases, or when a granolithic surface is being applied to an old concrete floor, the surface of the existing concrete should be well swept, hacked, and thoroughly saturated with water. Excess water should be swept off, and a wash of cement slurry, mixed to a creamy consistency, applied immediately before laying the wearing surface, the thickness of which should not be less than 1 in. The surfacing should be kept wet for a week or ten days. To ensure thorough cleansing of old surfaces, the latter should be washed with a 1 to 3 solution of hydrochloric acid. The acid should be washed off with clean water before applying the cement slurry.

TERRAZZO.—Terrazzo finishes are made by mixing one part of Portland cement with $2\frac{1}{2}$ parts of crushed marble of $\frac{1}{2}$-in. gauge (or as otherwise specified) and free from dust. These finishes are laid in a similar way to granolithic in a layer about 1 in. thick, and after curing they are rubbed or ground to a smooth surface by a stone or by machine. The grinding should take place not less than three days after laying the terrazzo. An alternative method is to lay a 1-in. thickness of mortar, upon which, while still soft, crushed marble is sprinkled. The mortar would be composed of two parts of coarse sand to one part of Portland cement. The amount of marble to be used, which can be ¼-in. gauge and must be free from dust, depends on the appearance required, but usually sufficient chippings are sprinkled on to the mortar to ensure that, when rolled in, adjacent pieces almost touch. When the surfacing has hardened it is ground down by machine.

Other floor finishes of the rendering type include mosaics and similar decorative surfacing, for detailed information regarding which the reader is referred to "Concrete Surface Finishes, Rendering and Terrazzo." *

Joints in Floor Toppings.—Where surface renderings are laid on slabs in which there are to be permanent joints, the joint should extend through the finish. It is also advisable to provide a joint through the surfacing where it passes over beams, or at least over main beams, thus minimising the risk of irregular cracking due to reverse bending. Joints should also be provided between the surfacing and walls, while granolithic and terrazzo finishes extending over large areas should be divided into panels. Generally the panels should not exceed 12 ft. square, although larger panels may be convenient in some instances. In the latter case the curing should be thorough and prolonged beyond the normal time. The joints between panels and elsewhere are formed by strips of brass or other metal, ebonite, or a line of mosaic cubes, although between walls and the edge of the finish or at any point where the joint is well concealed, a strip of untarred felt is often used. The object of the joints is to prevent the occurrence of uncontrolled cracks caused by movement in the concrete slab due to shrinkage or temperature changes or caused by shrinkage of the surfacing.

Non-slip Surfaces.—To increase the resistance to wear, stair treads are frequently covered with granolithic or with proprietary materials which combine non-slip qualities with resistance to wear. Non-slip surfaces are commonly formed by surface indentation, and this method is sometimes applied to corridors, entrance halls, or similar areas where structural concrete or granolithic forming the finished surface is subject to abrasion. A suitable finish for stair treads is also obtained by the addition of powdered carborundum, which is lightly trowelled into the surface within one and a half to three hours after placing the concrete. The amount of carborundum dust required is about 6 oz. per square yard of surface.

Floor Coverings.—Independent finishes applied to concrete floors include tiles set in cement mortar, linoleum, rubber, or similar materials glued to the concrete, and wood-block floors laid on a mastic applied directly to the concrete. Details of satisfactory methods of fixing such coverings are obtainable from the manufacturers of the particular type specified. These floor coverings are generally laid by subcontract or, more frequently in building work, the laying of floor surfaces is excluded from the reinforced concrete contractor's contract.

When applying bitumastic and rubber coverings, it is essential that the concrete surface should be absolutely dry, and the following test is recommended by manufacturers of rubber paving. A small quantity of calcium chloride crystals is scattered on an area of a few square inches of the slab and these are

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covered by a piece of plate glass, say 1 ft. square. The edges of the glass are sealed with putty and left intact for three days. If the concrete is dry enough for the application of the paving, the crystals will be unaffected. If the concrete is still damp, the crystals will dissolve and water may appear on the slab and glass.

WOOD FLOORS.—Boarded floors are usually laid with a clear space between the timber and the concrete. One method is to support the boarding on 2-in.

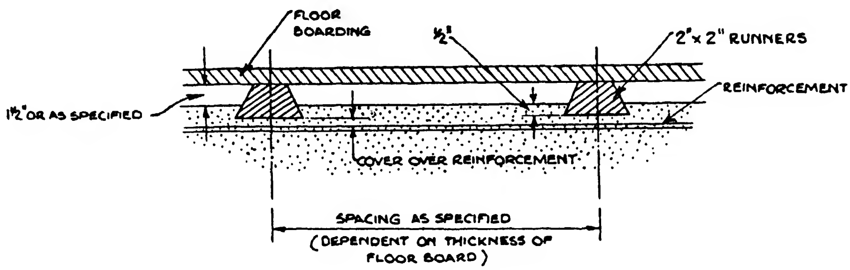


FIG. 296.—TIMBER RUNNERS FOR FLOOR BOARDING.

by 2-in. timber runners which are dovetailed into the concrete, penetrating about ½ in., as shown in *Fig.* 296. The runners are pressed into the concrete while it is still soft, and levelled by a straightedge extending across several adjacent runners. Any runner that projects above the general level when the concrete has hardened must be planed down to preserve a level seating for the boards;

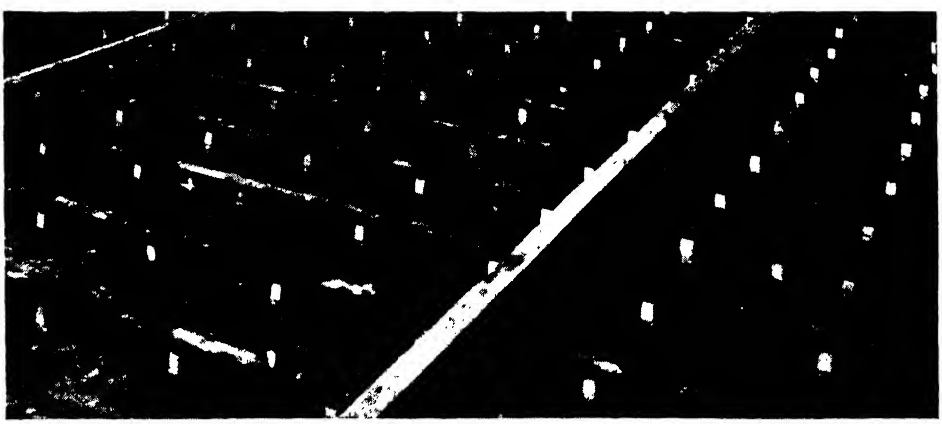


FIG. 297.—RUNNERS SECURED BY METAL CLIPS.

packing strips must be attached to runners that are below the general level. It is essential that the penetration of the runners into the concrete should not reduce the cover of concrete on the reinforcement below the specified minimum. For this reason it is therefore best to lay the runners parallel to the main reinforcement.

A modern method of fixing the runners to concrete slabs is by metal clips

(*Fig.* 297). These have two prongs that are pressed into the plastic concrete and two vertical wings which embrace the runner and lie flat at the level of the top of the concrete. There are no projections to interfere with the operation of screeding. Unless the clips are set out with the aid of a line, difficulty will be experienced in maintaining the clips in a sufficiently straight line to allow the joists or runners to be inserted easily. When the concrete has hardened the two wings are bent up into a vertical position, thus forming the socket in which the runners are placed and secured by nails. Vertical adjustment of the runners is obtained by raising or lowering the timber in the clip and nailing at the side when the desired level is attained.

Roof Finishes.

The dense concrete resulting from taking proper precautions during mixing, placing, and curing, is often found to produce roof slabs that are sufficiently impermeable not to require additional treatment. The roofs of industrial structures and small roof areas of non-residential buildings are often untreated, and if well designed to prevent cracking, water-tightness is attained. As a precaution against damp finding its way through accidental patches of poor concrete, a single coating of sodium silicate solution is a cheap and easily applied treatment. More effective, insofar as it is more permanent, is the following application to flat roofs. A sealing coat of tar is well brushed into the surface of the dry concrete. This is followed by another coat upon which, while still wet, sand and small chippings are sprinkled. Bitumastic paints, which may require to be renewed at intervals, may also be applied to concrete surfaces to produce dampproof roofs.

Asphalt is the normal roof finish for residences, offices, or shops, and is generally laid in two $\frac{3}{8}$-in. or $\frac{1}{2}$-in. coats, successive coats breaking joint. The concrete must be quite dry before applying the asphalt ; in normal conditions a minimum of six days should be allowed between the removal of the curing moisture and laying asphalt. This minimises the risk of blistering. Asphalting is frequently let as a sub-contract to a firm specialising in this work. Owing to the blanketing effect of asphalt on roofs, whereby the temperature of the upper surface of the concrete is often found to be higher than that of the superimposed asphalt, it is usual to provide a white surface to asphalt to reduce the heat absorption. This surface is provided by whitewashing at intervals, or, more permanently, by applying a light-coloured cement paint.

Bituminous sheeting, usually in a double layer, and laid directly on the concrete surface, is an alternative to asphalt, while for insulated roofs of warehouses, timber boarding, separated from the concrete surface by an air space and covered with bituminous sheeting, may be superimposed on the concrete roof. Some specifications require a layer of gravel to be laid on the sheeting in insulated roofs.

Untreated Concrete Walls.

Except that patches of honeycombing are made good by filling them with cement mortar, the faces of concrete walls are sometimes left untouched after stripping the shuttering. Board marks on the walls may be objectionable in exposed situations, especially as the joints between the horizontal boards may be emphasised in the course of time by the accumulation of dust, coal, etc. If the shuttering is made in panels and planed to a fair face before erection there will be few irregularities within the area of a single panel, unless the boards become distorted by being erected and exposed to the weather for some days before concreting. Relative movement of adjacent boards can be prevented if the panels of shuttering are substantially made from two layers of ½-in. boarding, the direction of the boards in the second layer being at right angles to that in the first. In all cases the boards should have wrought edges or be tongued and grooved, and be tightly placed together to prevent the formation of fins of concrete. Any openings or cracks in the shutters should be pointed flush with clay or plaster of paris to prevent the occurrence of fins and leakage of cement grout. Blemishes on plain concrete walls are sometimes caused by leaving the shuttering partly filled with concrete for some days before completing the filling. The concrete may then shrink away from the face of the boards, and when concreting is continued cement grout may find its way through the small space between the concrete and timber and trickle down the completed face. If the concrete cannot be placed in a single operation, the bolts or ties between the opposite faces of the shuttering should be tightened before resuming concreting.

SMOOTH FINISHES.—Smooth wall surfaces are obtainable by using plywood or grainless linings supported by a skeleton construction of framing of timber as shown in the illustrations in Chapter IV. The joints between adjacent sheets of the lining, together with the impressions of nail-heads, are often conspicuous in the finished concrete surface and, although these marks may not be objectionable on small broken areas, on large areas unsatisfactory patterns may be produced. It is therefore advisable to use as large a sheet of lining as is obtainable, and to consult the architect concerning a desirable arrangement of joint marks.

SHUTTER-JOINT MARKS.—Generally the joints should be vertical and horizontal and, failing other suggestions, the horizontal joints should extend continuously through a given length of wall, while vertical joints should be staggered between the series of horizontal lines. This will give a similar, but more subdued, resemblance to the jointing in masonry, and the absence of a continuous vertical joint line will emphasise the horizontal feature of the wall panels. If the general lines of the building or structure are vertical, the vertical joints should be continuous and the horizontal joint lines broken.

If it is desired to suppress joint markings on plain surfaces as much as possible, considerable care is required in cutting and fixing the sheets of plywood so as to avoid spaces between the edges of adjacent sheets. Where, due to bad cutting or other causes, these spaces are unavoidable, a fair surface may be obtained by filling the cavities with marine glue. On the other hand, the joints between the sheets of lining can be exaggerated to make a feature of the finished surface. The sheets can be cut into appropriate random sizes in imitation of stone blocks. Adjacent sheets are butt-jointed and if the nails attaching the lining to the timber framework are not too near the edges, the wet concrete will cause the edges of the lining to rise, thus making distinct and fairly uniform depressions along the joint lines in the finished concrete surface. Another type of emphasis is obtained by leaving a space of ⅛ in. between the edges of adjacent sheets. This leads to the formation of concrete fins along the joint lines, and if the fins are broken off after removal of the shuttering pronounced markings are left.

CONSTRUCTION JOINTS.—Whatever type of shuttering is used it is necessary to pay attention to the position of day's work joints if the wall-faces are to be free from disfigurement. Some remarks on this subject have already been given earlier in this chapter. Where possible plain walls of large extent should be avoided owing to the difficulty of obtaining a uniform surface. The addition of vertical lines, pilasters, horizontal lines, cornices, bands, or similar projections or indentations provides convenient places to locate construction joints and check and correct the vertical alignment of the shuttering. Relief to plain surfaces is cheaply and easily effected by indentations formed by attaching strips of timber to the inner face of the shuttering, as indicated in *Fig.* 298. In this example, 1-in. by 1-in. fillet strips, as used for beam and column chamfers, are nailed to the boarding. The edges of adjacent strips are separated by $\frac{1}{4}$ in. or so to prevent the formation of a sharp edge that may be difficult to preserve straight and undamaged. Strips of the size indicated would give an indentation

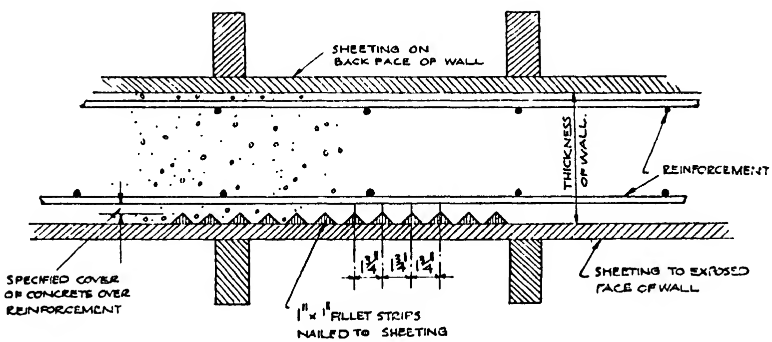


FIG. 298.—FORMING INDENTATIONS IN WALL FACE.

nearly $\frac{3}{4}$ in. deep, but if more pronounced grooving of the surface is required, the fillet strips can be replaced by lengths of 2-in. by 2-in., 4-in. by 4-in., or other timbers halved diagonally. An illustration of a bridge parapet treated in this manner is shown in *Fig.* 299. Similar surface effects can be obtained by nailing sheets of corrugated iron or corrugated cardboard to the face of the shuttering, such methods suggesting a variety of designs, one of which may be suitable in a given instance. In all these methods, however, it is important to ensure that the indentations do not reduce the specified cover of concrete over the vertical and horizontal reinforcement.

The appearance of untreated concrete is greatly affected by the water content of the mix and by the nature and proportions of the cement and aggregate. Variations in the materials produce pronounced colour variations; to avoid these necessitates the use of the same brand of cement and the same type, gauge, and source of aggregate, mixed in identical proportions, throughout a given area. Rapid-hardening Portland cement will generally give a lighter shade of concrete than normal Portland cement, and gravel from chalk districts will also produce a lighter material than that obtained from sources remote from such deposits. A uniform texture can only be obtained by conducting the mixing, transporting, and placing operations in such a way that there is eventually a uniform distribution of the aggregate and mortar on the exposed face. Ramming or tamping should, therefore, be conducted with this object in view, and the shuttering should be tight in order to obtain faces free from honeycombing. The shuttering should be struck as early as possible, avoiding prising the boards away from the concrete by nail-bars or other metal tools that may injure the surface or break off corners. Honeycombed patches should be repaired as soon as the shuttering is removed, while the concrete is green. The patches should be saturated with water and filled with mortar made from the same materials and in the same proportions as the concrete in the wall so as to preserve a uniform colour. The patching material should be mixed as dry as possible and worked

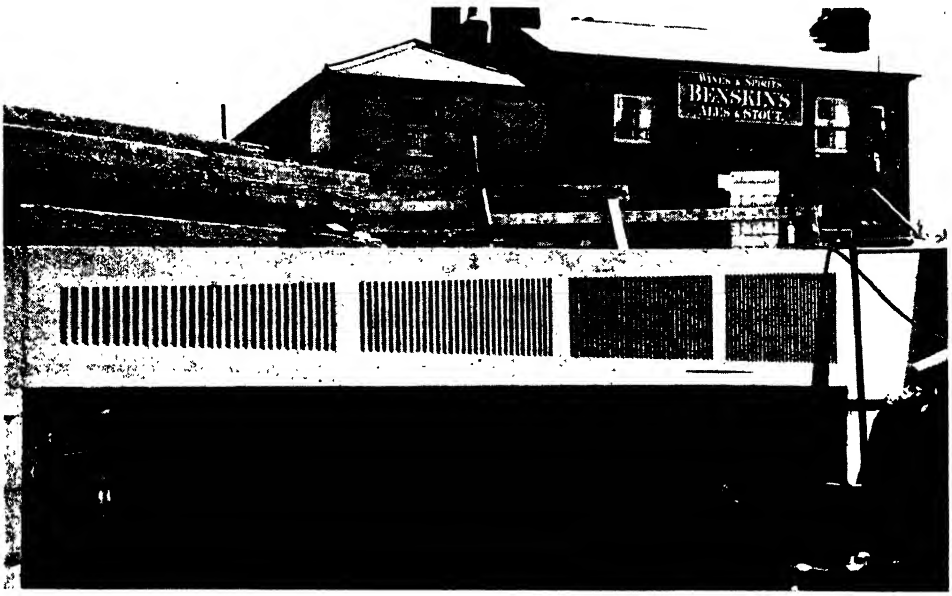


FIG. 299.—INDENTED SURFACE.

into the interstices by a small trowel and smoothed off flush with the concrete face with a wooden float. Any accidental depressions in the concrete face can be treated in a similar manner, preceded by hacking away the cement skin to expose the aggregate in the depression. Vigorous brushing will remove pieces of material loosened by the hacking, leaving a surface that will provide a good key for the patching material. After thoroughly wetting the concrete face, the mortar is applied as previously described.

In buildings the faces of exposed concrete walls are seldom left untreated, and the treatments may extend from simple rubbing down and grout washing to tooling, rendering, or applying special monolithic facings. In the following paragraphs various successful treatments are described, together with consideration of facings of masonry and similar materials suitable for bridge construction.

Treated Concrete Wall Finishes.

The principal methods of treating concrete surfaces without applying renderings or other covering materials include rubbing down by hand or machine, exposure of the aggregate, and tooling the surface. The object of rubbing down is to remove slight projections and produce a uniform colour on the wall. Irregularities due to board marks can be removed in this way, but hacking is required if the projections are greater than can be easily removed by rubbing. Rubbing down small areas is usually done by hand with a hard brick or stone, the surface of the concrete being thoroughly wetted during the process. For large areas, rubbing down by machine, as illustrated in *Fig.* 300, is used. In the example, the use of electrically-driven carborundum wheels to efface board marks on

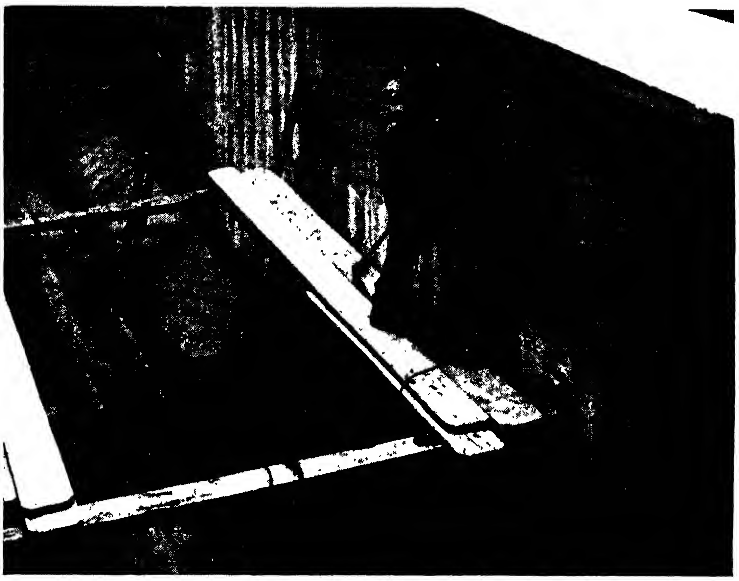


Fig. 300.—Rubbing Down with Carborundum Wheel.

walls is illustrated. These machines remove normal degrees of irregularity, but large projections are more economically cut away by hacking by hand or by an electrical or pneumatic chipping tool. In place of water alone, a sand-and-cement lubricant may be applied with the concrete face when rubbing by hand or machine.

Exposed Aggregate Finishes.—When exposed aggregate is to be the surface feature, steps should be taken while placing the concrete to ensure a uniform distribution of aggregate on the face. This can be done by inserting the spading tool into the concrete after the normal tamping is complete and pressing the coarse aggregate against the shuttering.

A simple method of partly exposing the aggregate is to remove the face shuttering as early as possible and to scrub the concrete with wire brushes after

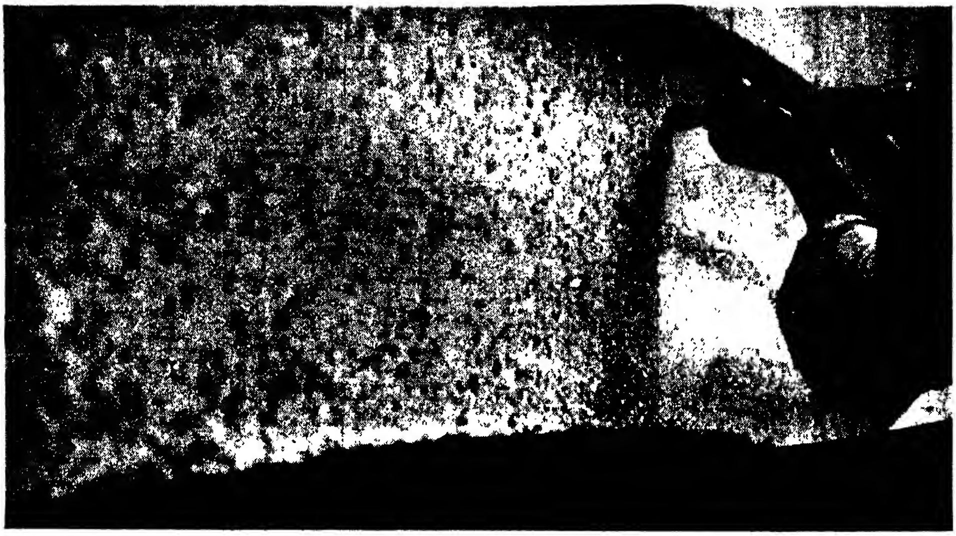


FIG. 301.—BUSH-HAMMERING.

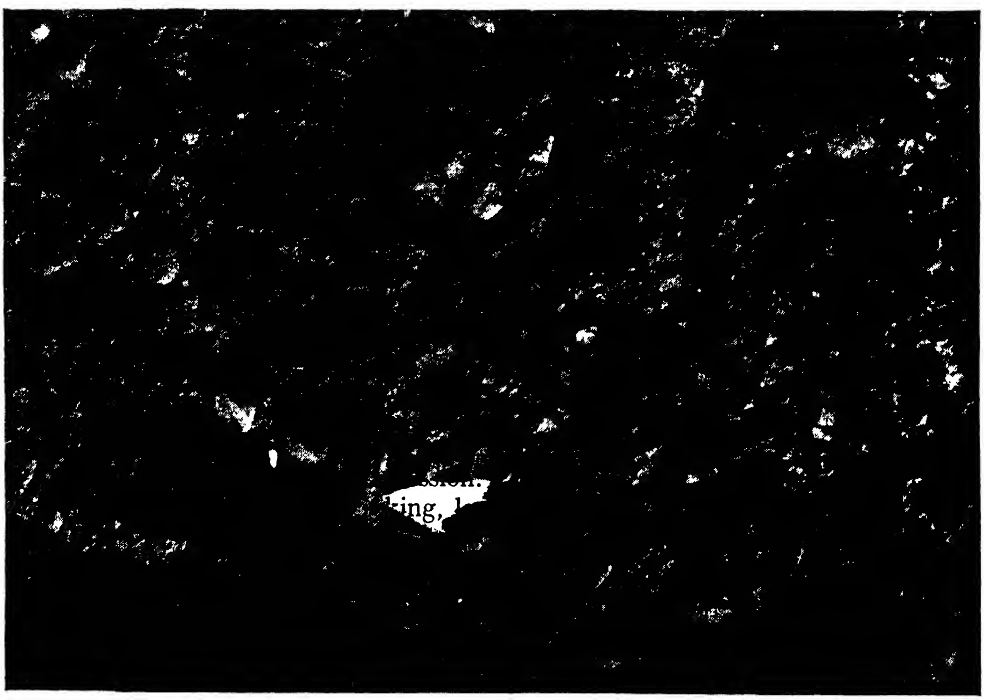


FIG. 302.—BUSH-HAMMERED SURFACE. (*Photograph half actual size.*)

thoroughly wetting the surface with water. This action removes much of the superficial cement, but hardened concrete may be difficult to remove by this means. In warm weather the surface should be treated within six to twenty-four hours of placing the concrete, and within two or three days in cold weather. Washing with a 25 per cent. solution of hydrochloric acid (muriatic acid) before scrubbing will assist in removing the cement skin when hardened, or alternatively the removal of this skin is facilitated by applying a cement-setting retarding liquid to the face of the shuttering. The use of these liquids is dealt with on page 167.

A variety of surface effects can be obtained by tooling the face of the hardened concrete. Bush hammering with an electric or pneumatic hammer fitted with a serrated disc is the most usual method. The discs are about 1½ in. in diameter and have twenty-four teeth; on medium-size hammers, giving about 1,600 blows per minute, these will cut $\frac{1}{16}$ in. into the concrete face, thus removing the cement skin and exposing the aggregate. Many of the larger stones become fractured in this process and become a feature of the surface. Lighter hammers cut $\frac{1}{32}$ in. into the surface. The method of using the tool is shown in *Fig.* 301, while in *Fig.* 302 the effect of a coarse hammer is shown. The normal type of hammer to which bush-hammering discs are attached can be fitted with other tools to produce such effects as full- or semi-reeded surfaces. As bush-hammering is likely to break concrete arrises, it is advisable to leaving a margin of untreated concrete at the corners of walls, columns, and other parts of the building.

Unless the wall surface is to be rendered, no form of surface treatment that removes the cement skin should be applied when impermeability under a head of liquid is an essential property of the wall.

Renderings and Other Applied Wall Finishes.

Cement Washes.—The simplest form of applied finish is a wash of cement grout, which, although frequently used on buildings and structures of industrial types, is not usually recommended for surfaces where architectural appearance is a feature, since these washes are not permanent but tend to flake and discolour with age. Cement-grout washes are made from either ordinary grey, or white, or coloured cements, mixed with water to form a slurry composed of about 14 lb. of cement to each gallon of water. The face of the concrete wall is first cleared of projecting fins, and patches of honeycombing are repaired and the surface rubbed down with a hard stone. After saturating the face with water, the wash is applied thinly and evenly with a brush, working either horizontally or in the direction of the board marks if the latter are distinct. Two coats are recommended, and, to obtain a better finish than is left by the brush, the wash can be rubbed in with a carborundum stone. If the wash is gently sprinkled with water daily for two or three days after its application, the risk of flaking and dusting of the dried-out wash is minimised.

Thin Renderings.—An improvement on cement washed surfaces can be obtained by applying cement by either of the following methods, both of which have proved to be more permanent than grout washes. A smooth finish can be produced by first preparing a key on the hardened concrete face by means of a retarding liquid. The process of "stoning-up" the face is then carried out by applying with a wooden float a mortar in a layer of about $\frac{1}{8}$ in. minimum thickness. A suitable mortar giving successful results can be prepared from one part of white cement to two parts of crushed Portland stone graded from $\frac{1}{4}$ in. down, with $\frac{1}{16}$-in. material predominating. A rough texture is obtained by "spraying" a white cement through a cement-gun. This operation (*Fig.* 303) follows the preparation of the concrete to form a key. In one instance, a surface applied in this manner to a partly hacked concrete face was found to withstand considerable abrasion during winter storms. A single coat, if sprayed on, is usually sufficient, but about twelve times more cement is used than is required to cover an equal area of brushed work. The cement-gun works at an air pressure of 80 lb. per square inch and delivers about 13 cu. ft. per minute.

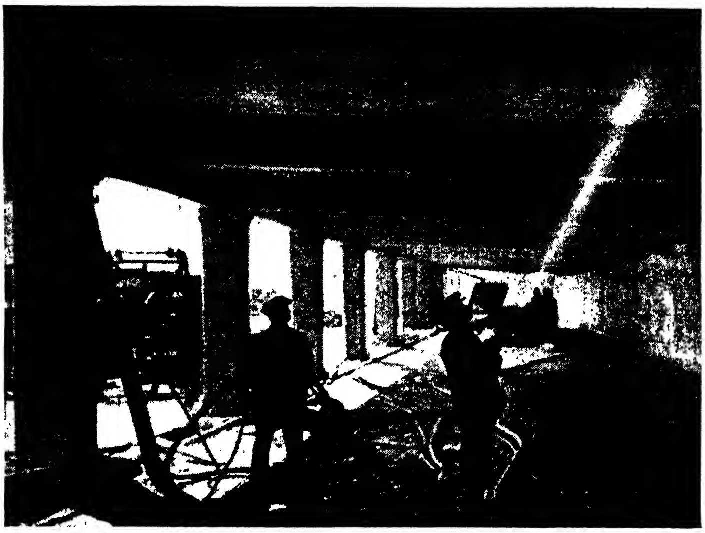


FIG. 303.—APPLYING CEMENT FINISH BY GUN.

KEY FOR RENDERINGS.—The primary requirement for rendering a hardened concrete surface with mortar is a good bond or key between the two materials. Most methods of obtaining the bond aim at roughening the surface of the concrete by hacking to produce indentations spaced at intervals of 3 in. to 12 in. all over its surface. Although in numerous cases this method may be successful, a more efficient bond is obtained by roughening the whole surface. This process can be carried out by hand, as shown on *Fig.* 304, but if large areas are involved it is laborious and expensive, especially if the concrete face has appreciably hardened before hacking commences. If a light electrical or pneumatic hammer, such as is used for bush hammering, is available, the surface can be roughened by fitting a chisel or point in the tool-holder.

For most structures, where large or small areas are involved, the cheapest method is to paint the shuttering with a liquid that retards the setting of the cement skin. Upon removal of the shuttering the skin can be brushed off with a wire brush, exposing the aggregate as illustrated in *Fig.* 305 and leaving a surface that is suitable for the application of rendering. The depth of penetration of the effect of the liquid may be less than $\frac{1}{8}$ in. and the retarding effect is not permanent, since the delayed setting of the cement is resumed when the concrete face is exposed to the air. The liquid is applied to the face of the shuttering

FIG. 304.—PREPARATION BY HAND OF SURFACE FOR RENDERING.

in contact with the concrete surfaces it is proposed to roughen. A retarding liquid containing spirit is very fluid, and can be spread on the shuttering with a long-handled tar brush or similar implement. It can be applied before or after erecting the shuttering, but the reinforcement should not be in place when applying the liquid to shuttering in position, otherwise it is very difficult to

FIG. 305.—CEILING TREATED WITH LIQUID FOR RETARDING THE SETTING OF CEMENT.

prevent it getting on the bars. Retardation of the setting of the cement in the concrete around the bars is most undesirable, and any liquid accidently falling on the reinforcement must be completely removed before concreting. The treated concrete surface should be brushed with wire brushes on the same day as the shuttering is removed, when with little effort the cement skin will come off like wet sand. If two or three days are allowed to elapse between striking and brushing more difficulty will be experienced in exposing the aggregate, while after ten days the surface may be so hard as to prohibit satisfactory treatment by brushing. Retarding liquids are generally designed for use with normal 1 : 2 : 4 slow-setting Portland cement concrete with which the shuttering can be left in position for seven to ten days without making brushing difficult, although the actual time will depend on the prevailing temperature and weather conditions.

As an alternative to brushing, the cement skin may be washed off with a strong jet of water immediately after removing the shuttering. If the concrete has begun to harden, making removal by brush or water-jet difficult, sand-blasting has been found successful in removing the skin and exposing the aggregate.

Among other methods of obtaining keys for rendering is attaching lengths of cord to the inner face of the shuttering. When the latter is removed the cords remain attached to the boards and leave a series of grooves in the concrete face which provide a key to the rendering. The diameter of the cords used is $\frac{1}{4}$ in. or $\frac{5}{16}$ in. and they are attached in roughly parallel lines 6 in. to 12 in. apart.

White and Coloured Renderings.—Mortar renderings on concrete should be applied with a wooden float after thoroughly cleaning and wetting the surface. Any surface to which renderings are to be applied should be free from oil. Normally the mortar should not be richer than one part of Portland cement to three parts of fine sand, but special mortars can be made to reproduce desired appearances, the matching of natural stone having been successful in many cases by using crushed stone as the aggregate in 3 : 1 mortars. The tone of fine-grained grey stone can be reproduced in building work by using a mortar rendering made from the fines screened out of $\frac{3}{8}$-in. granite chippings supplied for granolithic floors.

Coloured and white Portland cements are commonly used for rendering. White Portland cement costs about three times as much as ordinary grey Portland cement. White surfaces are therefore more economically obtained by a rendering applied to the face of a wall constructed in ordinary Portland cement concrete than by casting the whole wall in white cement. Alternatively a white cement facing can be cast at the same time as a concrete backing of grey cement. Coloured mortars are obtained by mixing coloured sands with ordinary, white or coloured cements. Since it is difficult to maintain a uniform tone throughout a mixture, mixing the materials on the site is not recommended, especially as many varieties of coloured cements can be purchased which are made by processes that ensure satisfactory results. For renderings or for integral facings, white and coloured cements can be obtained ready mixed in suitable proportions with graded aggregates. Apart from the more uniform tone assured by the use of these mixtures, there is the advantage that the most suitable aggregate can be used irrespective of the locality of the work.

White or coloured cement renderings are usually applied in two coats after a key has been formed on the face of the concrete wall or floor. The base coat is about $\frac{1}{2}$ in. thick and is applied to a thoroughly saturated surface. The mixture for this coat should be 1 part of cement to 3 parts of sand, $\frac{1}{8}$-in. maximum gauge, and the surface of the coat should be left with combings to provide a key for the finishing coat. The latter should not be less than $\frac{1}{8}$ in. thick and should also be applied to a thoroughly wetted surface, but not within thirty hours of applying the base coat. The composition of the finishing coat would depend on the type of surface required and should be mixed in accordance with the instructions issued by the manufacturers.

CRACKING OF RENDERINGS.—With all renderings precautions must be taken to prevent crazing and other defects. Crazing or more severe cracking may be due to the mortar being too rich or too wet, to lack of curing, or to undue stress. Shrinkage, the prime cause of crazing, may be reduced by limiting the amount of cement, by keeping the surface damp for as long a period as possible (at least 7 to 10 days), by protecting from frost, sun and wind and by using only just sufficient water to make a workable mix. Adjustments to the grading of the sand may help to reduce the water content, and a few trial mixes are advisable when using untried materials. Since it is the relative shrinkage between the concrete backing and the facing that is most important, renderings should be applied to concrete walls as early as possible.

Examination of numerous examples of rendered surfaces leads to the conclusion that cracking due to flexural and other stresses is not common in buildings and bridges where the dead load constitutes the bulk of the "actual" total load (not necessarily the design load), but in cases where vibrations or fluctuating stresses are encountered, as in crane gantries, rendering is not so successful. In one instance of a steel bridge faced with concrete, surface cracking was avoided by temporarily loading the girders sufficiently to obtain the dead-load deflection before applying the facing. As the construction of the concrete deck and parapets advanced the temporary loads were reduced to maintain a constant deflection.

PAINTED SURFACES.—Other surfacings applied to concrete surfaces include paints and distempers. Most of these are proprietary materials and should be applied in accordance with the manufacturer's instructions. It is usually recommended that, before applying washable external distempers, the concrete surface should be quite dry and should be treated with a solution of zinc sulphate. With oil-base waterproof paints, the concrete surface should, after drying out, be treated with a solution of 4-lb. of zinc sulphate to one gallon of soft water and left for forty-eight hours. A primary coat of boiled linseed oil is then applied, followed by successive coats of waterproof paint. For other finishes the concrete surface should be cleaned down and the irregularities removed. A dead smooth surface is not essential if a stippled finish is desired. A coat of primer precedes the application of two finishing coats.

Before applying white or coloured cement paints or similar coverings all loose material must be removed from the concrete surface by wire-brushing. The surface of the concrete should be wetted but the cement paint should not be applied whilst water is standing on the surface. The paint is applied by a brush and a second coat can be applied after an interval of twenty-four hours.

BRICK AND STONE FACINGS.—Facings of masonry, coloured pre-cast slabs, or brickwork are frequently applied to exposed faces of walls of buildings, swimming baths, spandrels and arch ribs of bridges. Stone or brick facings for bridges are usually set in mortar and erected in advance of the concrete, thus in part acting as shuttering for the latter. If the facing material is porous, the surfaces in contact with the concrete should be saturated before depositing the latter.

The thickness of applied stonework is less than would be required for an independent masonry wall, and practical thicknesses range from 4 in. to 9 in. Frequently 5-in. thick blocks with occasional 9-in. headers are provided. The facing is tied back into the concrete by $\frac{1}{2}$-in. diameter galvanised iron ties, copper clamps 1 in. wide by $\frac{1}{4}$ in. thick and 9 in. long, or other similar designs in non-corrosive metals. One end of the tie is cemented into grooves in the ends of the stones, the other being bent at right-angles to obtain an anchorage in the concrete. With a system of header blocks in vertical facings, ties are not always provided.

When brick or stone facings are applied after the completion of the concrete work, the ties are built into the concrete when it is placed, care being taken to place them so that they register with the joints in the facing or with the anchorage holes in the stones.

Facings to sloping walls, as stone pitching to sea walls, can be conveniently applied by forcing the stones into the concrete backing while the latter is still wet, afterwards pointing up the external joints.

In ornamental work, where the exposed faces are stonework placed against structural reinforced concrete, the stonework can be economically used as shuttering if it can resist the pressure of the wet concrete. If the stability is doubtful a few raking struts can be provided with a saving over close-boarded shuttering.

The attachment of hollow decorative terra-cotta blocks to concrete faces requires consideration during the design, since these are sometimes added after the structural concrete has been placed. It is frequently necessary to leave bars or bolts projecting from the concrete to bond with the concrete filling the blocks, and holes should be left in the blocks to allow the filling to be placed.

In other forms of applied wall finishes, it is often necessary to leave chases in the face of the structural concrete or to build in timber fillets. Care should be taken to ensure that the reinforcement is given the requisite minimum cover of concrete and that the bars are not exposed at the chases.

Concrete Slab Facings.—Polished, coloured, precast concrete slabs, used for facing walls of buildings and swimming baths, are generally laid simultaneously with the wall. The slabs, about 2 in. thick, have wire ties projecting from the back and are built up one course high, the concrete in the wall behind being placed after twenty-four hours to a level 3 in. below the top of the slabs. The second and subsequent courses are built up in a similar way. Close-boarded shuttering to the front face of the wall is unnecessary, as the slabs form the vertical shuttering. The slabs should be supported by a framing consisting of 2-in. by 1-in. horizontal battens spaced at about 9-in. centres and supported by 2-in. by 4-in. vertical soldiers and raking struts.

Integral Facings.

Special concrete and mortar facings are commonly provided by casting the surfacing monolithic with the concrete wall. In this practice the two materials are placed simultaneously, being divided by a metal plate, say, 4 ft. or 5 ft. long and 2 ft. deep. The plate may be kept in position by angle stops as shown in *Figs.* 306 and 307, which produce the desired thickness of finish. The facing material is placed first and is followed immediately by the backing concrete, the level of which is kept 1 in. or so below the level of the surfacing to prevent

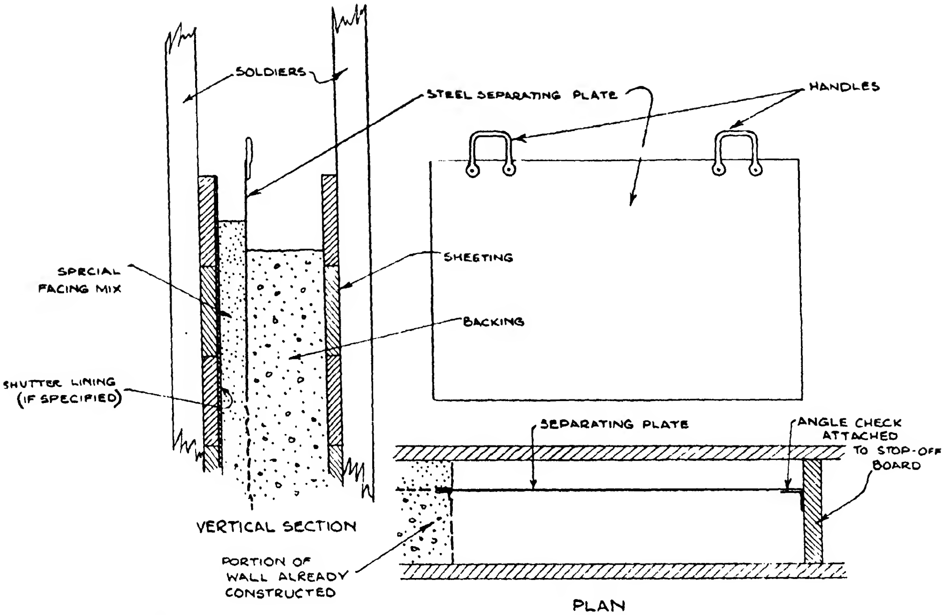


FIG. 306.—INTEGRAL FACINGS.

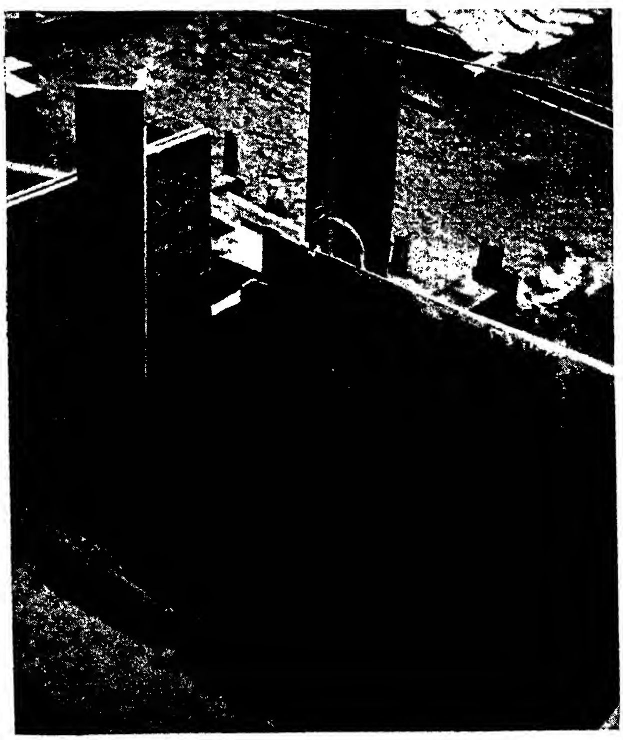


FIG. 307.—SLIDING PLATE IN USE.

staining on the surface. The metal sheet is then slowly lifted and removed, the two materials thereupon bonding together. On no account should the sheet be left in position until the initial set occurs. *Fig.* 307 shows the operation of forming integral facings in progress. The plate is usually $\frac{1}{8}$ in. thick or may be made of 16-gauge metal. In some cases a sheet of $\frac{1}{2}$-in. or $\frac{3}{4}$-in. gauge expanded metal has been permanently fixed to separate the two mixtures instead of using a removable metal plate. This method is quite efficient but is more expensive than the moving plate method.

The special facing material may be 1 in. to 2 in. thick for the walls of buildings, but as much as 6 in. is sometimes applied in a similar manner on large monolithic blocks in bridge and dock work. The backing, which is usually Portland cement concrete of normal mix, may be of any thickness, say, 3 in. minimum for panel walls. When a keyed joint is required between two adjacent panels the necessary indent in the end of the first section concreted can be formed by attaching by nails a length of steel angle to the stopping-off board as illustrated in *Figs.* 306 and 307.

When using the sliding metal sheet already described, it is necessary to ensure that the tamping of the upper layers of the backing does not force the grey concrete through the thin surfacing of white or coloured concrete and cause objectionable veins. The surfacing may be ordinary granolithic or a 1 : $1\frac{1}{2}$: $2\frac{1}{2}$ mix incorporating special aggregates or coloured cements ; white cement finishes are much used on the walls of buildings and swimming-baths. White cements already mixed in correct proportions with sand for the provision of monolithic facings are obtainable. Among successful mortars that have been site produced are the following : (i) three parts of crushed whinstone to one part of coloured cement, the aggregate being graded from $\frac{3}{4}$ in. to $\frac{3}{8}$ in. with fine crushings used as sand ; and (ii) a mix of one part of coloured Portland cement to three parts of crushed yellow sandstone from $\frac{3}{8}$ in. to dust. Effective results are obtained by bush hammering these finishes to expose the aggregate.

Compositions successfully imitating existing natural stones have been prepared and used in numerous cases, especially in bridge work. The appearance of granite has been matched by a mix of one part of Portland cement to two parts of screened sand to four parts of granite chips graded from $\frac{3}{8}$ in. to $\frac{1}{8}$ in. In another instance, a successful attempt to match a grey granite involved the use of six materials : granite chippings, calcined flint, nickel-steel slag, river sand, ordinary Portland cement, and white cement. At close quarters the texture of this facing differs considerably from granite, but from the distance at which the work is ordinarily viewed the white and black particles in the light grey matrix give the correct tone.

Internal Finishes.

The internal finishes on walls and ceilings of concrete buildings are in most cases specified in detail by the architect. In warehouses, basements, and industrial structures the concrete may be simply left as it comes from the shuttering or may be rubbed down with carborundum stone or similar hard substance and may subsequently be given a coat of cement grout, whitewash, or a skimming coat of cement. Plaster renderings are more common on buildings where appearance is important, and the remarks on the application of mortar finishes given previously apply equally to internal work. If lime plasters are used the reinforcement should not be exposed, and any poor patches in the concrete wall should be repaired with cement mortar.

To avoid the cost of hacking the soffits of slabs to receive ceiling renderings, dry stone chippings may be sprinkled on the centering before placing the concrete. A good key is obtained in this way. It is necessary, however, to be careful that the reinforcement is not exposed, and when this method is adopted more than the normal cover should be allowed to ensure that the bars are amply protected.

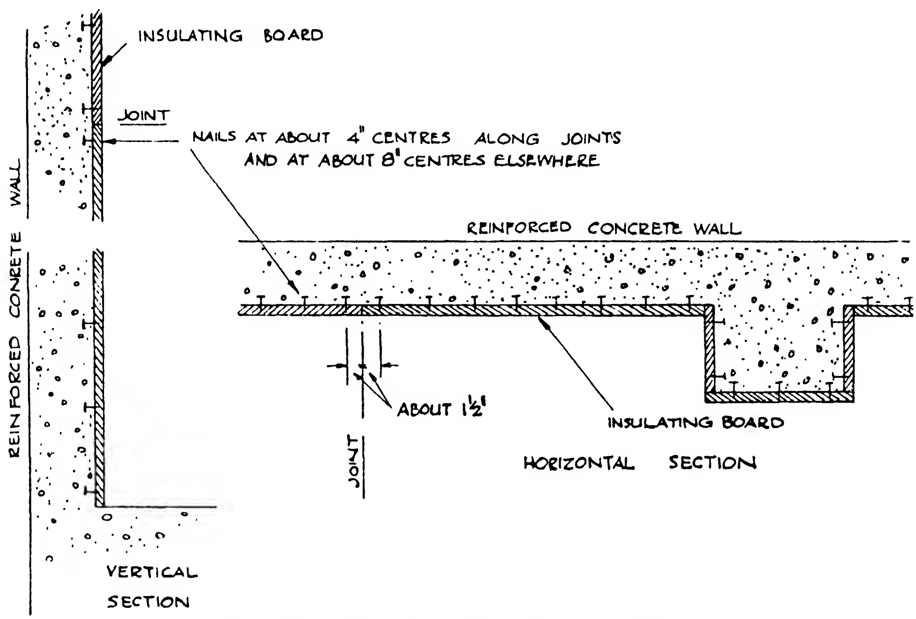


FIG. 308.—ATTACHING INSULATING BOARDING.

The walls and ceilings of domestic and office buildings may be lined with fibre board, which may be left exposed, distempered, or given a skimming coat of plaster. The most convenient method of attaching the boarding is to use it to line the shuttering; in the construction of walls the boarding may be lightly nailed to the timber. The heads of a second series of nails driven into the lining project into the concrete so that when the shuttering is removed the lining remains attached to the concrete. A detail of this method is shown in *Fig.* 308. If the boarding is placed after the completion of the concrete work, the concrete must be drilled and plugged.

Cost of Surfacing Processes.

The following notes relate to the labour units and quantities of material involved in producing the more common types of concrete surfaces previously described. No account is taken of the costs of attending skilled labour, scaffolding, hoisting, or similar equipment needed in elevated work. Unless otherwise stated, the mixing and distribution of mortars or other renderings are also excluded. Where the actual costs of equipment or material are given, these must be considered as an indication of approximate relative values rather than as absolute costs. In such items as making good and rubbing concrete wall surfaces, the amount of labour required depends upon the quality of the surface, which in turn depends on the quality of the shuttering and the care exercised in depositing the concrete. The hardness attained by the concrete surface affects the cost of wire-brushing and similar processes. The labour costs are also affected by the extent of the work, large plane areas, in general, being less expensive per unit area than areas broken by openings or ornamental features. Walls erected with continuously-moving shuttering require less rubbing down than when panel shuttering is used, since the surfaces have fewer imperfections and the finishing operations are carried out when the concrete is twenty-four hours old or less. The labour items are expressed in square yards of finished surface per man-hour ; unless otherwise stated, the time taken by various tradesmen and labourers is reduced to the equivalent of labourer-hours.

FLOOR, STAIR AND ROOF FINISHES:

Labour only: Floating surface with wooden float . . 10 sq. yd. per man-hour.
Trowelling surface . . . . 3 to 10 sq. yd. per man-hour.
Carpenter setting screeds to give finished levels . 20 sq. yd. per man-hour.
Laying, screeding, and trowelling granolithic or similar material to a smooth finish . . . . 4 sq. yd. per man-hour.
Mixing granolithic or other mortar, distributing and laying only, 1 in. thick (other thicknesses inversely proportional) . 2 to 3 sq. yd. per man-hour.
Sweeping, hacking and wetting old concrete surface before laying granolithic or similar material . . . . . 6 sq. yd. per man-hour.
Applying cleaning acid to old surface . . . 24 sq. yd. per man-hour.
Applying cement grout to cleaned surface before laying granolithic 20 sq. yd. per man-hour.
Trowelling surface of granolithic, or similar material, in panels 3 sq. yd. per man-hour.
Placing and removing sand or similar material, 1 in. thick, for curing 5 sq. yd. per man-hour.
Applying by brush, silicate of soda or other floor-hardening solution 12 sq. yd. per man-hour.

Materials: (See later for quantities of materials in renderings.)
Terrazzo: 1 cwt. of Portland cement and 3 cwt. of marble chippings applied ¾ in. thick covers . . . . 5 sq. yd.
Non-slip surfacing: 10 lb. of carborundum dust covers . . . . 27 sq. yd.

Inclusive Prices: Floating or trowelling surface with 1 part of Portland cement to 2 parts of sand, rendering ¾ in. thick . . 1*s.* 9*d.* per sq. yd.
Granolithic: 1 in. thick . . . . 3*s.* 6*d.* per sq. yd.
1½ in. thick . . . . 4*s.* 6*d.* per sq. yd.
Portland cement and sand screeding ½ in. thick for setting tiles, wood blocks, etc. . . . . 1*s.* 6*d.* per sq. yd.
Asphalt applied to flat roofs, ¾ in. thick (extra for skirtings and fillets) 6*s.* to 7*s.* per sq. yd.

WALL FINISHES:

Labour only: Making-good honeycombed patches, cutting off fins, etc. 10 sq. yd. per man-hour.
Rubbing down with carborundum stones by hand with cement grout lubricant, after making-good and removing fins . 2 to 4 sq. yd. per man-hour.
Rubbing down with carborundum stones by hand including making-good, etc., using cement grout . . . . 1½ to 2 sq. yd. per man-hour.
Superficial rubbing down by power operated disc machine (operator's time) . . . . 10 sq. yd. per man-hour.

Removing skin from moderately good surface with power-operated disc machine (operator's time) . . . . . . 6 sq. yd. per man-hour.
Rubbing to uniform colour with power-operated disc machine (operator's time) . . . . 6 sq. yd. per man-hour.
Grinding surface with power-operated disc machine from average surface (operator's time) . . . . . . 3 to 4 sq. yd. per man-hour.
Wire-brushing green surface to smooth finish . . 1 to 2 sq. yd. per man-hour.
Materials: 1 cwt. of cement used for rubbing down will cover . . . 100 sq. yd.
Equipment: Electrically-driven carborundum disc machine . . . £20 each.
Carborundum discs (each will grind about 8 sq. yd.) . . . 1*s*. 8*d*. each.
Cables, including plug adapters, etc. . . . . 1*s*. to 1*s*. 3*d*. per lineal yard.

INTEGRAL FACINGS:
Labour only: Mixing and placing materials, with use of sliding plate, in facing 2 in. thick (other thicknesses inversely proportional up to 4 in.) . . . . 1 to 1½ sq. yd. per man-hour.
Materials: See later for materials in renderings.
Supply and fix small mesh expanded metal in place of using metal plate . . . . 2*s*. per square yard.

INDENTATIONS:
Extra net labour and materials, cutting and fixing 1-in. by 1-in. fillet strips to face of shuttering to form indented surface (one use only assumed) . . . . 1*s*. 6*d*. per square foot.

BUSH-HAMMERING AND OTHER TOOLING:
Labour only: Bush-hammering (operator's time only) . 2 to 3 sq. yd. per man-hour.
Equipment: Medium weight hammer including bush-hammer tool-holder (power consumption 350 watts) . . . . . £30 each (approx.).
Light hammers (250 watts) . . . . £20 each (approx.).
Heavy hammers (420 watts) . . . . £40 each (approx.).
Bush-hammer discs, each serving for an area of 10 to 12 sq. yd. . 1*s*. 6*d*. each.
Cables, including plug adapters . . . . . 1*s*. to 1*s*. 3*d*. per lineal yard.
Inclusive Prices: Bush-hammering by pneumatic machines, including compressor equipment, attendance, etc., plain work . 2*s*. 6*d*. to 3*s*. 6*d*. per square yard.
Ditto, on intricate work, up to . . . . . . 5*s*. per square yard.

CEMENT WASH OR SPRAY:
Labour only: Cleaning fins from face, making-good honeycombed patches and applying single coat of cement wash . . . 2 sq. yd. per man-hour.
Making-good and preparing fair finish, rubbing in cement grout with float followed by brushing . . . . . 2 to 3 sq. yd. per man-hour.
Applying only, single coat of cement wash by brush . 8 sq. yd. per man-hour.
Materials: 1 cwt. of cement (coloured or otherwise) mixed with 8 gallons of water brushed on smooth surface in single coat covers . . 500 sq. yd.
1 cwt. of cement (coloured or otherwise) mixed with 5½ gallons of water sprayed on by cement-gun in single coat covers . . 40 sq. yd.
1 cwt. of coloured cement for application as paint . . . . 56*s*. per cwt.
Inclusive Prices: Rub down and apply single coat of cement wash 1*s*. 2*d*. to 2*s*. per sq. yd.

PAINTS:
Labour only: For labour preparing surface, see "Wall Finishes."
Applying single coat of primer or paint by brush to prepared concrete surface . . . . 8 sq. yd. per man-hour.
Materials: For three-coat treatment . . . . . . 1*s*. 6*d*. per square yard.
Inclusive Prices: Preparing surface, applying primer and finishing coats 2*s*. 6*d*. per sq. yd.

RENDERINGS:
Labour only: Hacking by hand occasional indentations to form key 10 sq. yd. per man-hour.
Hacking by hand complete face of concrete to form key . 2 sq. yd. per man-hour.
(The foregoing are average values; the actual areas will depend upon the hardness attained by the concrete at the time of hacking.)
Hacking by power-hammer occasional indentations (operator's time) 20 sq. yd. per man-hour.

Wire-brushing to expose aggregate while concrete is still green, without using cement setting retarder (depending on hardness of concrete) . 1 to 5 sq. yd. per man-hour.

Applying retarding liquid to shuttering . . . . . 6 sq. yd. per man-hour.

Wire-brushing green surface treated with retarder to expose aggregate 4 sq. yd. per man-hour.

Materials: 1 cwt. of Portland cement as neat mortar rendered 1 in. thick covers . . . . $1\frac{1}{2}$ sq. yd.

1 cwt. of Portland cement with $1\frac{1}{4}$ cu. ft. of sand (1 : 1 mortar) rendered 1 in. thick covers . . . . $2\frac{1}{2}$ sq. yd.

1 cwt. of Portland cement with $2\frac{1}{2}$ cu. ft. of sand (1 : 2 mortar) rendered 1 in. thick covers . . . . $3\frac{1}{2}$ sq. yd.

1 cwt. of Portland cement with $3\frac{3}{4}$ cu. ft. of sand (1 : 3 mortar) rendered 1 in. thick covers . . . . 5 sq. yd.

(For other thicknesses of rendering the areas covered are inversely proportional to the figures given.)

1 cwt. of white cement and aggregate mixture applied as $\frac{1}{4}$-in. rendering covers . . . . 5 sq. yd.

Inclusive Prices: White or coloured cements stippled on in form of stiff paste . . . . 1*s*. per sq. yd.

Coloured cement renderings . . . . 3*s*. to 6*s*. per sq. yd.

CHAPTER IX

BRIDGES AND CULVERTS.

SECTION I.—BRIDGE CONSTRUCTION.

General Principles.

Many of the constructional operations dealt with elsewhere in this volume apply to the erection of bridges. Apart from the general application of the principles of concrete mixing, placing, and curing, bending and fixing reinforcement, and the construction and dismantling of shuttering, the reader is referred to the following items:

Inspection of site and arrangement of constructional equipment (Chapter I);

Construction of timber gantries, cofferdams, and piling and other foundations (Chapters I and VII);

Retaining walls and abutments (Chapter X);

Construction and handling of large pre-cast blocks (Chapter XI);

Finishes (Chapter VIII);

Testing completed structures (Chapter XI).

Reinforced concrete bridges may be grouped in two classes: (i) Slab and girder construction, and (ii) Arches. Simple slab, rigid-frame, and bowstring girders are variations of the first type which may also include steel girders carrying reinforced concrete deck slabs. Arches may have open or closed spandrels, either with an arch vault slab or with arch ribs carrying a slab. In an open-spandrel arch the deck is usually of beam-and-slab construction supported on columns carried on arch ribs.

The majority of surface treatments described in Chapter VIII are applicable to bridges, the more common finishes for such structures being renderings, cement paints, bush hammering, and applied masonry facings.

The natural limitations of the site and the maintenance of traffic during construction usually have an important influence on the method of construction.

Only typical methods can be described, although these may be capable of wide variation to suit circumstances.

Maintaining Traffic During Construction.

If the site of the new bridge is clear of the old structure the obvious course is to retain the latter in service until the new structure and approach roads are complete. If the new structure is to occupy the site of the original bridge but is to exceed it in width, the excess width can be built first, leaving the original bridge to carry traffic; the traffic can then be diverted to the completed part of the new structure while the old bridge is demolished and the new work completed. If the new and old bridges are of equal width a similar procedure can be adopted by demolishing half the old bridge first, maintaining traffic over the other

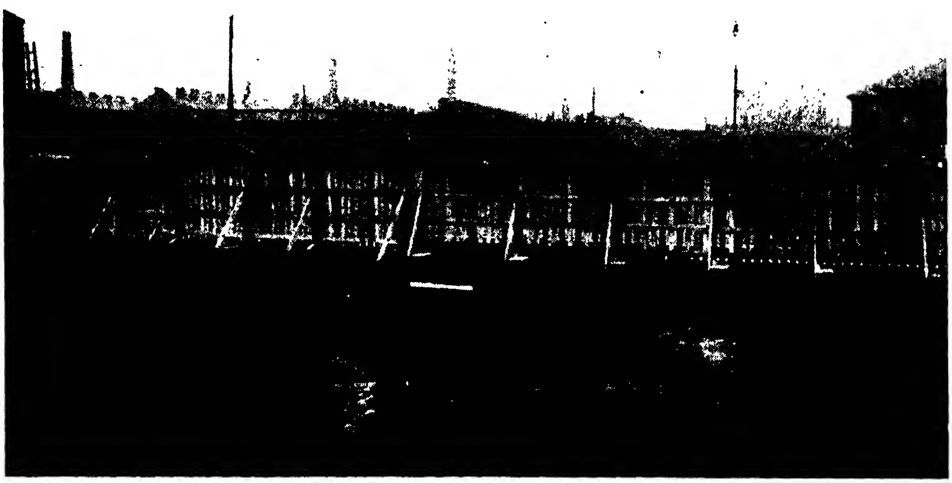


Fig. 309.—Temporary Timber Bridge.

half while constructing the first half of the new bridge. The traffic is then diverted to the latter, while the second half of the old bridge is demolished and the remainder of the new structure completed. Co-ordination between the engineers and the contractor is necessary to ensure that the design will permit the construction to be carried out in this manner.

When none of the foregoing methods can be adopted, it is necessary to erect a temporary bridge, either for foot traffic only or for vehicular traffic. If for the latter, the maximum weight of vehicles using the bridge may be limited and a speed limit may be enforced, thereby reducing to a minimum the loads and forces to which the temporary structure is subjected and enabling a light and economical structure to be erected. Temporary bridges are usually constructed in timber throughout, except over wide navigable rivers when the erection of a temporary steel bridge may be warranted.

A typical timber structure is illustrated in *Fig.* 309. Double rows of timber piles are driven at convenient positions for the piers, adjacent piles in each group being braced together with bolted connections. Cross-head pieces support main longitudinal runners, the latter spanning from pier to pier and carrying the timber decking forming the roadway. A parapet is provided along each side of the bridge, that adjacent to the new construction usually being close-boarded to prevent crowds gathering on the temporary bridge to watch the constructional work. The timbers should be of substantial size and should be designed in a similar manner to that described for timber gantries in Chapter I.

The principal constructional problem is the erection of the piers. In the case of slow-flowing streams it may be possible to drive piles from an anchored barge, but in the majority of cases it is necessary to mount the pile driver on staging. *Figs.* 310 and 311 show a method of driving light timber piles from a frame projecting in front of the staging which is being erected. The vertical guides for the winch-operated drop hammer are attached at the end of a cantilevered timber (12 in. square) as shown in *Fig.* 311, the timber being carried well back behind the last row of driven piles as shown in *Fig.* 310. The petrol-driven winch acts as a counterweight. The line of the hammer ropes and the guys to the head of the guides are also shown in the illustrations. In the case of a heavy temporary bridge, the piles capable of being driven by this equipment would be suitable only for the purpose of providing a staging upon which a larger pile frame could operate to drive the heavier and longer piles required to carry the bridge. In the case of light temporary bridges, such as foot-bridges, the equipment illustrated would be used to drive the piles forming part of the bridge.

Costs of Bridge Construction.

The wide variation of conditions in bridge construction is reflected in the wide differences in the costs. The basic cost of bridge work is generally considered as the cost per square yard of deck, and this tends to rise with increase in span. The provision of a temporary bridge, the depth to which foundations have to be taken, and the loading for which the structure has to be designed, will have considerable effect on the cost. Some representative costs for reinforced concrete bridges are as follows.

Type	Number of spans	Span	Approximate cost per square yard	Year built	Remarks
			£		
Girder	Single	24 ft.	22	1935	—
Arch	Single	60 ,,	29	1926	On piles
Arch	Single	100 ,,	17	1935	—
Arch	Single	100 ,,	11	1935	On piles
Bowstring	Single	117 ,,	9	1934	On piles
Girder	Three	20 ft. to 30 ft.	8	1934	On piles
Girder	Three	24 ft. to 50 ft.	3	1934	On piles
Girder	Three	75 ft.	15	1935	—
Arch	Three	60 ft. to 125 ft.	15	1929	—
Girder	Five	42 ft.	9	1926	On piles
Girder	Five	50 ,,	5	1934	On piles
Girder	Nine	35 ,,	13	1935	—
Girder	Several	18 ,,	10	1934	On piles

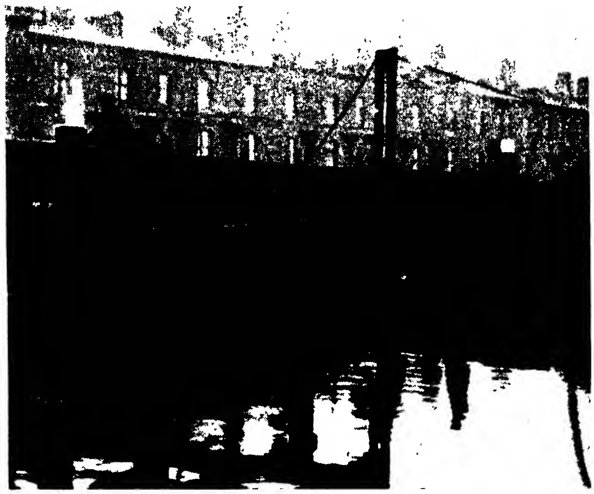


Fig. 310.—Driving Piles for Temporary Bridge.

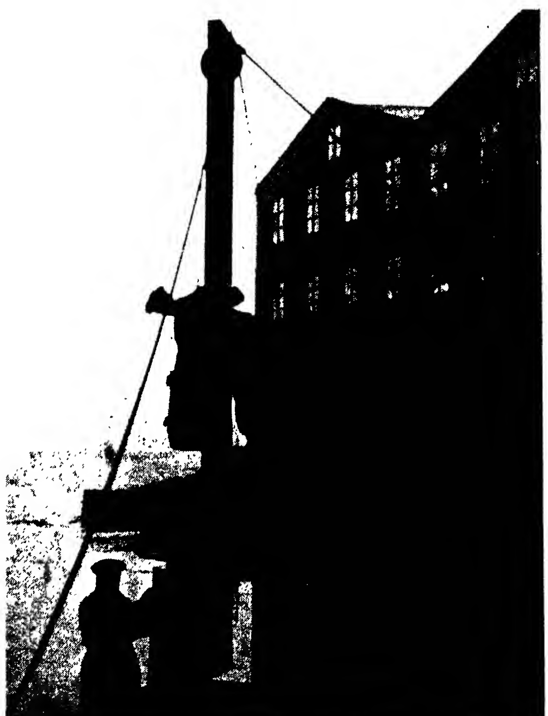


Fig. 311.—Driving Piles for Temporary Bridge.

Fluctuations are also encountered in the relative costs of the various operations involved in bridge construction. The following percentages of the total cost are taken from recent structures.

Type of bridge	Girder	Girder	Closed-spandrel arch	Arch
Number of spans	Five	Three	Single	Three
Total length	190 ft.	76 ft.	—	253 ft.
Maximum span	92 ft.	31 ft.	60 ft.	125 ft.
Foundations	Piled	Piled	Piled	Piled
Date	1926	1934	1926	1929
Preliminary items	7 per cent.	7 per cent.	6 per cent.	2 per cent.
Dams, pumping, piles, abutments, piers, etc.	34 per cent.	56 per cent.	44 per cent.	46 per cent.
Deck construction including arches, spandrel walls, columns, beams, slabs, etc.	47 per cent.	27 per cent.	41 per cent.	42 per cent.
Parapets	5 per cent.	5 per cent.	4 per cent.	3 per cent.
Surface finish	2 per cent.	2 per cent.	—	2 per cent.
Extras	5 per cent.	3 per cent.	5 per cent.	5 per cent.

SECTION II.—GIRDER BRIDGES.

Supporting Deck Construction.

One of the main problems in bridge construction is making provision for the temporary support of the superstructure at points other than the positions of the permanent piers. In the case of a bridge over a river this involves consideration of the depth of water, the nature of the river-bed, and the necessity of maintaining a clear waterway. For bridges over railways the maintenance of headroom is often the controlling factor.

In all cases where the depth between the deck construction and a satisfactory foundation is moderate it is preferable to provide as many vertical supports as possible, thereby transmitting the weight of the shuttering, wet concrete, and other working loads in the most direct manner to the ground. Where the foundation level is below water, and especially where the nature of the river-bed necessitates piling to carry even the loads of the temporary centering, it is usual to limit the number of points of support and to carry the centering on an assembly of inclined props radiating fanwise from supporting points as in *Fig.* 312. These inclined supports should be as nearly vertical as possible and in no case more than 45 deg. out of the vertical. *Fig.* 312 shows the application of this method to the construction of a girder bridge over a river where it is not essential to preserve a clear waterway. Where a clearance has to be maintained an arrangement of steel joists requiring a minimum of intermediate supports is more suitable, since the working space between the loading gauge and the bridge soffit is usually considerably restricted.

For each structure of any type it is generally necessary to design in complete detail a system of supports that can be erected and dismantled without interfering with the traffic. The following examples illustrate how these difficulties have been overcome in a number of typical cases. In later paragraphs further detailed consideration will be given to the support of centering, and the remarks on timber staging given in Chapter I should also be referred to in this connection.

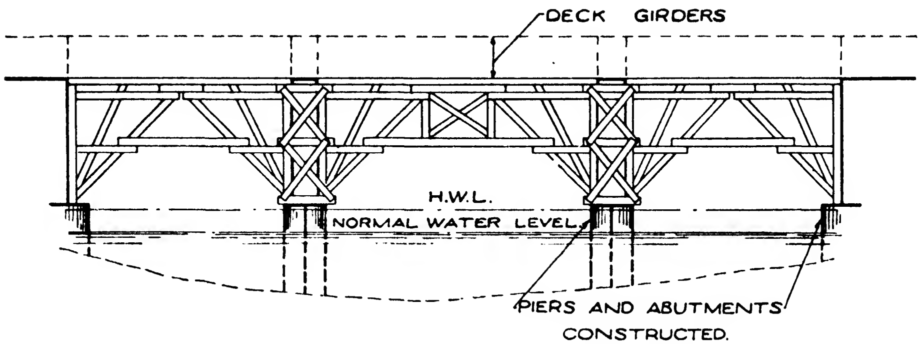


FIG. 312.—SUPPORTING DECK CENTERING WITHOUT MAINTAINING WATERWAY—LARGE SPAN.

Fig. 313 shows the support for a small-span bridge where the water is shallow and a hard bottom is available. The principal longitudinal members are 9-in. by 3-in. or similar timbers (or they may be steel joists) and are stiffened by the inclined corner braces and the central prop. The ends of the longitudinals are

FIG. 313.—SUPPORTING DECK CENTERING WITHOUT MAINTAINING WATERWAY.

supported on vertical posts. Transverse timber bearers laid on edge directly support the sheeting for the underside of the road slab.

In the case of the bridge shown in *Figs.* 314 and 315, a clear waterway had to be maintained and the available headroom prohibited the use of any supports below the soffit of the deck. The whole of the shuttering was therefore suspended from a steel joist acting as the bottom member with timbers completing the triangular truss. The mid-point of the steel joist was suspended by a pair of

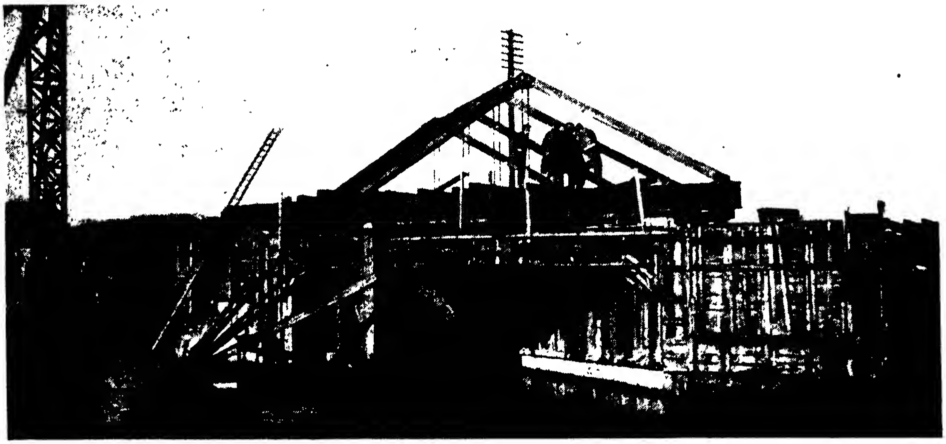


Fig. 314.—Supporting Deck Centering while Maintaining Waterway.

tie rods attached to the apex of the truss, and the ends of the joist rested on the abutments of the new bridge. As shown in *Fig.* 315, transverse timbers carried

Fig. 315.—Supporting Deck Centering while Maintaining Waterway.

on the top flanges of the joists supported the suspension rods from which the shuttering was hung.

FIG. 316.—SUPPORTING DECK SHUTTERING OVER RAILWAY.

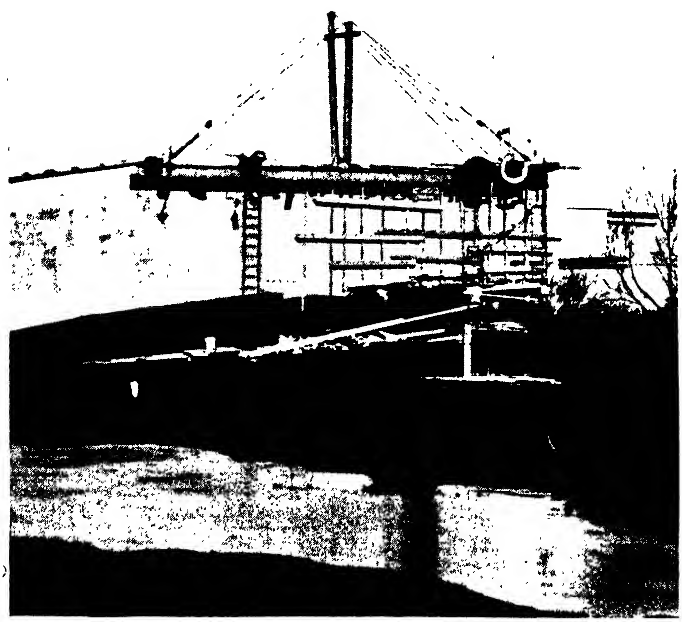


FIG. 317.—CANTILEVERED CONSTRUCTION.

A similar problem was encountered in the case of the bridge over the railway illustrated in *Fig.* 316. In this instance timber trestles were built on each side of the tracks to support the ends of a series of steel joists. Sufficient joists were provided to limit the load on each so that intermediate supports were unnecessary (see page 363). These joists carried the timber sheeting for the deck soffit.

In constructing light wide spans over rivers where it is advisable to avoid temporary works carried on the river-bed, and where headroom is insufficient to allow enough depth for a temporary supporting girder spanning between two piers, a method of cantilevered construction can be adopted. In the example shown in *Fig.* 317 the piers and end spans have been erected in the normal manner and the river span is in course of construction by cantilevering successive 5-ft. sections outwards from the completed piers on either side of the river. The apparatus required consists essentially of a horizontal frame built up from pairs of 12-in. longitudinal channels with transverse channel and angle bracing. This frame is carried on six wheels running on rails laid on the completed portion on the work to which the frame is anchored by vertical tie rods attached to horizontal bolts passing through holes cast in the webs of the girders. The overhanging portion is stiffened by chains and turnbuckles passing over vertical posts. The shuttering for the sides and soffits for a 5-ft. length of the girders is suspended from the frame, access being obtained by means of ladders also suspended from the frames. When the successive 5-ft. sections from both sides approach the centre of the span, the central closure section is cast, and the deck slab is cast after the two main girders are completed, this procedure being adopted to minimise the load on the temporary cantilevers. In adopting such a method of construction it is imperative for the contractor to collaborate closely with the engineer.

Construction with Pre-cast Members.

When constructing bridges over railways, or in similar cases where maintenance of traffic imposes restriction on headroom and clearances and limits the period available for working, pre-cast concrete is often used for the principal members of the bridge. In this way the beams can be cast under favourable conditions adjacent to the site and placed in position after the erection of the abutments and piers. This method reduces to a minimum the time during which work is being carried out over the railway tracks. Apart from the necessary co-operation between the railway company and the contractors, there must be collaboration with the designing engineers to ensure that the design allows for the separate construction of the beams, and for the stresses produced during handling operations. The age of the pre-cast members must be such that the concrete shall have matured sufficiently to withstand handling. The weights of single members must not exceed the lifting capacity of the cranes.

An example of pre-cast construction is illustrated in *Fig.* 318 in which the piers and abutments have been constructed and the seatings formed for the main beams. The latter are brought up to the piers on the bogies of jubilee wagons from which they are lifted by a loco crane and placed in position on the piers. If chain slings are used the concrete should be protected by sacking to prevent spalling. Timber packings between the chain sling and the concrete are seen in *Fig.* 319, where a 36-ton railway crane is handling a 9-ton per-cast beam by

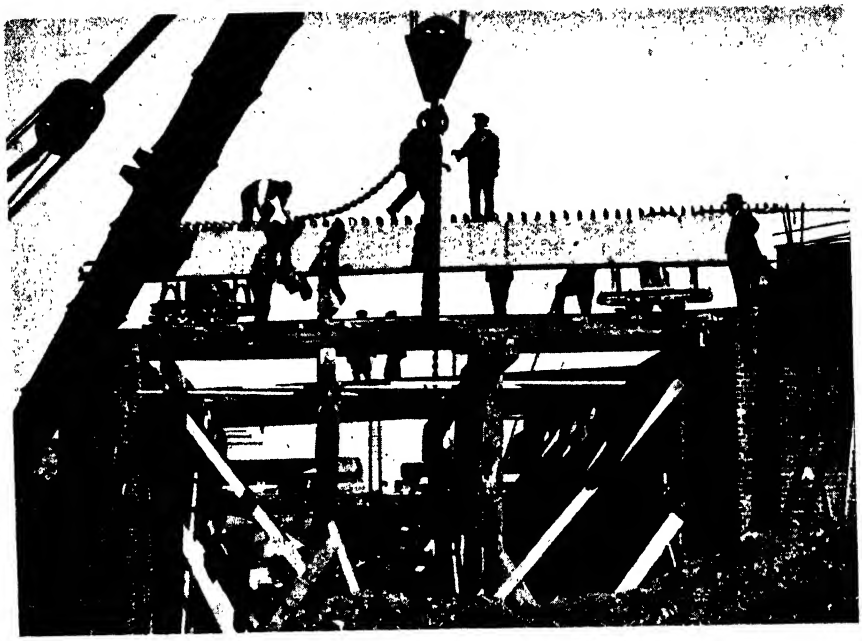


FIG. 318.—PRE-CAST BEAM CONSTRUCTION.

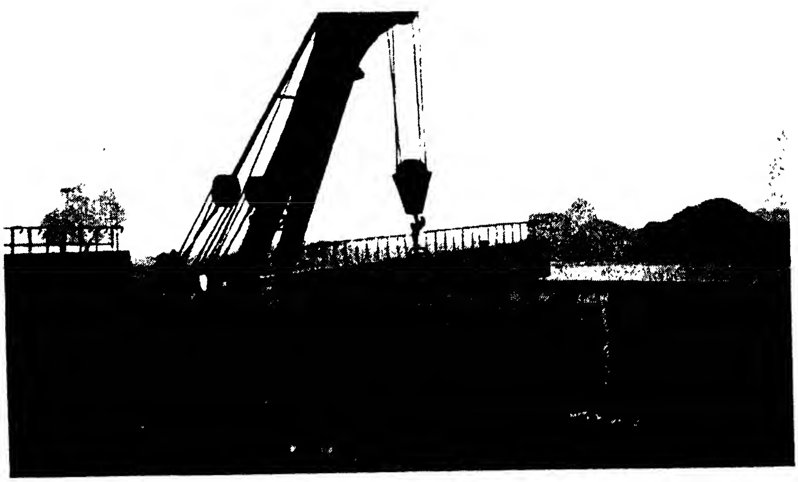


FIG. 319.—PLACING PRE-CAST BEAM IN POSITION.

centre point suspension. In *Fig.* 320, however, the crane sling is attached to the ends of the pre-cast beam, which is being slung into position with timber cleats already bolted to the sides for supporting the deck shuttering.

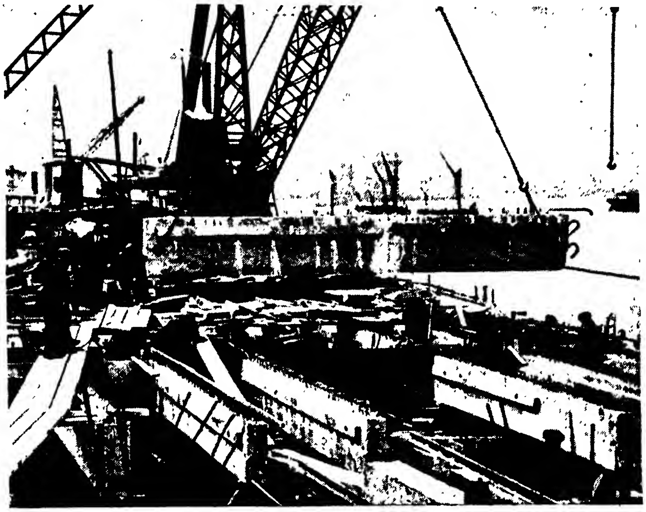


FIG. 320.—TWO-POINT SUSPENSION OF PRE-CAST BEAMS.

It is usual to cast horizontal ferrules through the beams to facilitate attachment and removal of the timbers bolted to the sides of the beams in place, from which the joists and sheeting for the slab shuttering are supported as shown in

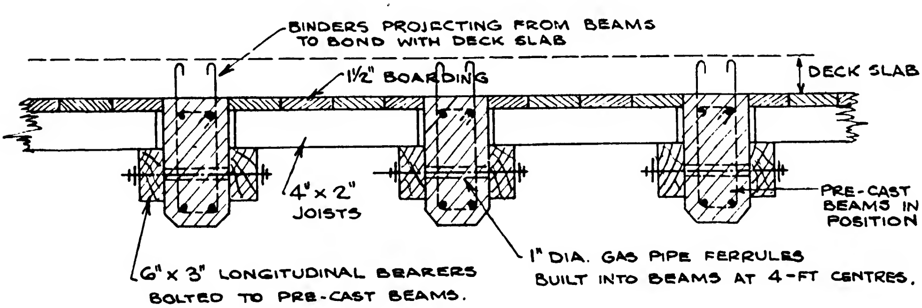


FIG. 321.—DECK SHUTTERING WITH PRE-CAST BEAMS.

Fig. 321. In this way it can be arranged that the deck falsework and its supports do not project below the soffit of the pre-cast beams and obstruct the clearance under the bridge.

Bowstring Girders.

Reinforced concrete bowstring girders are used for large spans where headroom is restricted and suitable abutments for ordinary arches are unobtainable

The hangers as well as the deck and arch boom are usually constructed in concrete. The centering and shuttering for the deck are constructed similarly to that for ordinary bridges, while one of the methods of arch shuttering described in subsequent paragraphs would be used for the top boom of the girder. The principal centering under the arch would be modified, since the boxing of the ribs can be supported directly off the deck construction; props are maintained under the latter until the concrete arch has matured. An illustration of the centering and shuttering for a bowstring girder is given in *Fig.* 322.

The position of construction joints and the sequence recommended for concreting the deck slab are given in *Fig.* 323. The first operation is to concrete a portion (1) of one end of a main deck-beam, the adjacent area of deck slab

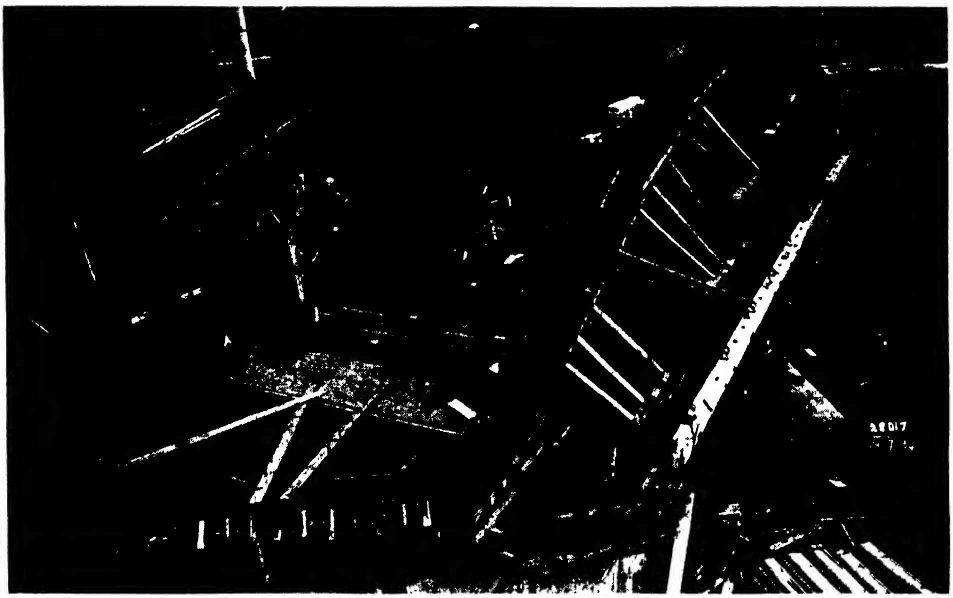


FIG. 322.—CONSTRUCTION OF BOWSTRING GIRDERS.

and transverse beam, and the springing of the corresponding arch rib. The corresponding section (2) at the end of the other main beam is then constructed, and the work then proceeds along the bridge, concreting panels in the rotation indicated by the figures (2) to (8). The transverse joints should be made along the centre line of the panels of the decking slab, but in some instances these joints have been made by leaving a gap of about 5 ft. along one edge of a transverse beam in each bay. These gaps are concreted later when the bridge is subjected to the full dead load.

When the deck slab has been completed the arch ribs, sections 9 and 10, can be concreted, concrete placing proceeding symmetrically on one girder at a time working from the springings and leaving a shrinkage gap of 2 ft. or 3 ft. at the crown. When both girders, with the exception of the shrinkage gaps, are complete, the transverse struts (11) can be concreted, and when the concrete in

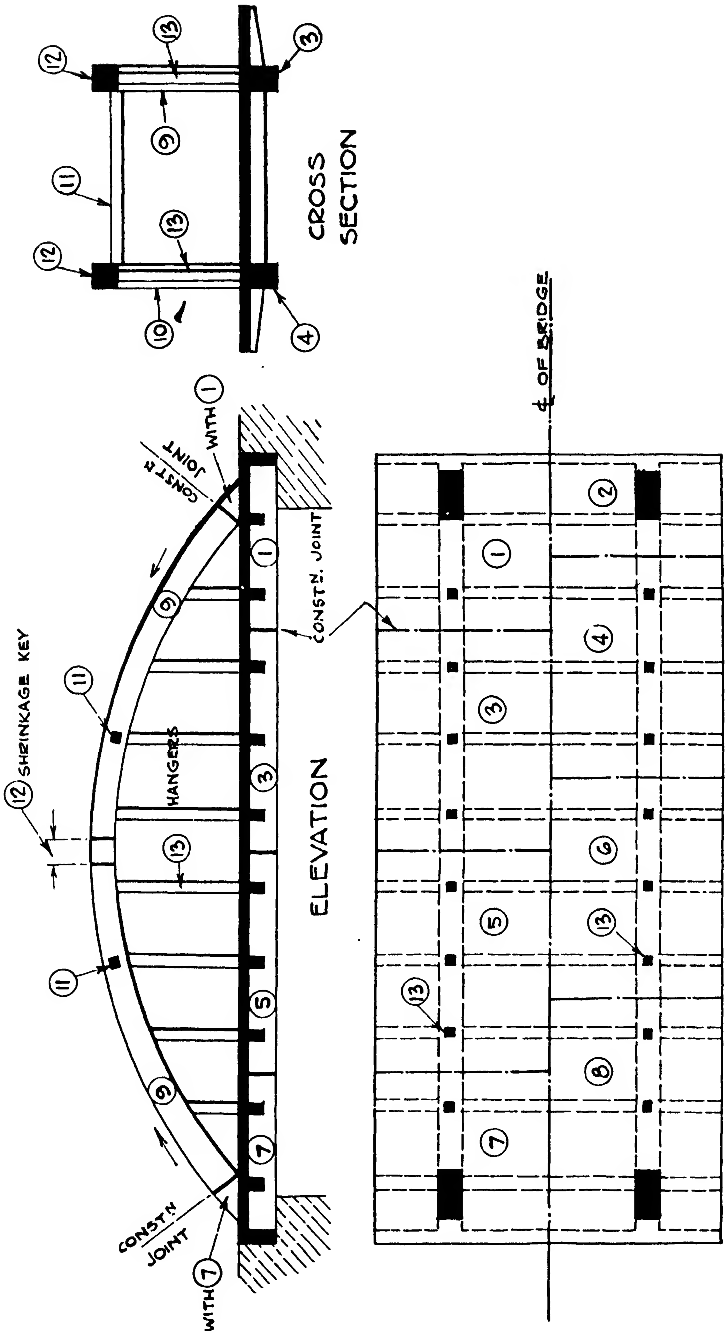


FIG. 323.—CONSTRUCTION JOINTS IN BOWSTRING GIRDERS.

the arch has matured the shrinkage keys (12) can be concreted. The hangers (13) should not be concreted until the full dead load has been imposed on the bridge. The dead load should be temporarily augmented to its permanent value if any operations, such as surfacing the roadway, are not complete when the hangers are to be concreted.

SECTION III.—ARCH BRIDGE CONSTRUCTION.

Centering for Arches.

The principles underlying the construction of centering for arch construction are (i) absolute rigidity, (ii) ease of removal, (iii) conformity to traffic requirements, and (iv) economy. Since a considerable proportion of the cost of an arch is due to the falsework, rigidity and economy are better assured if the centering and shuttering are properly designed. The main supporting members, sole-plates, and piles (if any) should be designed to carry the estimated weight of the falsework and the weight of the wet concrete and any incidental loading that may be present during construction. The safe loading on timber posts, beams and piles has been considered in Chapter I. The sheeting of the soffit of the arch and the members supporting it and in direct contact with the concrete are designed in accordance with the method given in Chapter IV. All posts and struts should be well braced, and unless calculations are made to cover cases of slender columns it is advisable, if the struts are to carry their full load, to arrange the bracing so that no piece has an unbraced length of more than fifteen times its width. Thus, with a 6-in. by 6-in. post, braces should be about 7 ft. apart in both directions; with a 9-in. by 3-in. post the braces in the 9-in. direction should be at 10-ft. centres or less, while those in the 3-in. direction must not be more than 3 ft. 6 in. apart.

Centering composed of steel scaffolding or steel trusses and girders is frequently used and is economical when wide waterways or other clearances have to be spanned.

The effect of wind should be allowed for in the design of the main centering, especially for high narrow bridges in exposed positions.

The principal supporting members of arch centering follow one of three principal arrangements, but numerous variations of these methods occur in practice dependent on site conditions and requirements. The diagrams in *Fig.* 324 illustrate the primary arrangements.

Type (*a*) is suitable when little clearance is required and where the ground beneath the arch is capable of providing a satisfactory foundation for the posts of a continuous trestle. The latter is usually in timber, but tubular steel scaffolding may be used for this purpose. An example of a timber trestle is illustrated in *Fig.* 325.

Suitable sizes for timber centering of this design are 6-in. by 6-in. or 8-in. by 8-in. Oregon pine posts in 12-ft. lengths, the posts and horizontal braces being arranged to form panels about 10 ft. square. Each panel is doubly cross-braced with 6-in. by 2-in., 8-in. by 3-in., or 9-in. by 3-in. white pine braces, the transverse and longitudinal braces being either 6-in. square or 9-in. by 3-in. Although

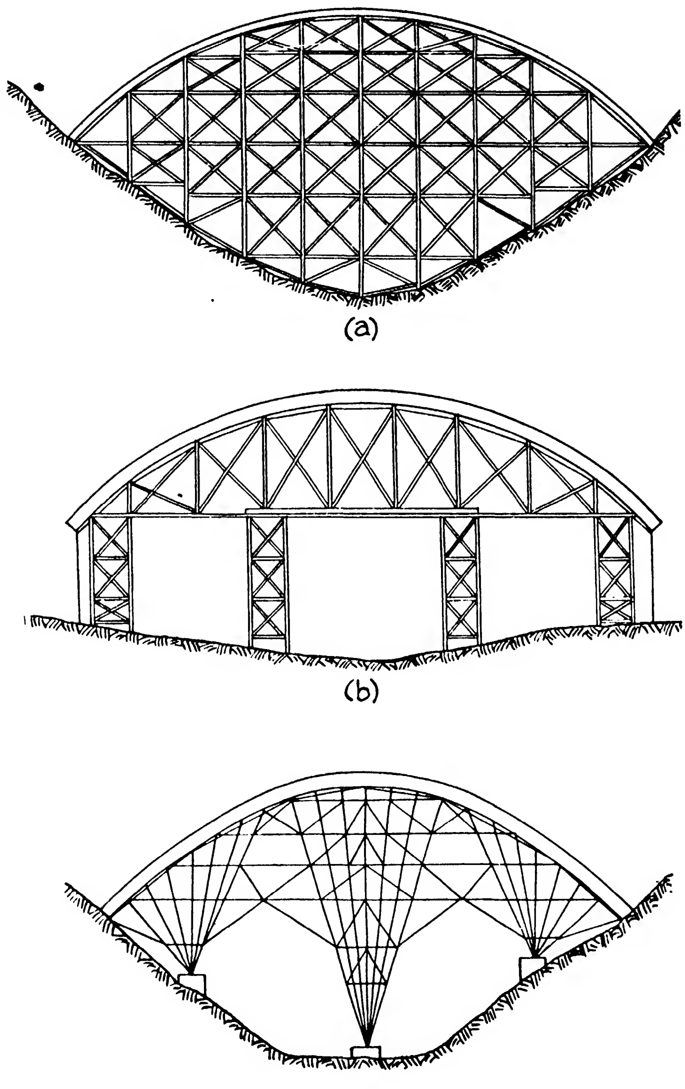


FIG. 324.—TYPICAL ARRANGEMENT OF CENTERING.

all connections should be bolted where practicable, economy of material and time have been claimed for the use of nailed joints throughout.

When the arch spans over a wide waterway type (*b*) is more suitable; type (*c*) is used when large spans extend over deep valleys. Type (*b*) consists of a support at each end of the span with one or more intermediate piers supporting longitudinal members which, in turn, carry the braced posts supporting the shuttering. In type (*c*) a minimum number of principal supports is founded on the bottom of the valley or in the river bed, and struts are spread out fanwise to produce a series of close supports for the arch soffit. An illustration of this type of centering is given in *Fig.* 326.

Details of the treatment of the feet of posts or struts for a trestle of type (*a*) are given in *Fig.* 327. When the bottom of the valley is firm ground, the post can be anchored into a block of concrete as in (i). Excavation is carried down to firm ground and levelled off, the level being low enough to prevent lateral

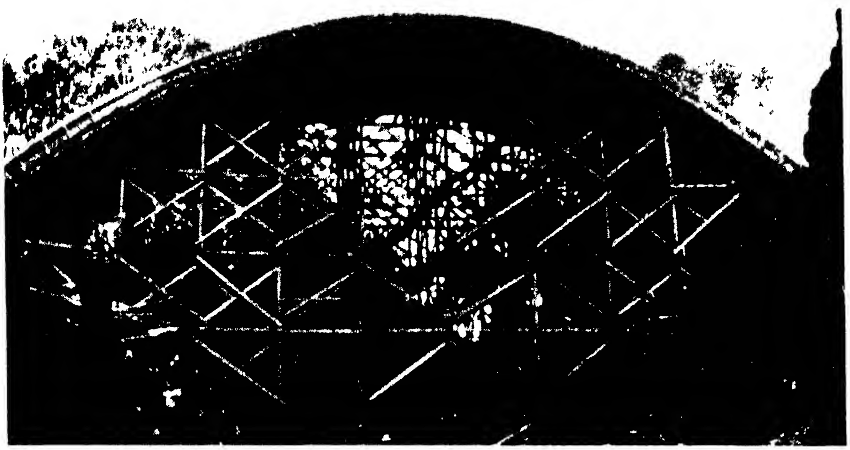


FIG. 325.—TRESTLE CENTERING.

slipping when the natural slope of the ground is steep. When the post bears on a rocky surface, a mild steel dowel-rod driven into the rock and penetrating the foot of the post, as in (ii), can be used to secure the foot of the post.

The construction of a pier for centering arranged as in *Fig.* 324(*b*) is most conveniently effected by driving three, four, or more timber piles into the river bed and bracing the piles together above low-water level. When the bed is rocky, or so firm that pile driving is impracticable or unnecessary, the method of construction indicated in *Fig.* 328 can be used. A mass concrete block is cast on the bed of the river within a timber box or within a crib formed of old rails and lined with rough boarding. Precautions must be taken to ensure that the stability of the under-water work is not impaired by scouring; thus bases should be carried through any mud and well into the firm stratum and stone pitching should be laid on the river-bed around the base. The pier is constructed of four timber posts braced together as shown, a cross-head and bearers being attached to the heads of the posts to carry the longitudinal joists or beams

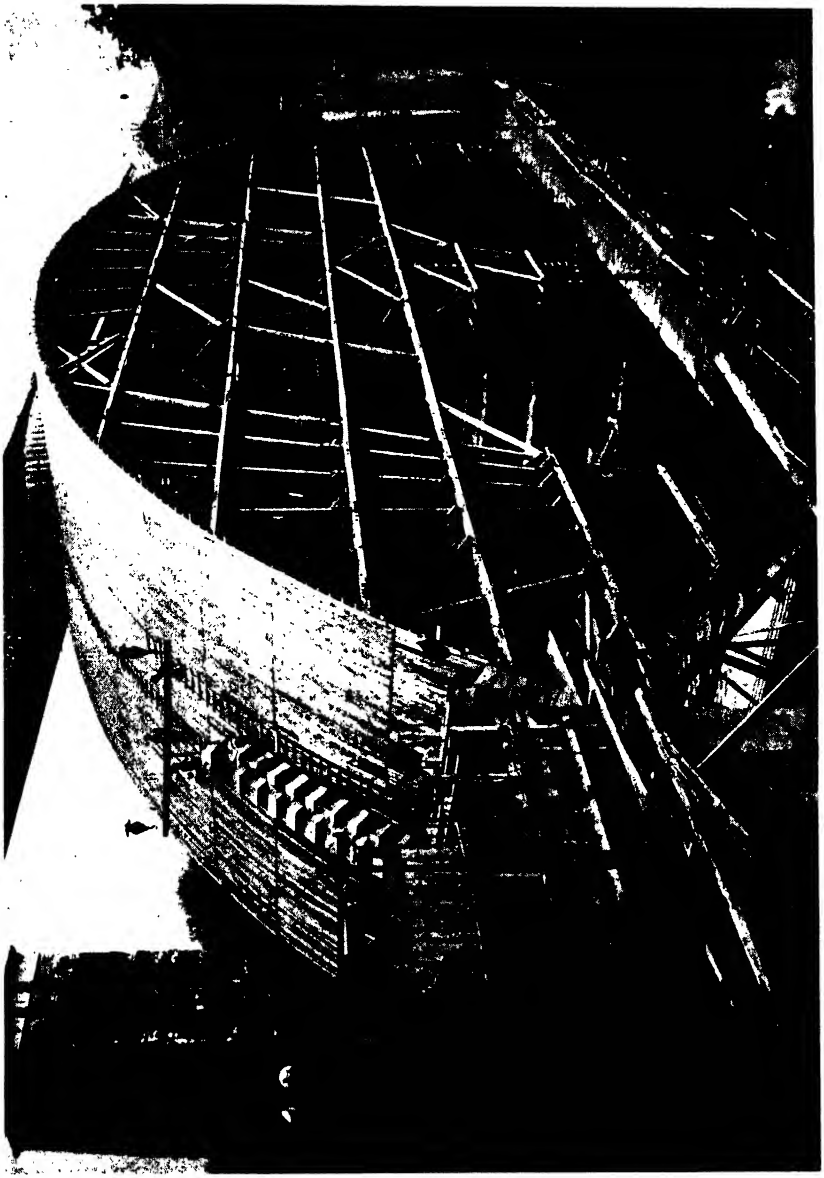
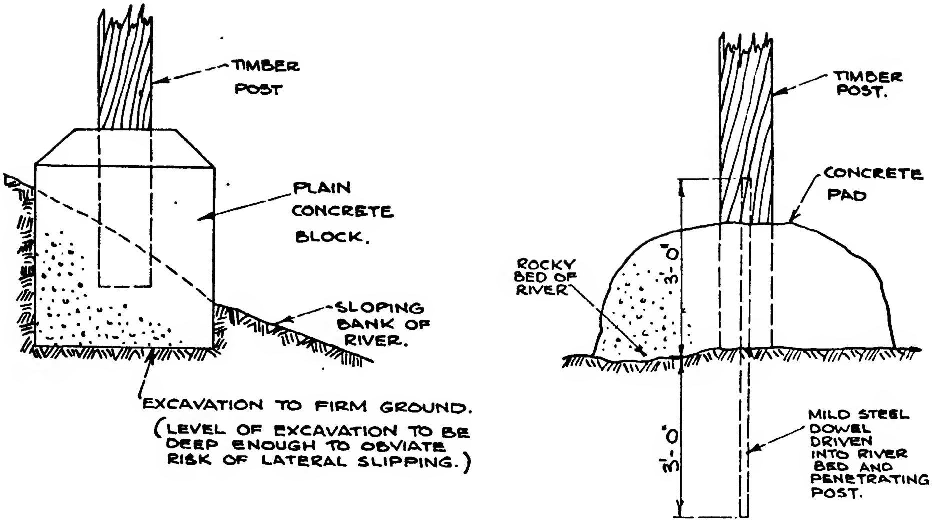
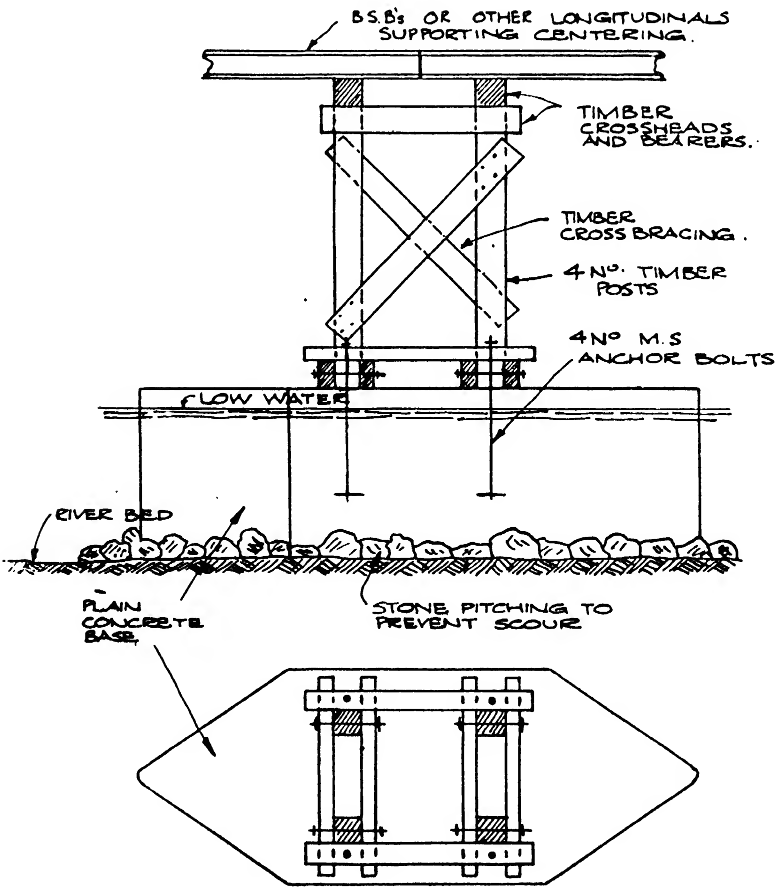


Fig. 326.—Radial Struts for Arch Centering.



(i) Fig. 327.—Foundations for Trestles. (ii)



supporting the centering. A framework of timbers attached to the foot of the posts is secured to the concrete block by four mild steel anchor bolts.

A concrete block can also be used for the principal support of centering arranged as in *Fig.* 324(*c*), or if piling is necessary a foundation can be constructed as illustrated in *Fig.* 329. A reinforced concrete platform is supported on four

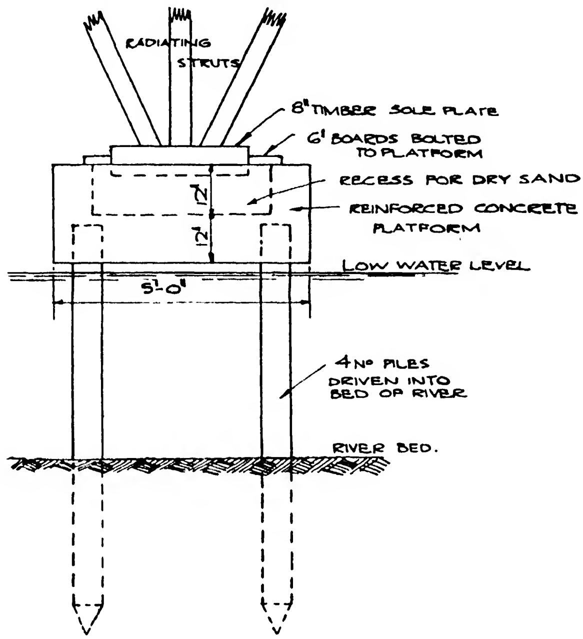


FIG. 329.—PILED FOUNDATION FOR SUPPORT OF CENTERING.

timber piles, which may be undressed tree-trunks if available. In the example illustrated the foundation is arranged to support a series of radiating struts. A recess in the platform contains dry sand upon which bears a timber sole-plate, the struts being secured to the latter. Until the centering is about to be dismantled the sand is prevented from escaping by covering the space around the sole-plate with boarding bolted firmly to the concrete block.

Variations of the typical arrangement of centering shown in *Fig.* 324 are illustrated in *Fig.* 330. The design at (*a*) shows a timber trestle suitable for a bridge of several spans, the piles being 12-in. by 12-in. spaced about 15 ft. apart in each direction or to suit the spacing of the piers and width of the bridge. In this form of construction little waterway is possible, and it is therefore suitable only for non-navigable rivers where flood debris is not anticipated. The piles are capped by 12-in. square transverse sill beams placed above normal flood level and braced longitudinally and transversely as illustrated. Longitudinal bearers are carried on the sills to form a rigid gantry, which would, in the first place, be used to carry cranes and other equipment required for constructing cofferdams, piling, blockwork, etc. When the piers are completed and the cofferdams removed, the gantry is used to support the arch centering.

The arrangement shown in *Fig.* 330(*b*) is more suitable for big spans over

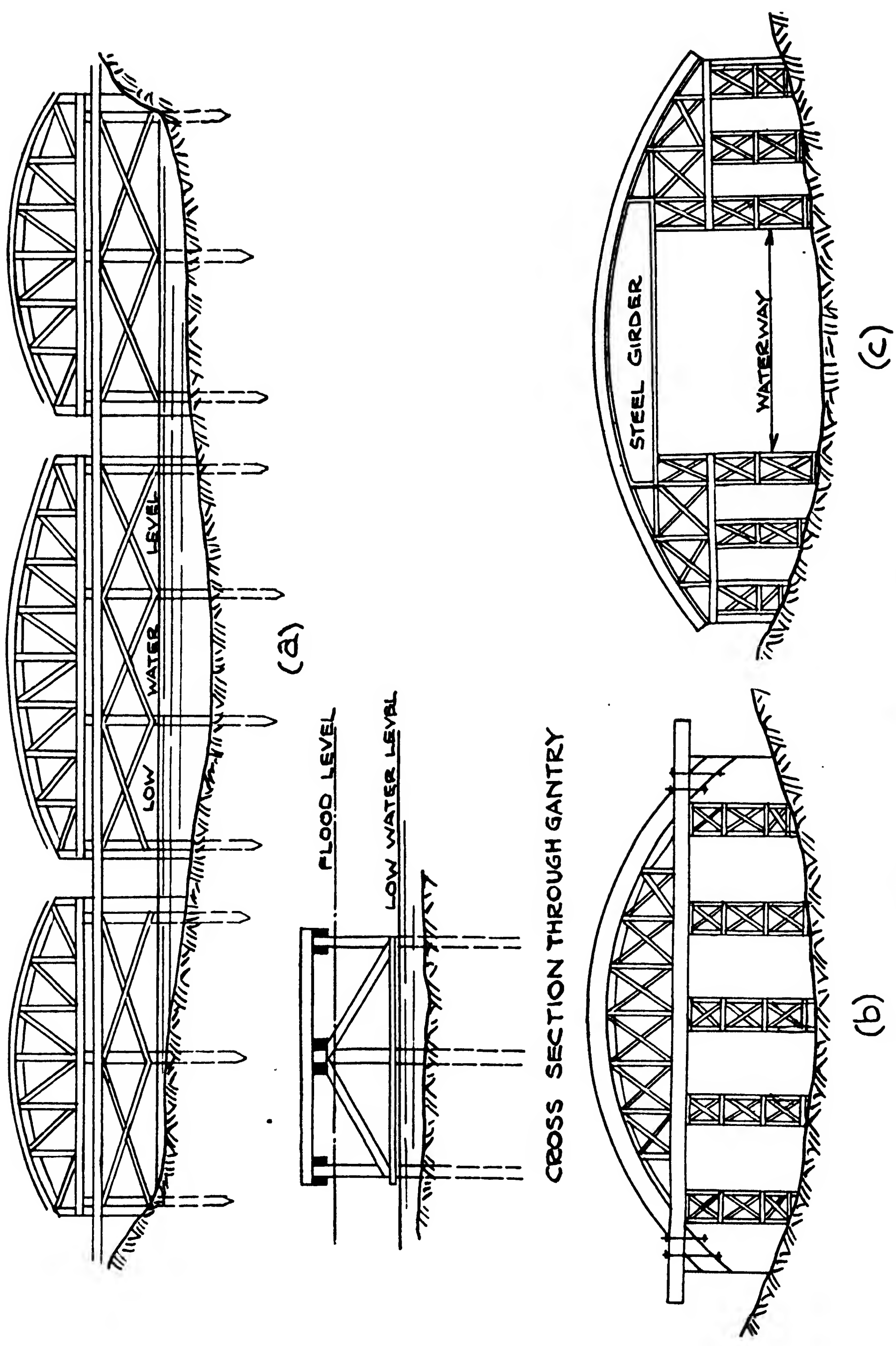


FIG. 330.—VARIATIONS OF TYPICAL ARRANGEMENT OF CENTERING.

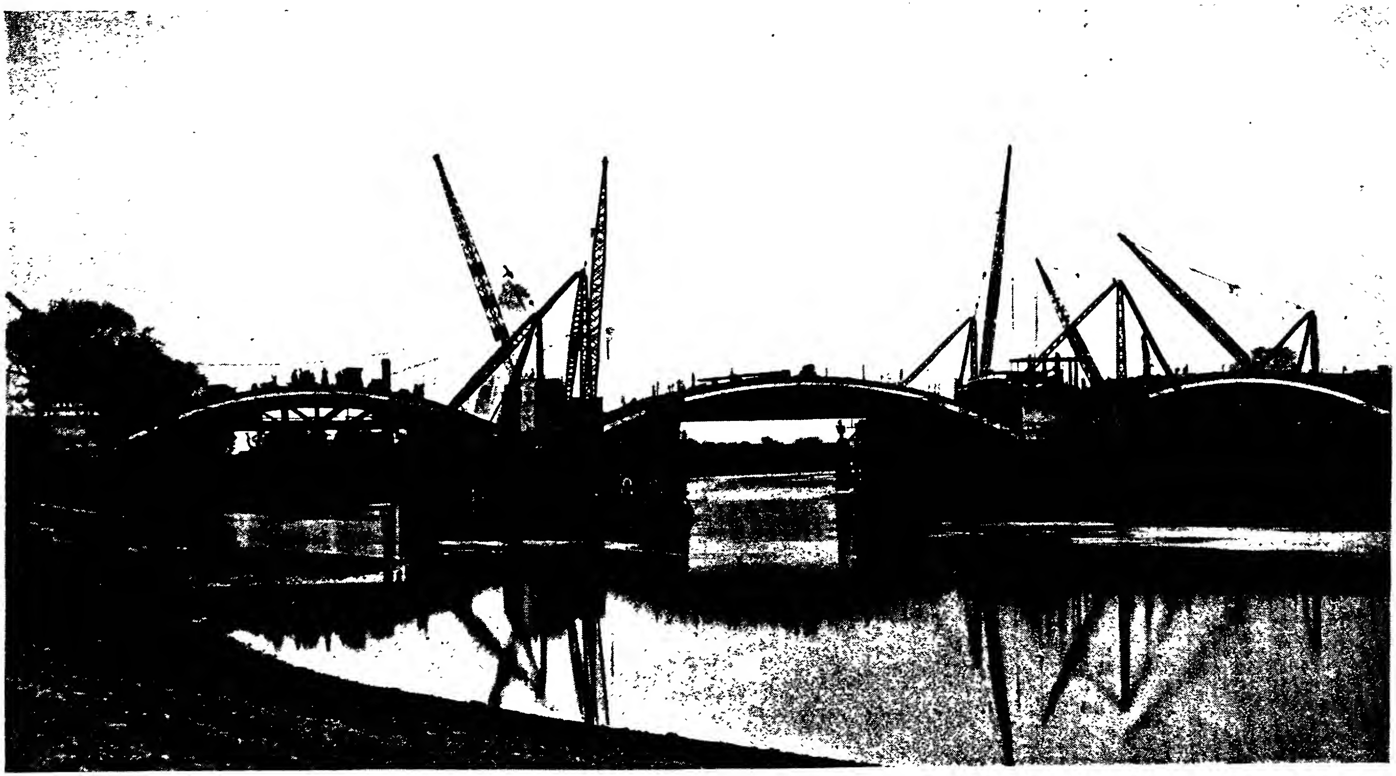


Fig. 331.—Steel Girder Supporting Arch.

rocky river bottoms where piling is impossible, or when a navigable waterway has to be maintained. The temporary piers can be composed of two rows of 12-in. by 12-in. vertical posts braced together and founded on a block of concrete similar to that illustrated in *Fig.* 327. The head pieces of these piers carry longitudinal girders—a single timber in the case of small spans, or, for large spans, a built-up timber truss, a steel joist, a steel truss, or a plate girder, upon which the main centering is supported. For arch ribs where the springing level is low it is often convenient to suspend the shuttering for the end portion of the ribs as in *Fig.* 330(*b*).

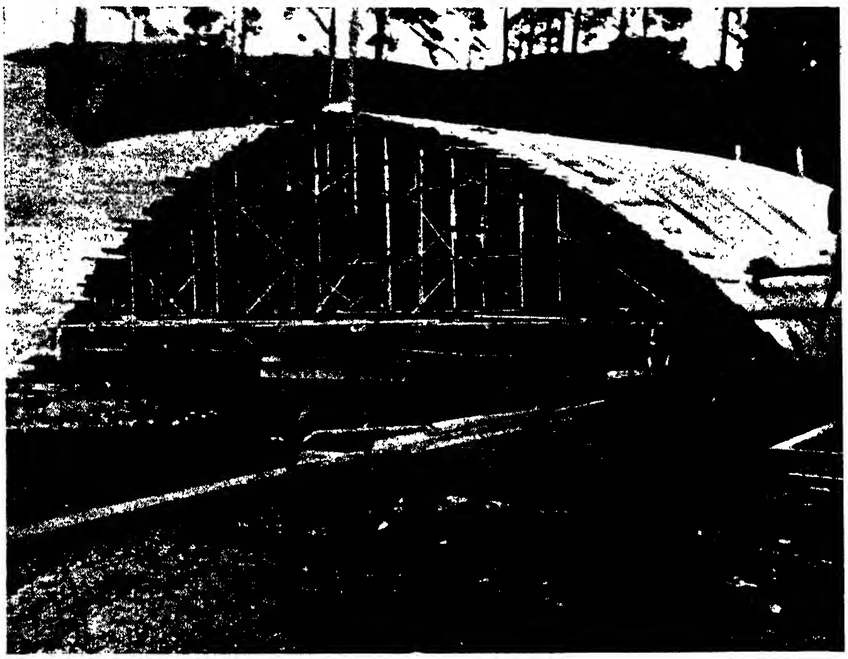


FIG. 332.—STEEL SCAFFOLDING SUPPORTING CENTERING.

Where a considerable width of waterway has to be maintained in a long-span bridge, the ends of the spans may be supported on a timber piled trestle, while a steel girder shaped to suit the arch soffit spans the centre opening as shown in *Fig.* 330(*c*). An illustration of the application of this latter method is given in *Fig.* 331. An example of a 77 ft. span for which the arch centering is supported on steel scaffolding is illustrated in *Fig.* 332. In this instance the verticals are fixed to adjustable jacks which in turn are carried on timber piles.

Shuttering for Arches.

Supported from the main stagings are the templates or profiles carrying, in the case of arch slabs, the sheeting to the soffit, or in the case of independent arch ribs the boxing for the ribs. As described for roof frames in Chapter VIII, the usual arrangement is to form the template from longitudinal boards laid on edge as shown in *Fig.* 333, the upper edge being cut to the required profile, making due allowance for the combined thickness of the transverse bearers and sheeting. If the template joists are placed sufficiently close together the sheeting can be laid directly on the templates, but if the templates are spaced too far apart for the thickness of the boarding the sheeting would be carried on bearers spanning between the templates. In the construction of some large spans, steel girders with the upper booms accurately curved to the arch profile have been used in place of built-up timber templates. An interesting adaptation of steel joists as templates is shown in *Fig.* 334, the joists being bent to radius and supported on piled trestles. Although the bridge crosses the river on the skew, the joists

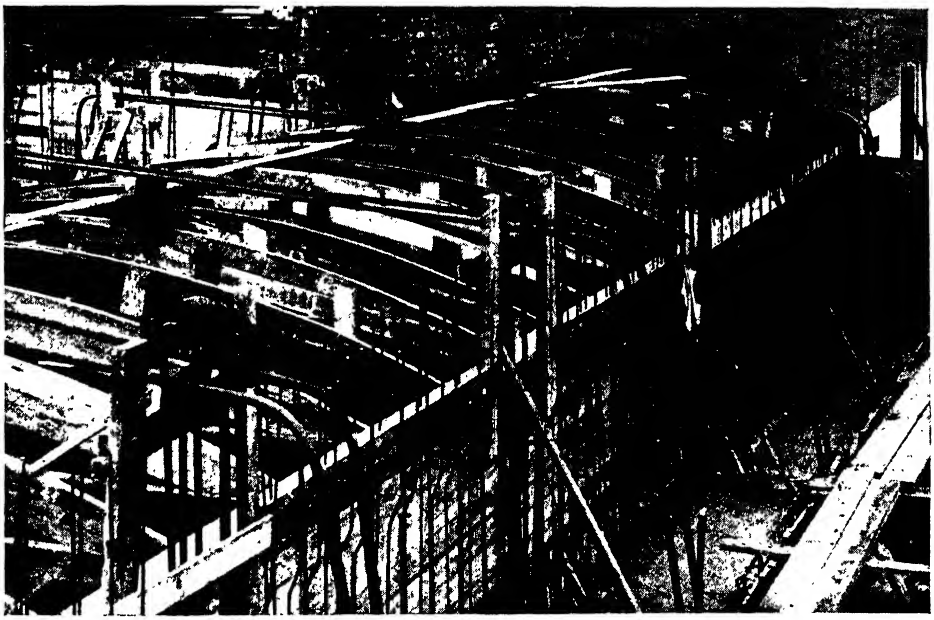


FIG. 333.—TEMPLATES FOR ARCHED SLAB.

span squarely across and give a clearer navigation channel than would timber centering. The sheeting, composed of 3-in. by 2-in. timber lined with plywood, is laid on the joists. Various details of arch centering and shuttering are given in *Figs.* 335, 336, 337, and 338.

The design shown in *Fig.* 335 is suitable for combination with the general arrangement shown in *Fig.* 324(*a*) and (*b*) or *Fig.* 330(*a*) and (*b*). Vertical posts, wedged off the main longitudinal bearers, are braced together diagonally and horizontally, suitable sizes being given in the diagram. Cross-heads bolted across the heads of the posts support the shaped ends of the templates. The template boards are, in this example, in short lengths between the cross-heads. Where adjacent templates overlap they are bolted together. If the bridge is an arch-slab design bearers laid on the flat span between the templates, and attached to these bearers are the sheeting boards laid longitudinally and conforming to the curve of the soffit.

Sheeting, although necessarily substantial, does not usually exceed 2 in. in thickness, as it must have sufficient spring to conform to the soffit profile when laid longitudinally. When a good surface to the soffit is particularly required, the sheeting can be covered with plywood or other smooth lining, in which case wrought boarding is not necessary and adjacent boards need not be butted together.

An alternative detail for an arch slab is shown in the cross-section of the shuttering illustrated in *Fig.* 336. Here the principal longitudinal bearers are steel joists connected by bolts and spreaders and supporting a timber centering

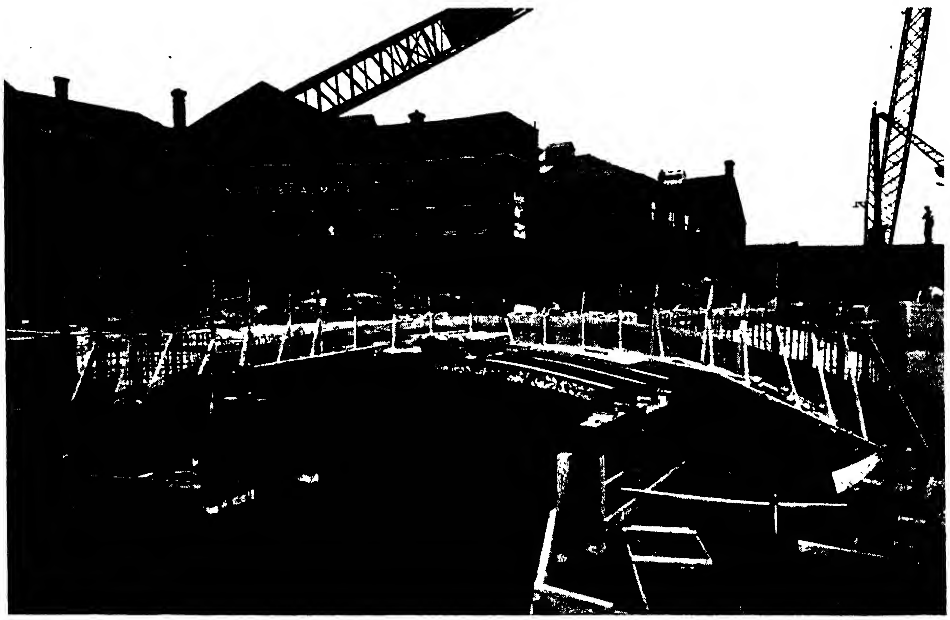


FIG. 334.—STEEL JOIST TEMPLATES.

similar to that in *Fig.* 335, except that the posts are supported on heavy transverse bearers carried on the steel joists. The wedges in this case are directly under the templates, the latter consisting of pairs of timbers bolted together and extending the full span of the bridge in a unit. Since the templates are closely spaced the sheeting spans transversely, an overhang at the outer edge of the arch providing a support for blocks maintaining the facing board in position. When the bridge is of arch-rib design with the ribs closely spaced, a completely sheeted soffit as for an arch slab is sometimes provided, and upon this surface the boxing for the sides of the ribs is erected. Usually, if the ribs are spaced widely apart, the templates are arranged in pairs under each rib as in *Fig.* 337. Bearers placed on edge carry the curved bottom boards, and the side shuttering is kept in position by inclined struts or by props from adjacent boxes. Occasional

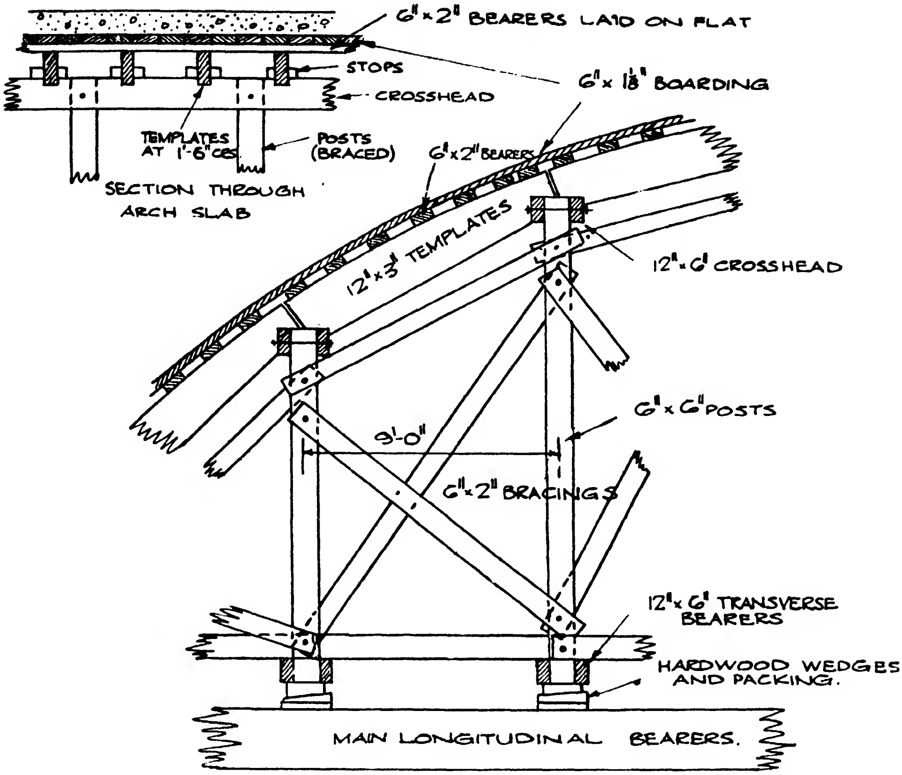


FIG. 335.—SHUTTERING FOR ARCHES.

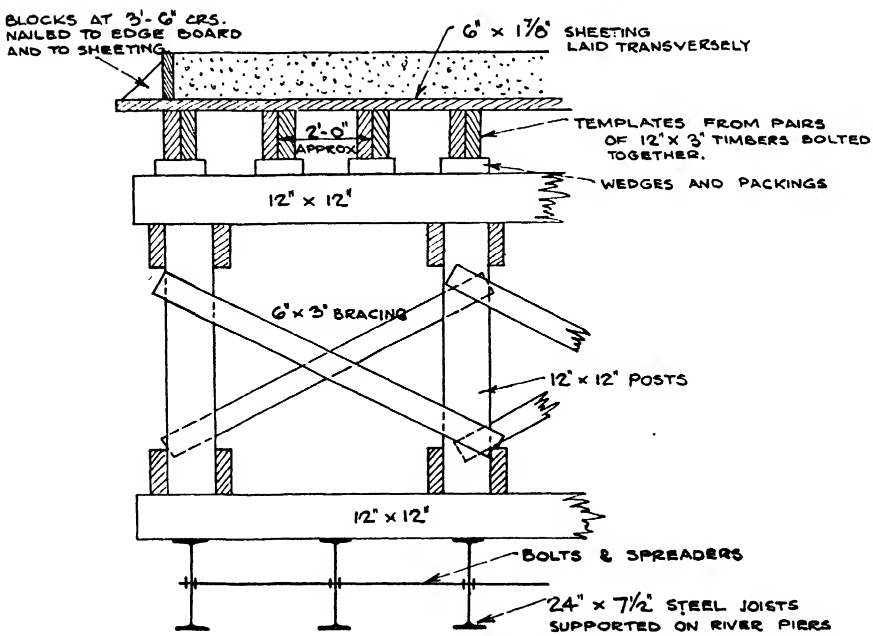


FIG. 336.—ALTERNATIVE DETAIL OF SHUTTERING FOR ARCH SLAB.

spacers between the tops of corresponding side boards maintain the shape of the box. Where the slope of the rib is steep, as near the springings, short lengths of transverse boarding are nailed across the top of the side boards as concreting proceeds.

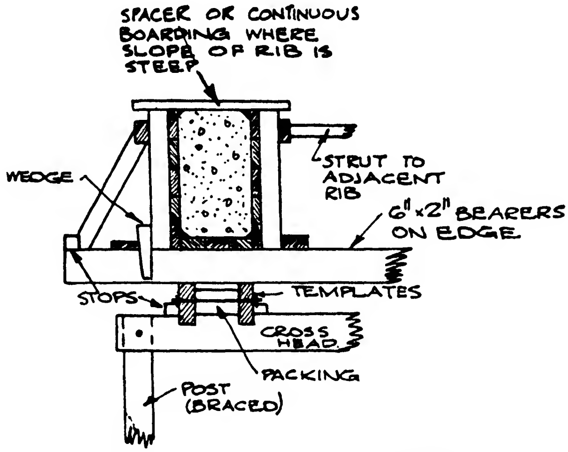


FIG. 337.—BOXING FOR ARCH RIB.

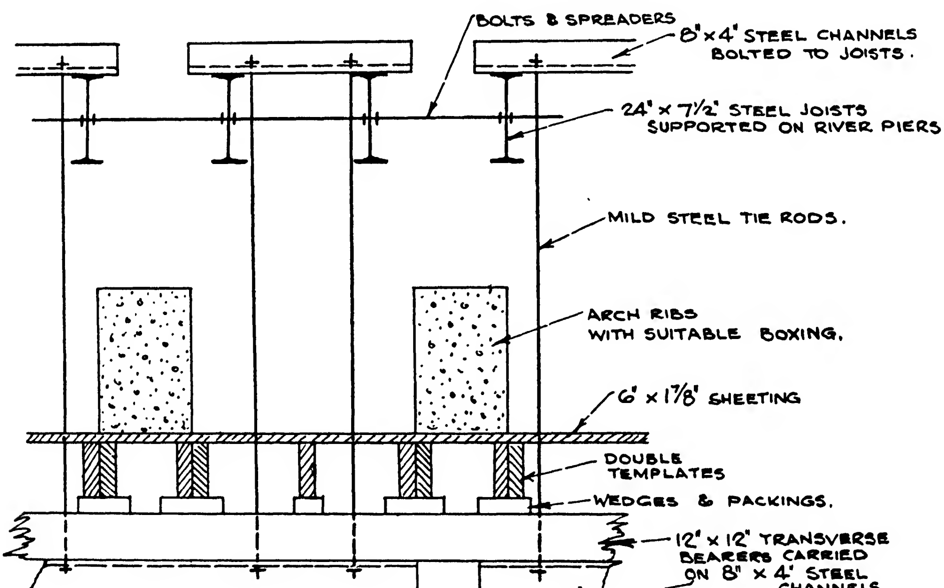


FIG. 338.—SUSPENDED RIB SHUTTERING.

When the shuttering for the ends of the arch ribs is suspended from the main longitudinal members, the method of suspension shown in *Fig.* 338 is adaptable. A short length of channel is bolted across adjacent pairs of longitudinals, and the upper ends of a pair of mild steel tie-rods are secured to the channels.

Attached to the lower ends of the tie-rods are other lengths of channels which support the main transverse bearers off which the templates are wedged. Upon the sheeting nailed to the templates, the side boards of the ribs can be erected as shown in *Fig.* 337.

In setting out the centering, allowance must be made for compression in the vertical members and settlement due to the weight of the concrete. The allowance is largely a matter of experience, but simple approximate calculations can be made. The principal object to attain is a centering so rigid that settlements are reduced to a minimum and that all points throughout the supported span settle uniformly, avoiding local points of high or low strength and of extreme rigidity or whippiness. In extensive timber trestling, arranged as in *Fig.* 324(*a*), the compression under full load may be 2 in. to 4 in.

Arrangements must be made to enable the centering to be eased before it is dismantled. For this purpose sand boxes, wedges, or jacks may be used. Sand boxes are frequently incorporated in the detail of the foot of the principal supporting posts (see *Fig.* 329), and their construction must be such that the sand is kept dry and confined until the time of striking. Hardwood folding wedges are sometimes preferred to sand boxes as the latter are reputed to be liable to sticking, while wedges can be provided at more frequent intervals and are more readily eased.

Concreting Arches.

The specified method of concreting arch ribs and slabs should be strictly followed, since it has usually been devised to minimise shrinkage stresses in the final structure and to prevent distortion of the centering during construction.

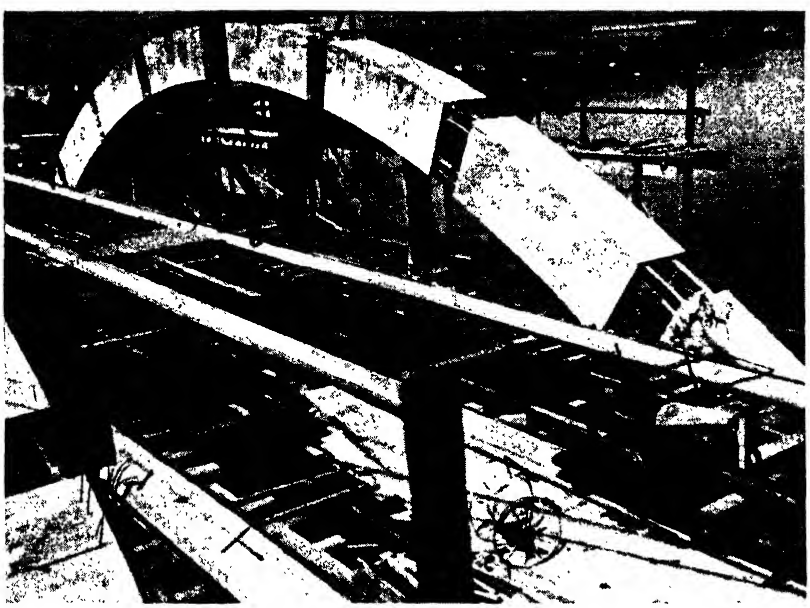


FIG. 339.—SHRINKAGE KEYS.

To attain the latter object concreting should proceed symmetrically about the centre of the arch, commencing either at the crown or at the springings. In large arch slabs it is best to commence concreting on the longitudinal centre-

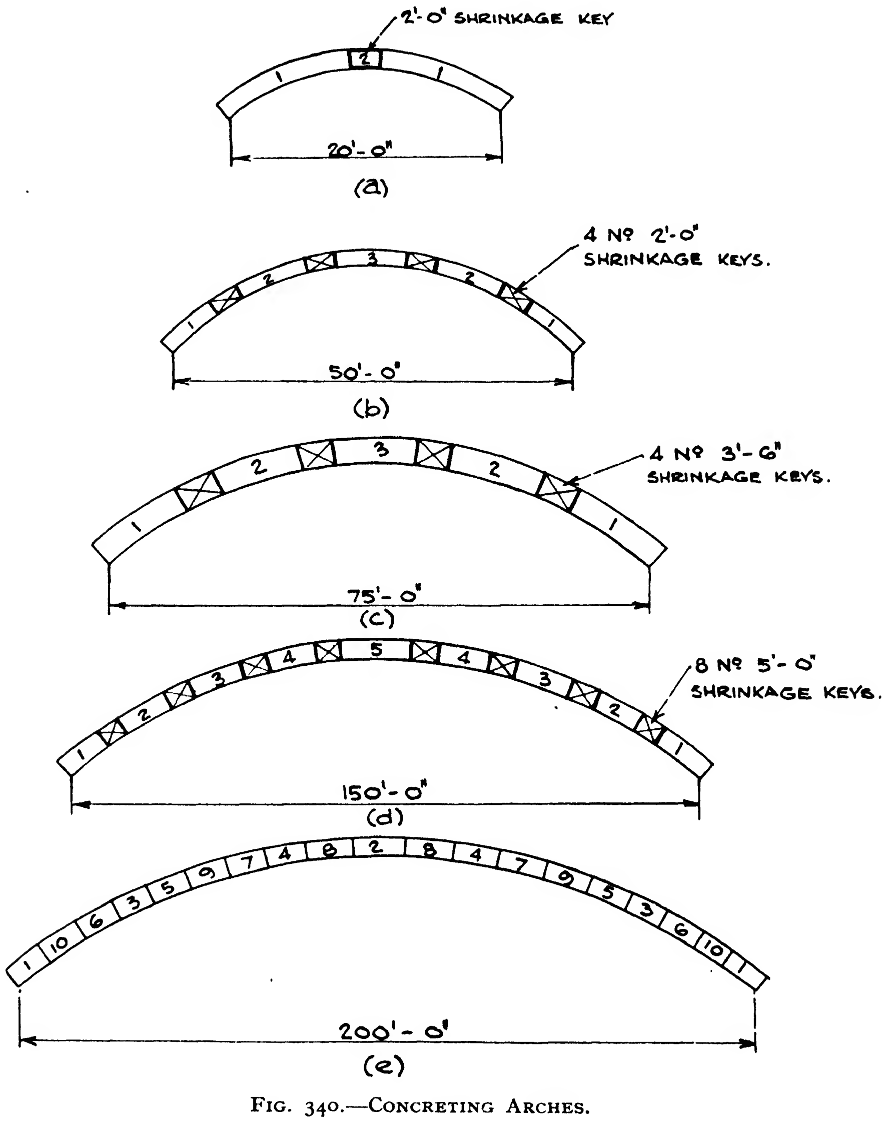


FIG. 340.—CONCRETING ARCHES.

line and work outwards towards the edges; in arch ribs, if possible the central rib or ribs should be concreted first and then the remainder in outward succession; that is, with seven ribs, No. 4 would be first, Nos. 3 and 5 would follow simultaneously, then Nos. 2 and 6, and finally Nos. 1 and 7. Alternatively, work can be started at the outer ribs and continued inwards, reversing the foregoing order. When there are only two ribs supported on a common centering, both should be concreted simultaneously.

Each rib should be divided into an even number of sections constituting a day's placing. Two adjacent sections should not be concreted consecutively. Thus the concrete in a number of isolated sections will be placed first, the intervening gaps being filled as late as possible afterwards. This allows each section to undergo its initial shrinkage before it is bonded to the completed sections. The illustration in *Fig.* 339 shows an arch rib concreted in sections with 4-ft. spaces left to be filled in after the bulk of the shrinkage has taken place. *Fig.* 340 shows typical examples of procedure. In some cases, as at (*e*), the sections are more or less equal in length or in volume, while in others the sections in the first series to be placed are longer than the final closure sections or shrinkage keys, the latter being 2 ft. to 5 ft. long. These sequences cover spans from 20 ft. to 200 ft. It is an advantage to groove the faces of the first sections in order to ensure a better key for the closure sections, the form of the joint being similar to that described in Chapter VI.

Precautions against movement of the piers during construction should be taken by adopting a symmetrical sequence of construction in adjacent spans or by shoring against unbalanced forces. If the arches are designed without hinges at the springings, small movements of the piers, either translational or rotational, produce appreciable concrete stresses for which the arches may not have been designed; these stresses may occur at a time when the concrete is not fully matured and unable to resist overstrain. It is possible to relieve piers of unbalanced thrusts from arches by retaining the centering in position until adjacent spans are complete. It is recommended that regular measurements of the pier positions should be taken to detect any movement, so that the latter can be counteracted before any damage is done.

SECTION IV.—CULVERTS.

Stream Diversion.

When a cast-in-situ reinforced concrete culvert is provided to conduct an existing stream, and is to be constructed wholly or in part along the original water-course, there arises the problem of accommodating the existing flow of water without hindering the work. If the quantity of water to be dealt with is small or intermittent, the stream can be dammed above the site and the water can be led from the " pond " thus formed through a pipeline or along an open timber or sheet-metal flume to discharge into the original stream below the site of the work.

This arrangement is shown in *Fig.* 341 with a detail for a timber flume. The gradient of the pipe or flume should be as uniform as possible. The faster the flow of water in the flume the smaller the conductor required; to obtain the maximum flow the pipe should be laid at the greatest possible slope. Clearly the shorter the path from the take-off to the point of discharge the greater the slope; the shortest path is usually along the centre line of the proposed culvert. By carrying the conductor outside and alongside the culvert, however, the advantage of obtaining a clear working space more than compensates for

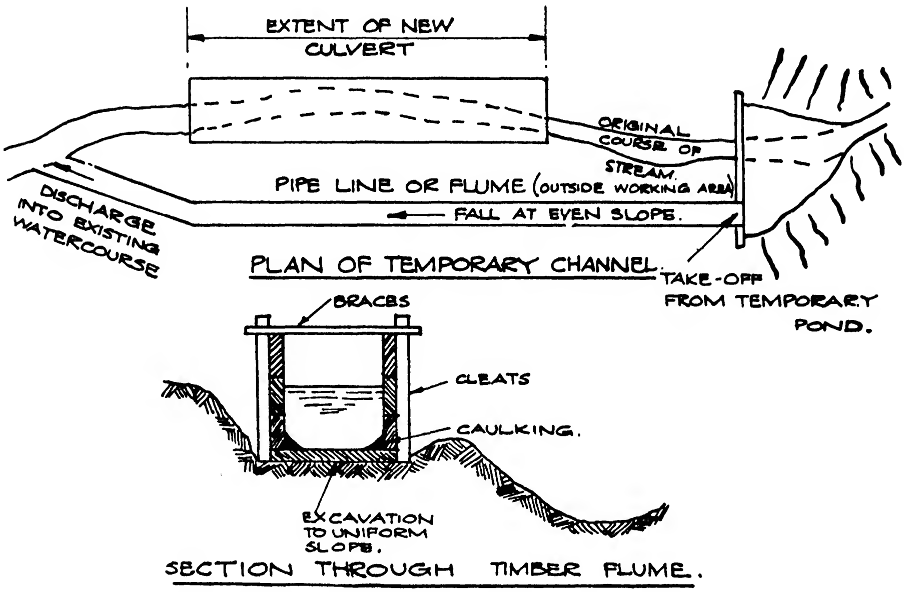


FIG. 341.—STREAM CARRIED IN PIPE OUTSIDE WORKING AREA.

the extra cost involved in providing a flume of larger capacity to conduct the water by the more devious path.

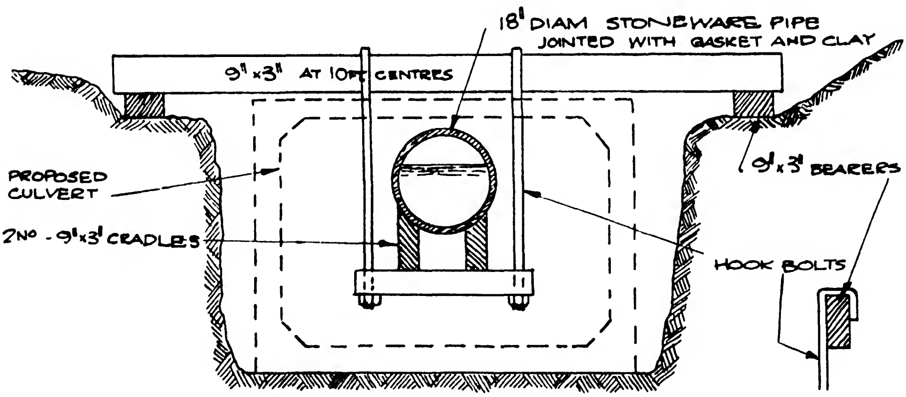


FIG. 342.—STREAM CONDUCTED IN PIPE THROUGH SITE OF CULVERT.

When adjacent property rights or other conditions make it necessary to confine the course of the pipe or flume within the boundaries of the work, a suitable arrangement is that illustrated in *Fig.* 342.

A number of transverse joists is laid across the excavation for the culvert, the ends of the joists being carried on longitudinal bearers. Hook bolts, hung from the joists, suspend a cradle which holds the pipe carrying the water. The size of the pipe required depends upon the amount of water to be dealt with, and in the case illustrated the pipe is an 18-in. diameter stoneware pipe jointed with gasket and clay. A working space of at least 12 in. should be allowed between the bottom of the cradle and the finished level of the concrete slab forming the floor of the culvert.

In many cases it is possible to dig a temporary diversion channel alongside the proposed culvert. Since the water level in this channel will generally be above the level of the bottom of the excavation for the culvert, it is necessary to take precautions to prevent water leaking into the excavation. *Fig.* 343

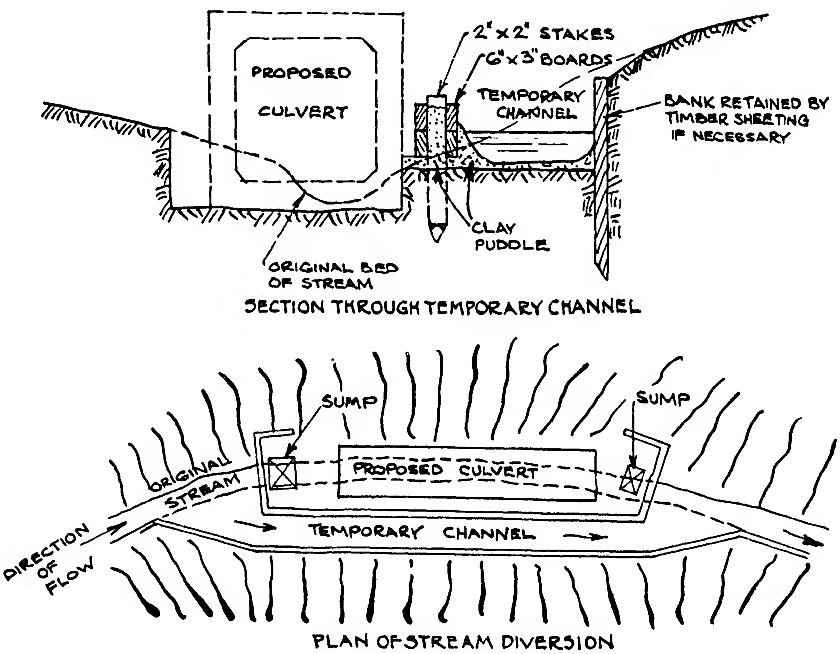


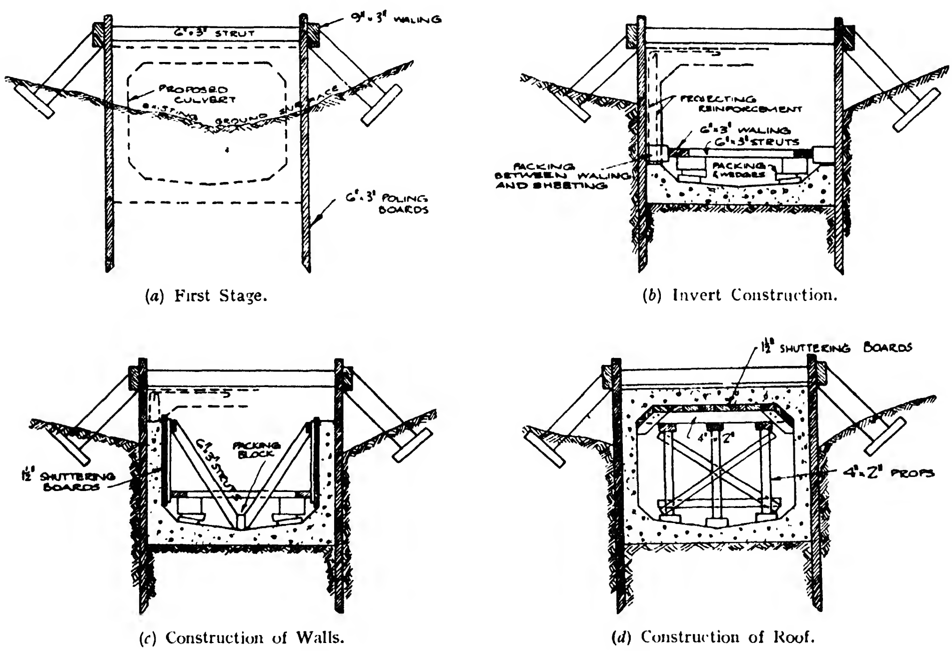
FIG. 343.—STREAM DIVERSION.

indicates a successful and economical method of building a cofferdam for this purpose. After digging a trench for the cofferdam and channel, the stakes are driven and, except for a short length across the original stream-bed, the boards are nailed to the stakes. Clay puddle is filled between the boards and the joints are caulked, and steps are taken to retain the bank of the new channel opposite the cofferdam. The bank of the stream at the lower end of the channel can now be cut through, and the upper end of the channel opened to allow the water into it. The opening in the cofferdam at the upstream end is now closed, and the lower end is closed when the water has drained out, thus isolating the working space. The excavation for the culvert now proceeds, and the concrete construction is carried out in the dry if sumps are provided.

When the culvert is complete the stop-end at the lower end of the channel is taken out, followed by that at the upper end, and the stream becomes free to flow through the new culvert. The cofferdam is then dismantled and the temporary channel filled in.

Culvert Construction.

The construction of a reinforced concrete culvert in the dry presents few problems. The work consists essentially of three stages: constructing the invert, the walls, and the roof. An arrangement of shuttering for each stage is illustrated in *Fig.* 344. If the excavation can be carried far enough at a slope to avoid timbering, economy results in shallow culverts and the operation of constructing the walls is greatly facilitated.



(*a*) First Stage. (*b*) Invert Construction.

(*c*) Construction of Walls. (*d*) Construction of Roof.

FIG. 344.—CULVERT CONSTRUCTION.

After the existing stream, if any, has been diverted or otherwise accommodated, the first operation is to drive a double row of poling boards with the upper ends strutted and fixed as at (*a*). Excavating can then proceed to the bottom of the culvert. Walings packed off the poling boards and strutted as shown at (*b*) are then inserted close to the bottom of the excavation but sufficiently high to enable the whole of the invert to be concreted. When the latter operation is completed and the concrete has matured sufficiently to take the thrust from the lower ends of the poling boards, the packings can be removed and the shuttering for the inner face of the walls erected as shown at (*c*), the poling boards acting as the external shuttering.

When the wall shuttering has been removed, the final stages illustrated in *Fig.* 344(*d*) can be carried out, these consisting of the erection of the propping and sheeting for the roof slab and concreting the roof. When the concrete in the roof slab has matured the internal shuttering can be removed and the poling boards withdrawn.

The thickness of the poling boards and shuttering timbers, the time of removal of the wall and roof shuttering, and similar factors should be considered in relation to what has been said in earlier chapters concerning these matters.

CHAPTER X

RETAINING WALLS, TANKS AND SIMILAR STRUCTURES.

SECTION I.—RETAINING WALLS.

Reinforced Concrete Retaining Walls.

THE provision of walls retaining banks of earth occurs in constructional schemes of diverse nature. Simple retaining walls in road or railway cuttings or around the boundaries of levelled sites, the walls of swimming-baths, reservoirs and docks, or the abutments and wing walls of bridges are a few instances; the construction of basement retaining walls has been considered in Chapter VIII. The stem of a retaining wall is the only part of the construction calling for special consideration, as the base slab or other foundation will normally present no unusual features.

Shuttering.—The general principles of shuttering for panel walls apply to retaining walls with such modifications as are required by the fact that retaining walls are usually built either in an excavation or with an earth bank on one face.

If the wall has not to retain an existing bank and is being built above the general ground level to retain earth or other filling that will be subsequently placed behind it, the shuttering will be erected as described later for tank walls.

When walls are constructed to retain existing banks, the design is ordinarily one of two types, (i) where the stability of the walls depends upon the counterweight obtained by the base slab extending under the retained earth, and (ii) where the base slab extends forward from the stem. In the first instance the excavation has to be taken back sufficiently far to allow the base slab to be constructed as indicated in *Fig.* 345(*a*) and the temporary retention of the face of the excavation will not affect the construction of the wall stem. Examples of the second type, as in *Fig.* 345(*b*), occur in connection with swimming-baths and in cases where the wall is against the boundary of the site, and permanent encroachment beyond this to provide a base of ample width behind the wall stem is not allowed.

In these instances, if the earth is of such a nature that it will not stand vertically, or almost vertically, for the full height of the wall, it is usual to excavate the earth as shown in *Fig.* 345 to the maximum slope at which it will stand during the period the wall is being constructed and while the concrete is maturing. In this way neither the shuttering nor the wall is called upon to resist earth pressure until the stem has attained sufficient strength to allow backfilling to be undertaken.

Reinforced concrete retaining walls are usually shuttered on both the exposed face and the earth face, except in the case of walls retaining firm clay or other non-friable material. If the earth is loose, it is difficult to maintain a vertical face; also, concrete placed in contact with it will be contaminated by earth and consequently its strength and impermeability may be impaired and the reinforcement subjected to the risk of corrosion.

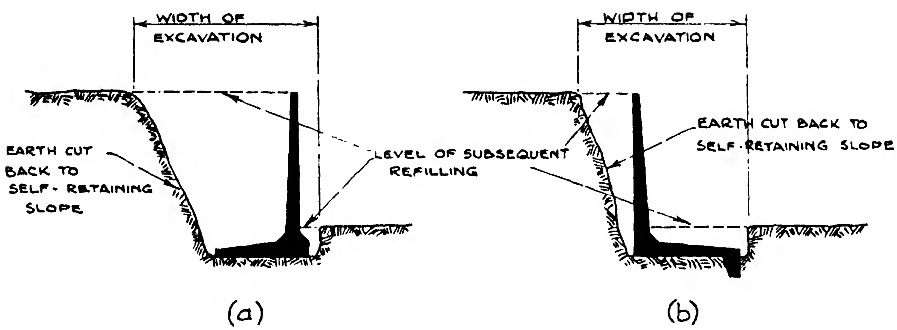


FIG. 345.—EXCAVATION FOR RETAINING WALLS.

A typical example of shuttering for the case shown in *Fig.* 345(*b*) is given in *Fig.* 346, the sheeting for the back face being propped off the earth bank and that for the front face being supported and aligned by the arrangement of soldiers and rakers illustrated. If the wall face is battered the soldiers would be set at the same slope, one, two, or more raking struts (the number depending on the height of the wall) being provided to give the soldier support at the top or at intermediate points. The horizontal bottom strut or tie keeps the foot of the soldier in position, and longitudinal alignment is obtained by the provision of two or more horizontals running continuously along the row of soldiers. Lateral movement of the whole framework is prevented by attachment to a line of well-driven stakes. The sheeting for the wall-faces is made into panels 3 ft. high and 10 ft. or 12 ft. long. These panels are raised into their successive positions as each lift of concrete is placed. No bolts or other ties are required with this arrangement, as the hardened concrete at the bottom of a lift and the top strut between opposite panels will keep the sheeting the required distance apart, while spreading is prevented by wedging from the front line of soldiers and propping from the earth bank. The weights of the panels are taken by two or more props extending from the base slab.

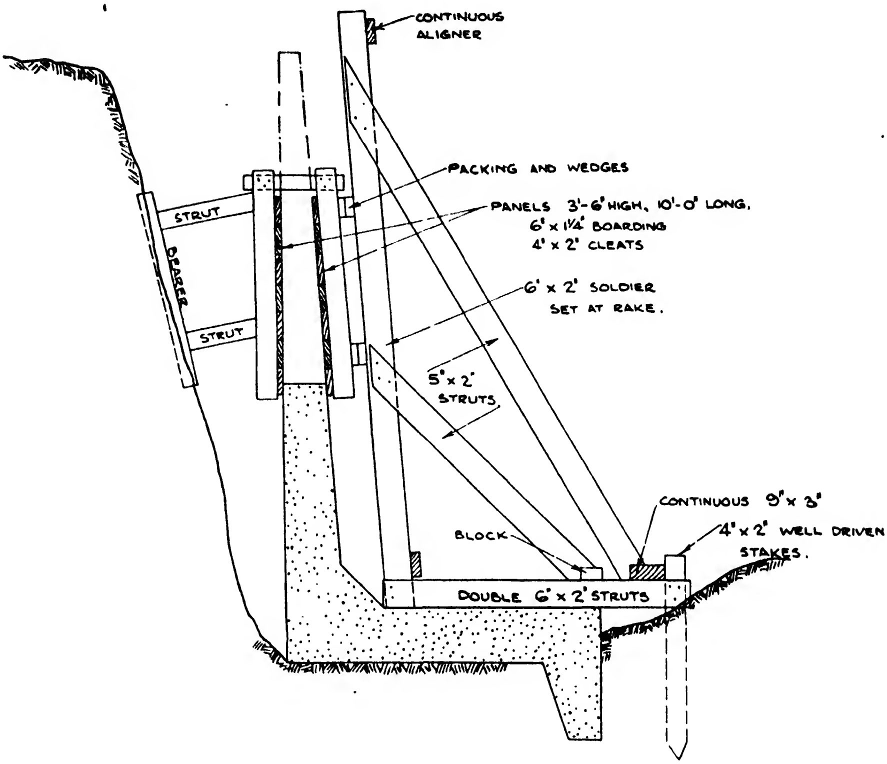


FIG. 346.—SHUTTERING FOR STEM OF RETAINING WALL.

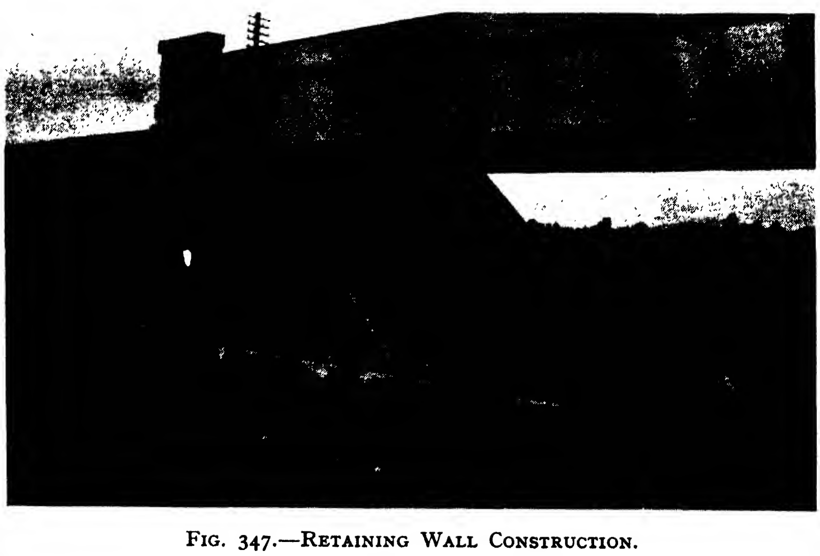


FIG. 347.—RETAINING WALL CONSTRUCTION.

A similar arrangement is adopted in the illustration on *Fig.* 347, except that the sheeting panels are the full height of the wall. This method is often convenient for long walls of moderate height that are built a section at a time and concreted to the full height in a single lift or in two lifts at close intervals. Some of the panels are then dismantled and moved laterally to the succeeding section. With high walls of moderate length a single lift of sheeting throughout the length of the wall which can be raised as concreting proceeds requires less timber than other methods and is economical.

In the case of wall-stems built away from earth banks, as in *Fig.* 345(*a*), the shuttering would be arranged as in *Fig.* 348. Two systems of soldiers are necessary, but a single system of raking struts is sufficient to ensure the stability of the complete assembly. The sheeting can be wedged from the soldiers as in *Fig.* 346, but in the example in *Fig.* 348 the sheeting is attached

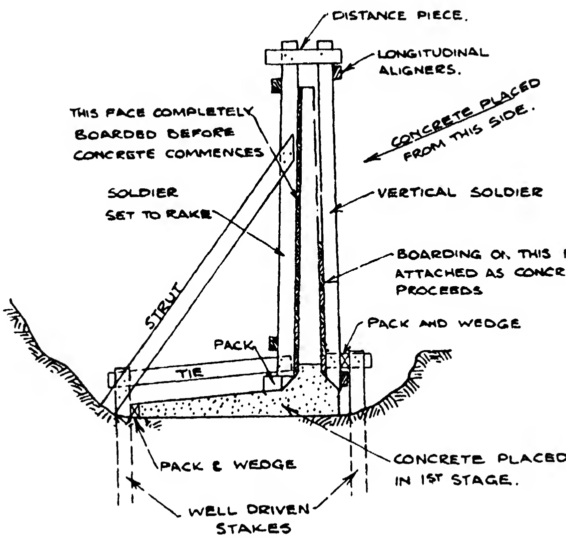


FIG. 348.—SHUTTERING FOR LOW RETAINING WALL.

directly to the soldiers. In this case the sheeting for the whole of one face and for about 3 ft. of the opposite face is attached before concreting commences, the remaining height being boarded up before placing the second lift of concrete. In the construction of the swimming-bath wall illustrated in *Fig.* 349 the boarding is also attached directly to the verticals and is provided for the full height of the wall.

For the back, or earth, face of a retaining wall, unwrought boarding can be used, but for the front face wrought boarding is essential if a fair finish is required. For swimming-baths, where rendering or tiling is applied to the inner face, unwrought boarding could be used throughout but for the fact that the amount of making-good due to defective faces at board joints may be considerable.

The spacing of the vertical members in shuttering of the type illustrated in *Figs.* 348 and 349, depends upon the thickness of the boarding and the depth of concrete placed in a single operation. This subject is discussed in Chapter VIII.

Joints.—When the wall stem is not concreted to the full height in a single operation, intermediate day's work joints should be horizontal. Many defective wall surfaces are due to sloping joints between successive deposits of concrete. The concrete should not be deposited at a single point and worked along the length of the wall, but should be placed evenly in layers 6 in. to 12 in. deep throughout the length of the section. The number of layers required depends on the depth of wall being concreted in a working shift ; it is usual to limit this depth to 3 ft. or 4 ft.

In long walls permanent vertical expansion joints are usually provided ; the design and construction of such joints is considered in the next chapter. The section of the wall concreted in each shift should, if practicable, extend between two consecutive permanent joints, but if temporary day's work joints

FIG. 349.—SHUTTERING FOR SWIMMING-BATH WALL.

are required intermediately they should be vertical and should be rebated as described in Chapter VI. When permanent joints are not specified in a wall 30 ft. or more in length, cracking due to shrinkage is liable to occur. This risk can be minimised by proper curing (see Chapters VI and VIII) and by casting the wall in sections, each 10 ft. to 15 ft. in length, leaving a gap of 1 ft. 6 in. to 2 ft. 6 in. in width between adjacent sections. The vertical end faces of the gaps should be rebated to provide a key for the concrete filling the spaces. The filling should be carried out as long as possible after casting the main sections, thus allowing the bulk of the shrinkage of the long sections to take place before completing the wall.

Earth Filling.—Earth filling behind retaining walls should not be placed before the concrete has attained sufficient strength to resist the stresses produced by the pressure from the filling. The engineer generally specifies the period to be allowed between depositing the concrete and the filling, but when low temperatures have not been experienced and when the concrete has been properly cured this period should be not less than four weeks with normal-hardening Portland cement and ten days with rapid-hardening Portland cement. When back-filling, the earth should be deposited in layers up to 12 in. deep evenly spread along the wall and should not be dropped from a height. When a rubble backing is specified it should be placed to the required thickness in contact with the wall in advance of the earth filling. If large rubble is used it is advantageous to pack this by hand in such a way that it is largely self-retaining.

Weep-holes to prevent the accumulation of water behind the wall can be conveniently formed by inserting clay-tile pipes between the sheeting of the wall stem before depositing the concrete. The use of iron pipes for this purpose causes rust marks on the wall.

In constructing important concrete retaining walls, consideration should be given to possible contamination of the water that may percolate through the ground and come into contact with the wall (see "Ground Waters," Chapter VII). The superficial appearance of walls in exposed positions may be important, and relief to plain surfaces may be afforded by panelling or by one of the methods described in Chapter VIII.

Mass Concrete Retaining Walls.

Shuttering.—Heavy mass concrete retaining walls of the types met with in the construction of docks, canal-locks, and similar structures built below the general ground level, are usually shuttered on one face only, the face of the excavation providing the shuttering for the opposite face. In the case of dams, reservoirs, or other structures which are mostly above ground or in an open excavation, both faces of the wall require to be shuttered.

In the example of the dock wall reproduced in *Fig.* 350 the lower part of the wall is built against the trimmed face of an excavation in rock or other firm ground, but the upper section being in a loose top-soil is shuttered on both faces. The shuttering for the exposed face of the lower part of the wall consists of panels of 2-in. boarding, allowing the wall to be built in successive vertical lifts of about 5 ft. The panels, constructional details of which are shown on the illustration, are maintained in position by soldiers set parallel to the face of the wall. As shown, these soldiers require to be moved for each lift, their alignment being maintained by two bolts left in the portion of the wall already concreted. The bolts should be well greased or inserted in cardboard tubes so that they can be easily removed for use in subsequent lifts. Upon withdrawal of the bolt the nut and washer are left in the concrete, and the bolt-hole is grouted up upon completion of the work.

For mass concrete walls of moderate height shuttered on both faces, the construction illustrated in *Fig.* 351 has been successfully employed. The sheeting is attached directly to the double soldiers erected on both sides of the wall which are set vertically, or to the same slope as the wall-face in the case of battered walls. The feet of the soldiers are prevented from spreading and their heads are held in position by a timber cross-piece. At intermediate positions, tie-rods extend between pairs of opposite soldiers. These rods, which are well greased and are given a half-turn while the concrete is still green to facilitate withdrawal, are set at an angle to allow grout to run down the bolt hole when the latter is stopped at the completion of the work.

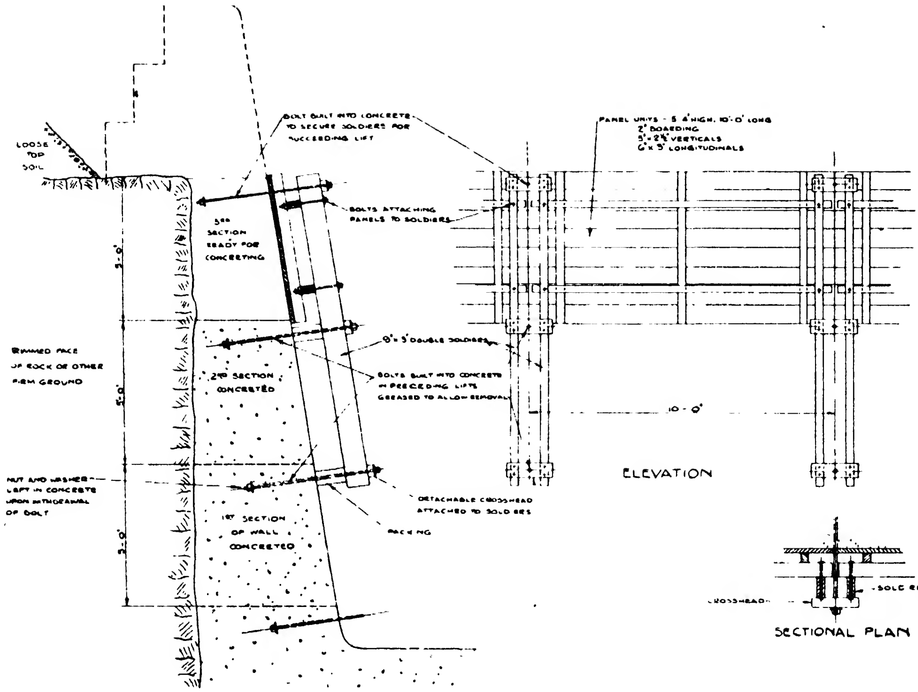


FIG. 350.—MASS CONCRETE WALL SHUTTERED ON ONE FACE.

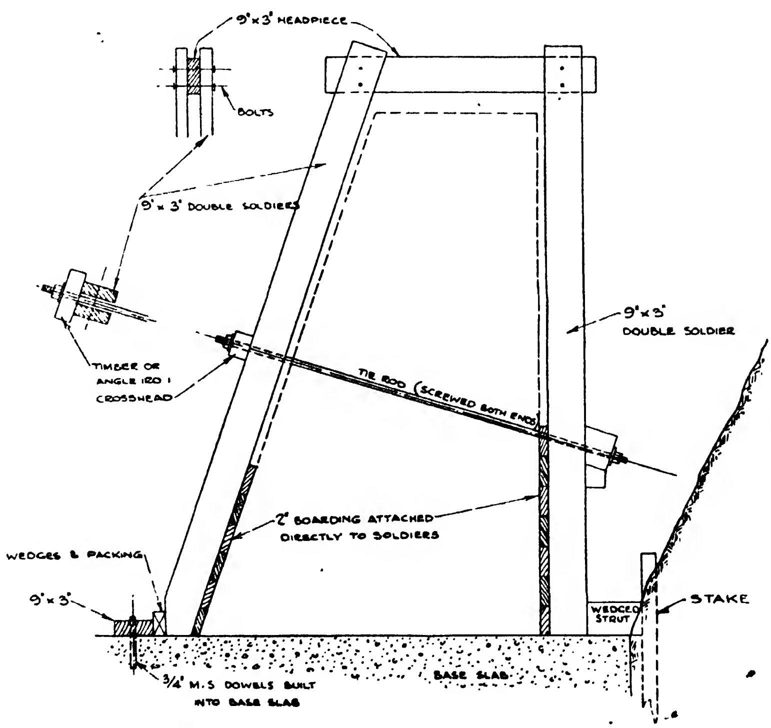


FIG. 351.—MASS CONCRETE WALL SHUTTERED ON TWO FACES.

For walls of greater height and considerable thickness in open excavations the method of construction shown in *Fig.* 352 is applicable. The sheeting is made into panels consisting of horizontal boarding and transverse joists aligned by a pair of longitudinal walings, through which passes a series of bolts. Before concreting proceeds the bolts are driven in (as shown in the lower waling) and thus the screwed end-plate becomes embedded in the concrete. When the panels are released for erection in the next lift, bolt-holes and end-plates left by the top waling of one lift provide attachment for the bolts of the lower waling in the next lift. Thus the foot of the panel is secured to the portion of the wall

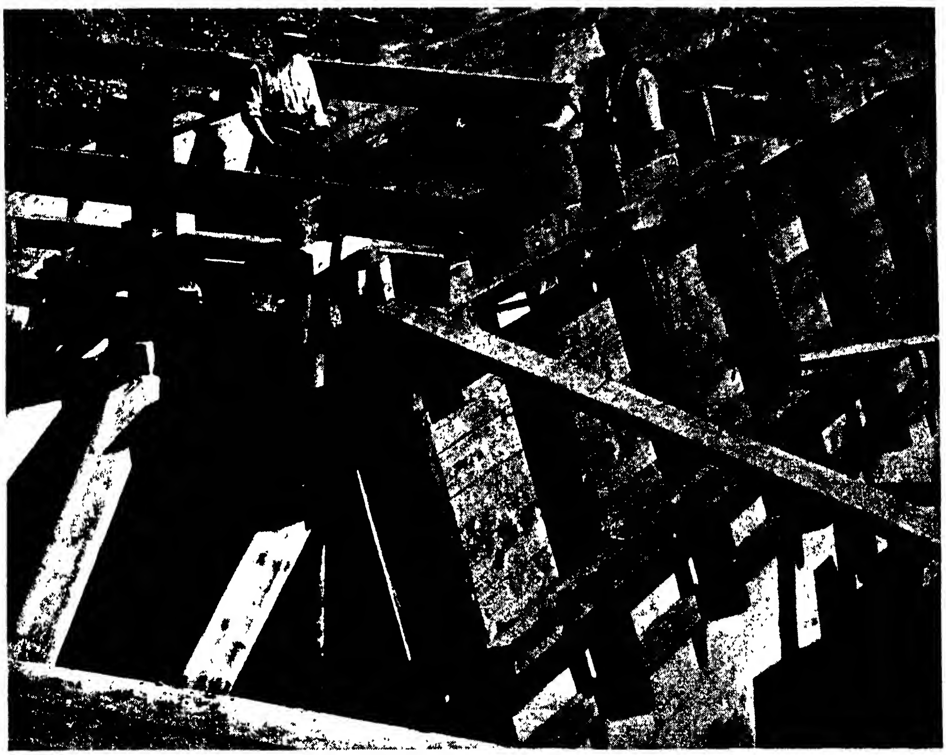


FIG. 352.—MASS CONCRETE WALL IN OPEN EXCAVATION.

already concreted, and the tops of opposite panels are connected by cross-pieces placed at the junction of adjacent panels.

Joints.—When placing concrete in heavy mass walls it is just as important as in lighter reinforced concrete walls to maintain day's work joints horizontally and vertically, avoiding inclined joints. Horizontal joints particularly should be rebated, the size of the key being commensurate with the dimensions of the wall, and thereby increasing the resistance of the joints to horizontal shearing actions. The wall shown in *Fig.* 351 is cast in sections each 30 ft. in length.

Consistency of Concrete.—As concrete in walls of this kind is placed in large quantities, a considerable volume of excess water would collect on the surface of the concrete if the consistencies common in reinforced concrete work were used. Much drier concrete is therefore desirable, slumps of 1 in. or less being common. In many cases the concretes are only "earth-damp" and require continuous tamping until liberal moisture appears on the surface. The absence of reinforcement and awkward corners permits the drier mixtures to be used. The gauge of aggregate material and the use of "plums" in mass concrete work have been considered in Chapter V.

Sheet Pile Walls (Reinforced Concrete).

The principles governing the design, manufacture, and driving of king piles as described in Chapter VII apply in most respects to sheet piles, but the latter are usually in smaller units and are interlocked to maintain alignment and to form a wall of sheeting that will prevent seepage of the backfill. A number of interlocking methods are shown in *Fig.* 353. The piling is driven with the groove facing the next pile to be driven, the latter being placed so that its tongue interlocks with the groove of the preceding pile. A projection on the shoe of one pile serves to clean out the groove in the preceding pile.

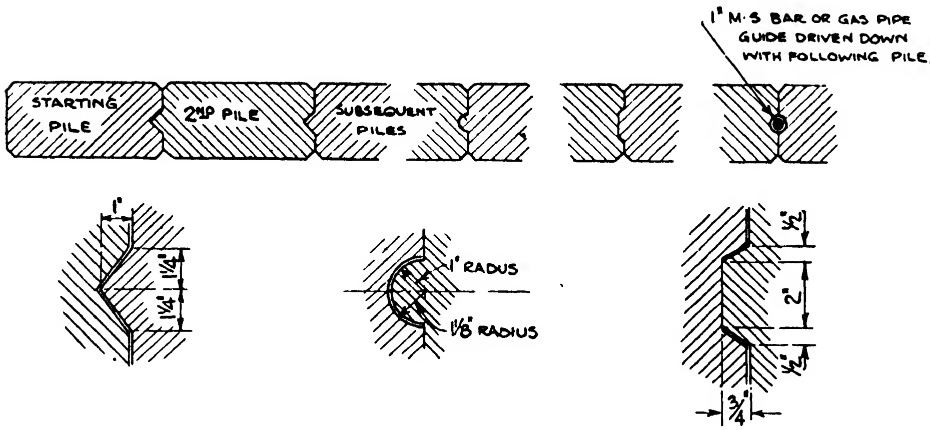


FIG. 353.—INTERLOCKING SHEET PILES.

Shoes.—Light concrete sheeting when driven into soft ground may not need shoes, but some form of protection is needed for heavy sheet piles driven into gravels or ground where hard patches or obstacles may be encountered. The toe of a light pile may be protected by a mild steel vee-shape plate, but one of the standard designs of cast-iron shoes shown in *Fig.* 354 is generally used with piles 12 in. by 5 in. or larger in cross-section. The shoes are attached to the piles by wrought-iron straps and are assembled with the reinforcement in the mould before concreting.

Manufacture.—The moulds for sheet piles are similar in construction to those for king piles; a simple design in timber is illustrated in *Fig.* 355 in which the side boards are arranged so that they can be removed without disturbing the pile or the bottom board. A method in which steel sides are used with a timber bottom board is given in *Fig.* 356. The reinforcement can usually be

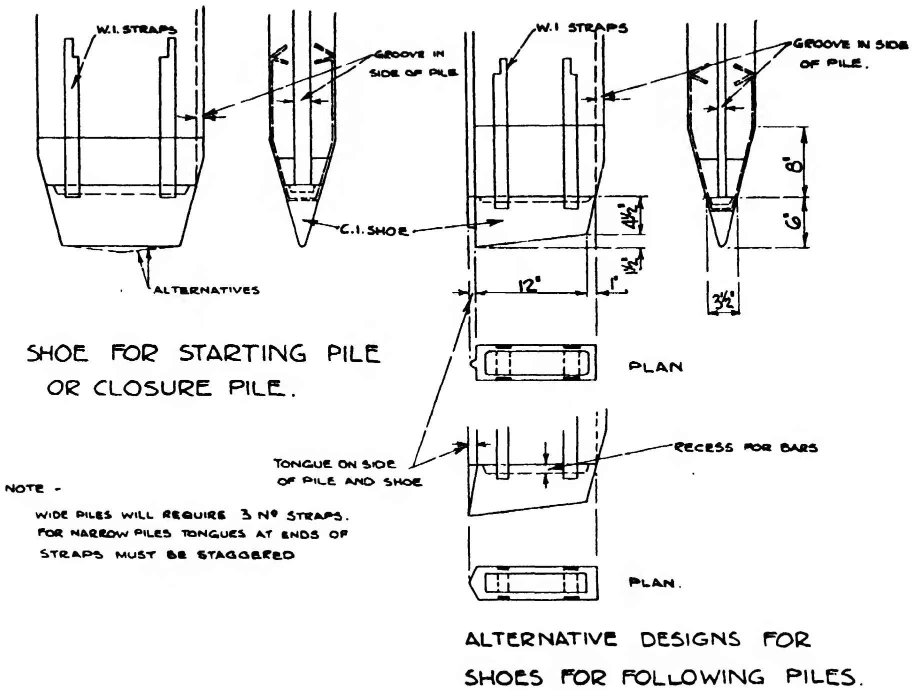


FIG. 354.—SHOES FOR SHEET PILES.

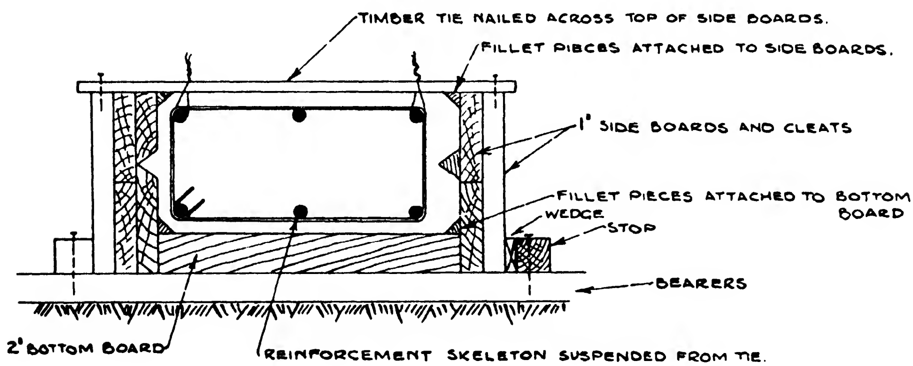


FIG. 355.—TIMBER MOULD FOR SHEET PILES.

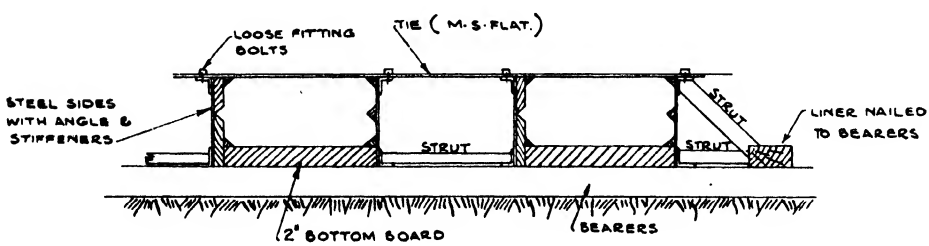


FIG. 356.—STEEL MOULDS FOR SHEET PILES.

assembled as a unit before placing in the moulds. Toggle holes provided for guiding can usually serve also for lifting, and are formed by inserting lengths of gas pipe tubing vertically in the moulds. The number of moulds required depends upon the number of piles that can be driven in a day and the time required for curing them. Since the driving of sheet piles may be less severe than that of bearing piles, sheet piles made with normal Portland cement can be driven at twenty-one days and at seven days with rapid-hardening Portland cement if the conditions of curing are suitable. If heavy piles are involved or hard driving is anticipated, the periods specified in Chapter VII for bearing piles should be adhered to. The sides of the moulds can be removed after, say, two days, and the pile can be rolled off the bottom board at not less than fourteen days, using ordinary cement, but lifting at this early age is better avoided. When the piles are stacked for curing they should be supported at close centres and should be so arranged that air can freely circulate around all four faces.

The minimum number of moulds required in a given case can be assessed as follows. Assume eight piles to be driven daily. The rate of casting should be at least equal to the rate of driving, unless casting commences considerably in advance of driving; therefore eight piles must be cast each day, requiring $8 \times 2\frac{1}{2} = 20$ sets of side boards, since these can be removed at two days, and allowing half a day for cleaning, re-oiling and re-assembling. Each bottom board will be in use for, say, fourteen consecutive days per pile, that is eleven working days. The number required is therefore $8 \times 11 = 88$. Thus there will be required a minimum of twenty complete moulds with sixty-eight additional bottom boards. Unless several hundred piles are required this would not be economical. If only fifty piles were required it would be better, if possible, to start casting the piles in advance of the driving. Since the driving of this number of piles would take about seven working days, the last of the piles should be cast a fortnight before commencing driving so as to allow three weeks for curing. If five uses are expected from each set of side boards, the number of moulds required for fifty piles is ten. Casting can then proceed at an average rate of ten piles in each $2\frac{1}{2}$ days, or fifty piles in $12\frac{1}{2}$ days. Thus casting should begin about one month before driving commences, and in addition to five complete moulds, forty-five bottom boards would be required. If rapid-hardening Portland cement is used and the side boards are released at one day, and the piles taken off the bottom boards at seven days and driven at fourteen days, making should begin about a fortnight before driving commences, and in addition to five complete moulds about forty bottom boards would be required. The number of bottom boards can be reduced by starting casting earlier. When large numbers of piles are to be cast at the same rate as driving proceeds, sixteen complete moulds and thirty-two additional bottom boards would be required if rapid-hardening Portland cement is used and eight piles are driven daily.

Driving.—Sheet piles principally resist horizontal forces and are driven to a prescribed depth without reference to the set obtained. Thus driving sheet piles is usually easier than driving bearing piles. A greater number can therefore be driven in a given time, and six to twelve piles driven in an eight-hour day would be a fair output for average lengths. The larger the pile the fewer driven in a given time, and vice versa.

While driving the piles the usual precautions should be taken to protect the heads from damage. Light sheet piles can be conveniently driven by a double-acting pneumatic hammer suspended from a tripod or by a light monkey suspended from a frame and operated by hand. Heavier piles require similar

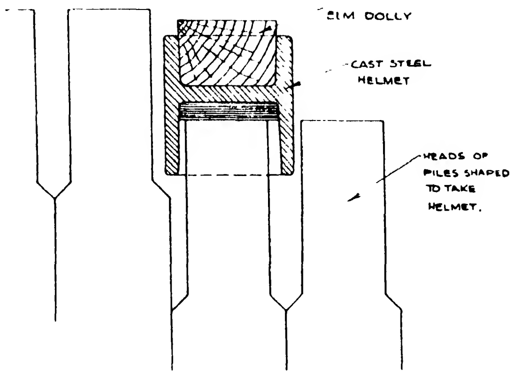


FIG. 357.—SHEET PILE FITTED WITH HELMET.

equipment to bearing piles. If hard driving is experienced it is necessary to provide a helmet, dolly, and cushion, and to shape the head of the pile as indicated on *Fig.* 357 to accommodate the helmet.

Light piling is often driven in "banks" (*Fig.* 358), this system assisting

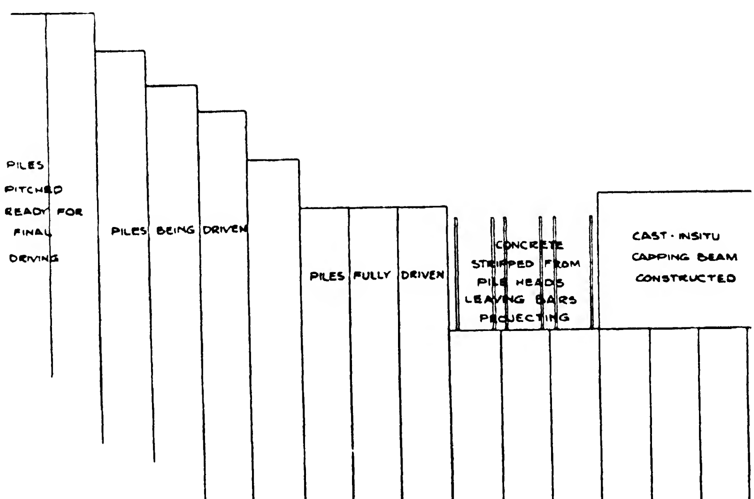


FIG. 358.—DRIVING SHEET PILES.

in preserving the alignment and being easily carried out if a portable hammer is used. Temporary waling timbers are also provided to maintain alignment, especially when the line of sheeting is to be driven on a curve. When obstructions are met that tend to deflect the pile from its true course, the retention of these walings attached to piles already driven for some distance behind the piles being driven will help to keep the line. When the obstruction, or other cause, leads to severe departure from the alignment the difficulty may be overcome by driving another "starting" pile on the original line as close to the deflected pile as possible and continuing with "following" piles. The gap between the deflected pile and the additional starting pile can be closed by driving a closure pile similar to a starting pile behind these two piles. When driving into sand, a water-jet facilitates sinking the pile without undue hammering, the arrangement for carrying the water to the toe of the pile being similar to that described for king piles.

Ties.—The heads of sheet piles used in constructing river walls and wharves are generally built into a capping beam or other cast-in-situ extension and tied back to anchor blocks or piles. A typical arrangement is shown in *Fig.* 359. In the simplest cases the procedure after driving is to strip the concrete from the pile-heads far enough to expose sufficient length of reinforcement to bond into the capping beam. The shuttering for the latter is then erected and the

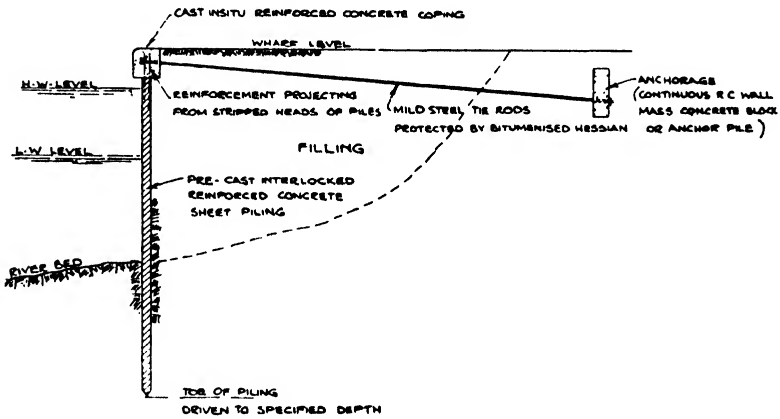


FIG. 359.—SHEET PILE WALL.

anchor rods are put in position, taking care that the turnbuckles, if any, allow for ample adjustment in both directions. When the reinforcement in the capping beam has been fixed the beam is concreted, and the concrete is deposited in the anchor block or the head on the anchor pile. When the concrete in the beam and block has matured the tie-rod should be tightened by turnbuckles or otherwise; this tightening should be only sufficient to take up slackness in the rod without overstressing it.

Back Filling.—The earth filling behind the wall can be placed after the protective coating (bituminised hessian, concrete, or other specified material) has been placed around the tie-rods. The filling should be placed in layers, 1 ft. to 2 ft. thick, working forward from the anchorage to the sheeting. It is important that the sheet piling should not be subjected to any considerable lateral pressure before the anchors are firmly embedded. To minimise the pressure behind the wall and to prevent escape of the filling with any water that may seep through toggle holes or open joints in the wall, a backing of rubble, hand-packed where practicable, is advantageous. Generally toggle holes and bad joints should be grouted up unless the engineer desires to prevent water being trapped behind the wall due to falling tides or other causes.

SECTION II.—TANKS.

Impermeable Construction.

An essential in producing impermeable concrete is a dense concrete. This is obtained by using clean graded aggregates, the concrete being well mixed and thoroughly tamped into position, paying due regard to the position of the construction joints, and to effective curing. With these conditions assured concrete will be watertight even when subject to high hydrostatic heads, and some authorities quote 40 ft. head of water as a practical maximum. It is a prime necessity that the thickness of the concrete and the amount of reinforcement be designed in accordance with the principles controlling good practice.

Limitation of the effective tensile stress on the concrete is used to prevent the formation of cracks due to excessive bending or direct stresses caused by the pressure of the liquids. Neglect of this factor may cause defects in otherwise well-constructed tanks. If a concrete tank or reservoir has been well designed and conscientious construction can be assured, other aids to impermeability are unnecessary.

Density can only be obtained by using well-graded aggregates. Particular attention should be paid to the proportion of the smaller particles of the fine aggregate, and an excess of the latter is not objectionable. Gravel aggregate is often preferred to broken stone for watertight work, and the mix is usually richer in cement than for normal construction. Thus, mixes equivalent to $1 : 1\frac{2}{3} : 3\frac{1}{3}$, $1 : 2 : 3$, or $1 : 1\frac{1}{2} : 3$ are frequently specified for tanks. The recommendations concerning mixing, placing, and curing given in earlier chapters should be complied with throughout every operation. The concrete should be plastic, extreme dryness being avoided; on the other hand, sloppy mixes should not be used. An increase in mixing time may be advantageous in giving better workability. With a gravel aggregate a slump of 3 in. to 4 in. is reasonable.

The concrete between the reinforcement and the shuttering on the water-face should be well compacted and the board joints tight, so as to produce a face free from honeycombing or large pores through which the seepage can commence; external vibration can be employed to produce a compact concrete with a dense skin.

Joints.—Joints are potential points of leakage; where possible, in circular structures of small or moderate diameter, vertical joints should be avoided. This can be done by completing a ring 2 ft. or 3 ft. deep in a continuous operation, working around the circumference in both directions from the starting-point. It is also advantageous to avoid horizontal joints by using continuously-moving shuttering or by providing sufficient shuttering of normal type to enable the whole of the walls to be constructed without interruption. Where day's work joints are formed, whether horizontally or vertically, they should be rebated as described in Chapter VI. The old concrete should be well hacked to expose the coarse aggregate and after washing the surface a layer of mortar or grout should be spread on it immediately before resuming concreting. A strip of 20-gauge copper, acting as a water-bar, is sometimes placed through the joint, especially where the tank walls bond into the floor. Where an effective bond cannot be assured at horizontal joints, a method that has been successful is to form a spigot and socket joint well caulked with asphalt. The joint should be designed to ensure that the water pressure tends to force the asphalt into, rather than out of, the joint. Joints should not be made at changes of concrete section if they can be conveniently placed elsewhere. Moving them even a few inches is often sufficient to prevent the secondary stresses being concentrated in one place..

In long reservoir walls the design often incorporates permanent joints which assist in preventing cracks due to shrinkage and temperature changes. When these joints are not indicated on the drawing it is best to concrete the wall in short sections, leaving a space of 1 ft. 6 in. to 2 ft. 6 in. between sections. As long as possible after completing the sections the intervening spaces can be filled in, taking care to bond the old with the new work.

Fittings.—Pipes or other fittings passing through the walls and bottoms of water-containing structures are another potential source of weakness, and should be on the site in time to erect them in position in the shuttering before concreting commences. They will then be well embedded in the concrete and, if provided with normal water-bar flanges, there is little risk of leakage. An objection to building in fittings is that if the flanges are not in line with connecting flanges extra labour is required in making adjustments. Some engineers therefore prefer to leave holes and fix the pipes and fittings after completing the concrete work, but it is difficult to ensure that this can always be executed so well that leakage is avoided.

Fittings built into concrete tanks should preferably be of non-corrosive material such as phosphor bronze, thus avoiding replacements that may affect the future watertightness of the structure. If made of ferrous metals the sections should be sufficiently substantial to prevent them being weakened by a small amount of corrosion. Care should be taken when incorporating lead, zinc, or aluminium fittings, as when these metals are in contact with Portland cement concrete, especially wet concrete, deterioration of the metal takes place. Parts made of these metals should be protected by bitumen or other coatings.

Integral Waterproofing.—Many authorities do not favour attempts to secure impermeability by using admixtures, but numerous proprietary substances are available that serve this purpose. The manufacturer's instructions should be followed implicitly if success is to be attained. The object of most integral proofers is mechanically to fill the pores in the concrete, thus obtaining a denser and less permeable material; others obtain a similar result by chemical action on the cement. Although improvement may result from using admixtures it is doubtful if any admixture fully compensates for defective concrete, that is if the defects are due to poorly-graded materials, indifferent mixing, or careless placing. There is no doubt a field for such admixtures when conditions are such that strict control of concrete making and placing is unavailable.

Hydrated lime is an admixture that can be considered as a pore filler. The usual quantity recommended for a 1 : 2 : 4 mix is 8 per cent. by weight of the cement, increasing for leaner mixes to, say, 16 per cent. for a 1 : 3 : 6 mix.

Other inert void fillers in use are felspar, clay, trass, and pozzolana, but such materials can be dispensed with if sands, well graded in the finer sizes, can be obtained in the first place. The quantity of trass required to produce impermeable concrete varies with the grading of the sand, but normally about 25 per cent. of the weight of cement is required, although up to 40 per cent. may be required to resist chemical attack.

Powdered pig-iron or iron filings, which upon oxidisation expand to fill the pores, has been tried effectively. The addition of soap (0·05 per cent. by weight of the mix) was in vogue at one time as a waterproofing agent; if it is used it is necessary to keep the concrete surface damp for several days to ensure success. These methods are not generally recommended to-day. Whereas inorganic pore fillers may not adversely affect the strength of the concrete, the admixture of soap or oil seriously reduces the strength.

Waterproof Linings.—Waterproof linings include paints, renderings and asphalt. These are used to guard against seepage and to ensure that the contents of the container will not attack the concrete. Details of protective treatment are given in a subsequent paragraph. To be successful a coating must be insoluble, impervious, non-corrosive, and to a certain extent elastic. It should be applied after cleaning the surface of the concrete, and the latter must be quite dry when asphalt or bituminous coatings are used. The following are among the most common treatments applied by brushing.

Alternate application of alum and soap is one of the oldest methods of attempting to fill the surface pores and so check seepage, but is not frequently used to-day, being largely superseded by the sodium silicate treatment. The alum solution, consisting of 2 oz. of alum in a gallon of hot water, is first well worked into the surface by a stiff brush, and is followed immediately by the soap solution, the latter generally containing 12 oz. of soap to each gallon of water. This process should be repeated a number of times; its effectiveness depends on the formation of insoluble aluminium soap in the pores of the concrete.

Paraffin wax may be applied in two or three coats of a solution of one part of paraffin wax in two parts of paraffin oil. Each coat should be well rubbed into the concrete surface with a stiff brush. Alternatively, in the form of molten wax, it can be applied to a concrete surface heated by a blow-lamp.

The most common way of impregnating a surface is by brushing on sodium silicate, or alternatively aluminium, magnesium, or zinc fluosilicate. A high-silica solution is preferable and gives better results than the lower silica solutions such as commercial waterglass.

Renderings of cement mortar are also used with the object of covering weak patches in the concrete. The rendering should be applied to the concrete while it is still green, the shuttering being struck as early as possible and the rendering applied immediately. The concrete surface should be well wetted and, if necessary, hacked or otherwise treated to form a key. If the rendering is applied in two coats they should break joint. The mortar should not be steel trowelled, but finished with a wooden float.

For water tanks, where the concrete may be subject to considerable pressure, asphalt linings are common. Metal, brick, glass, or similar linings are often adopted when corrosive liquids are stored. Asphalt should be applied to a perfectly dry and clean surface in two coats breaking joint. It is a particularly suitable lining where there is any probability of the concrete cracking due to the adoption of high working stresses in the design, lack of provision for settlement, contraction, or the high temperature of the contents of the tank.

Protection against Chemicals.

Since many acids and chemical compounds have an injurious effect on concrete, storage vessels in industrial structures and floors in certain factories require to be protected against penetration and disintegration.

The method to be adopted in a particular case is a question of design and is usually in the specification. It may vary from simple coating of the surface with asphalt or a solution such as silicate of soda, magnesium fluosilicate, etc., to the construction of a complete lining of slabs, tiles, or plates or a corrosion-resisting metal container inside the concrete storage vessel. In the simple treatments there is no difficulty in making joints, but a lining of tiles, or similar materials, must be jointed with equally non-corrosive material if it is to be effective. Metal linings, also, require to be carefully jointed. Riveted joints are untrustworthy unless the seams are also soldered, brazed, or welded, and generally require to be made by a coppersmith. Not every metal can be placed in direct contact with the concrete and remain unharmed. Copper, for instance, is safely used in breweries for storage, but aluminium corrodes when in contact with concrete, and if it is to be used as a lining there should be a layer of asphalt to separate it from the concrete tank.

So far as the contractor is concerned the principal point requiring consideration is to ensure that the concrete is perfectly dry before asphalt or paint is applied, and thoroughly wet if any kind of mortar is to be used for bedding tiles or in the form of rendering. Asphalt laid on wet concrete becomes pitted, and mortar laid on dry concrete loses some of the water required to enable it to set properly.

In more complicated treatments—some of which involve several operations —the contractor should follow the manufacturer's instructions very closely to obtain satisfactory results.

Construction of Cylindrical Tanks.

Shuttering.—Timber shuttering for the walls of cylindrical tanks of diameters up to 30 ft. is usually constructed as shown in *Fig.* 360, where details of inside and outside units are illustrated; in each case the units consist of vertical boarding attached to horizontal ribs. The latter are concave for the outer units and convex for the inner, and are cut from timbers 6 in. to 9 in. wide and 1½ in. to 3 in. thick. The minimum width of the ribs when cut should not be less than 3 in. The thickness of the vertical boarding should be not less than 1½ in. if the edges are shot or less than 1 in. if they are tongued and grooved; the width should be 4 in. to 6 in., the narrower boards being used on tanks of small diameters. With 6-inch boards, the resulting surface is made up of a number of 6-inch flats which in practice gives an appearance approximating to a true cylindrical surface. If for any reason flats of this width are objected to, narrower boards would have to be used with consequent increase in labour in constructing the units. Since the shuttering for circular work is more expensive than for plane work, the units should be capable of many re-uses. For this reason it is advisable to use screws rather than nails. Tongued and grooved boarding offers advantages, as tight joints are assured in spite of alternate wetting and drying under which action the joints between shot edges are liable to spring. Damage to the tongues is eliminated once a unit has been made-up as it is not dismantled while still serviceable. The overall length of each unit should be between 4 ft. and 8 ft., increasing with the diameter. Its depth should correspond to the height of a lift of concrete. Sufficient units should be made to enable two complete belts to be erected around the tank walls. In this way the concrete can be allowed to harden in one lift while the units forming the shuttering for the preceding lift are dismantled and re-erected for the next lift. Units in any one belt butt up against each other as shown in *Fig.* 361, and are connected by a pair

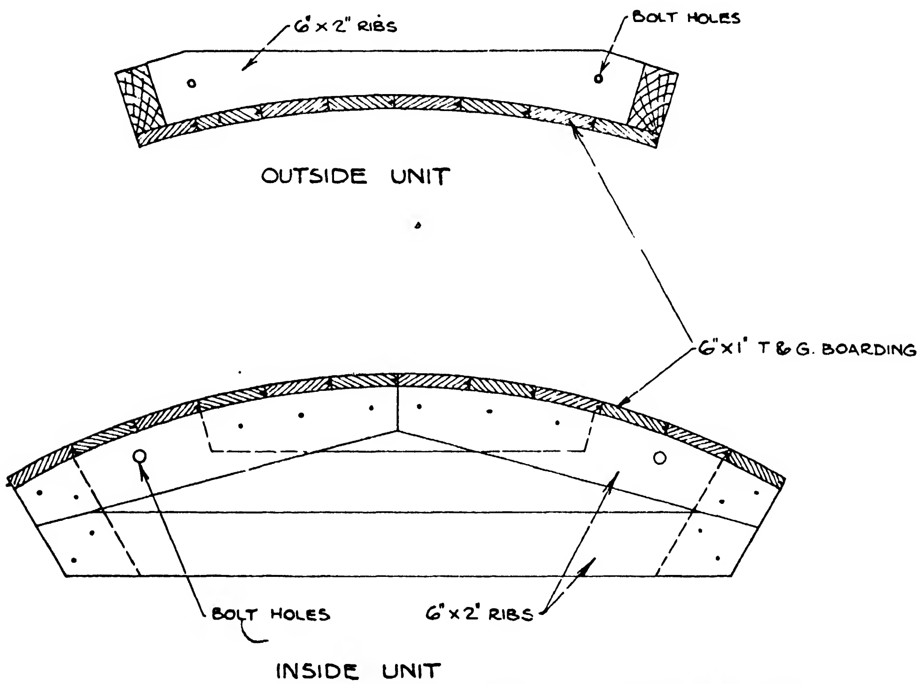


FIG. 360.—SHUTTERING UNITS FOR WALLS OF CYLINDRICAL TANKS.

of bolts passing through vertical end stiffeners. Each belt of shuttering is similarly connected by vertical bolts passing through holes in the ribs. All bolt-holes should be carefully set out so that the units can be assembled in any position without risk of the holes not registering. An additional aid to vertical alignment is obtained by extending the boarding about 1 in. below the lower rib and depressing the top boarding the same distance below the top of the upper rib, thus enabling two successive belts of shuttering to key into each other. The inner and outer units in any belt are kept the correct distance apart by distance-pieces attached as shown in *Fig.* 361.

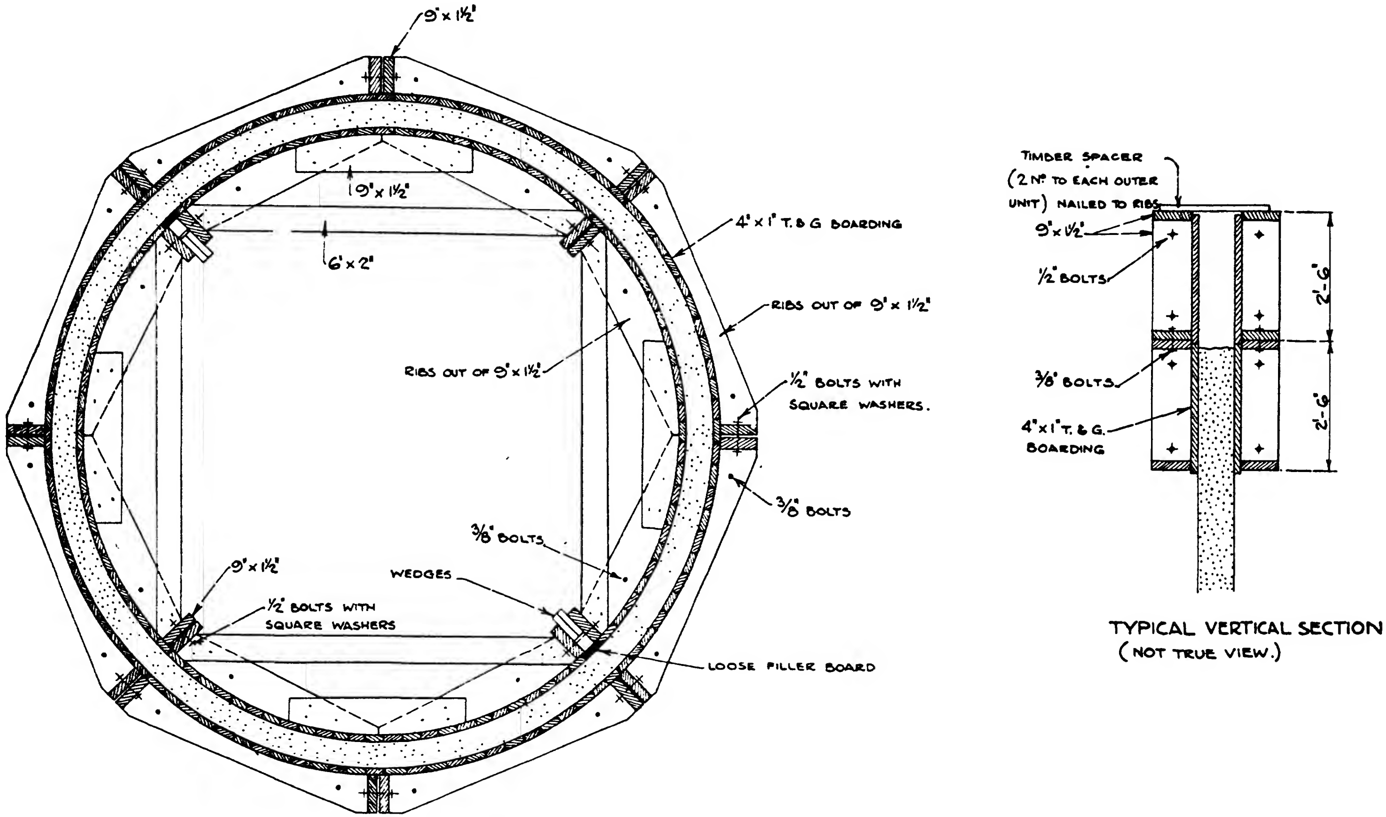


FIG. 361.—SHUTTERING FOR CYLINDRICAL TANK.

The outer units are easily freed and removed by withdrawing the bolts, but, to enable the inner units to be taken down, gaps into which wedges are driven when erecting the shuttering must be left and the space in the wall sheeting made up by a loose filling-piece. The first step in dismantling a unit is to remove the bolts and wedges. A pair of 1-in. diameter holes drilled through each rib makes it easier to lift them.

Fig. 362 shows an alternative method of constructing and interlocking units of shuttering for the walls of cylindrical tanks. The units are made from 7-in. by $1\frac{1}{4}$-in. ribs and 4-in. boards $1\frac{1}{4}$ in. thick with shot edges, the ribs extending so as to overlap the ribs in the adjacent units. Bolts passing through the overlapping sections secure the units.

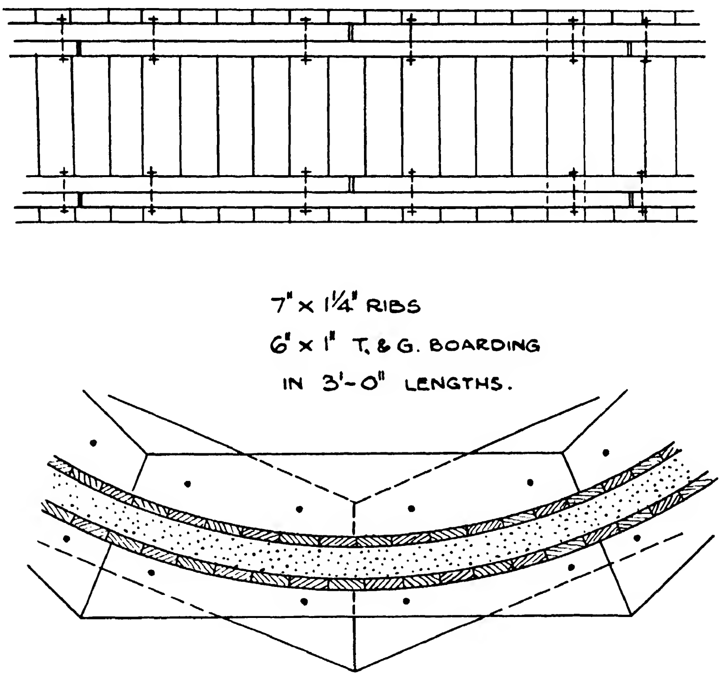


FIG. 362.—ALTERNATIVE SHUTTERING FOR CYLINDRICAL TANKS.

In deep tanks the walls are usually thicker at the bottom than at the top. This is most readily allowed for in the shuttering if the outer face of the wall is vertical. The external units can then remain unaltered throughout the height of the wall. If the inner units are constructed to the minimum diameter, that is the diameter at the bottom of the tank, and one or more filler boards are inserted around the circumference, the diameter of the belt can be increased by increasing the width of the filler boards for each successive lift. If the batter on the wall is considerable it may be found necessary to taper the filler boards to allow for the difference in diameter between the top and the bottom of the belt.

After use on one tank it should be possible to use well-constructed units on several similar tanks, small variations in diameter being made by altering the sizes of the filler boards and wedges. Standardised steel shuttering can be obtained for circular tanks and it may prove more economical to hire it than to construct timber shuttering if only one tank is to be built. For tall tanks the use of continuously-moving shuttering may be worth consideration; particulars of this type of shuttering are given in a subsequent section.

Tanks of large diameter may have plain walls or be provided with counterforts. In the latter case the wall panels, if a slight departure from true cylindrical form is not objectionable, can usually be built in straight lengths between the counterforts, a change of angle being made at each of the latter. The shuttering for a wall of this class would be similar to that for counterforted walls of rectangular tanks (see p. 469).

The walls of plain cylindrical tanks or reservoirs of large diameter can be built with panel units of shuttering similar to straight walls. If the units are made short they need not be specially shaped to the curvature of the walls, a

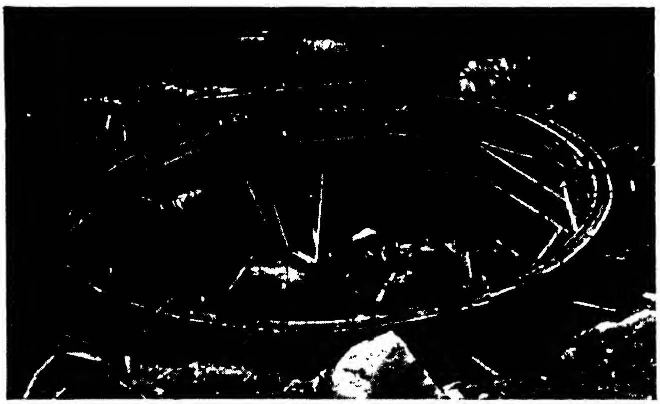


FIG. 363.—TIMBER SHUTTERING FOR LARGE DIAMETER TANK.

series of short straight lines often being satisfactory. This type of construction was used in building the 45-ft. diameter tank illustrated in *Fig.* 363.

In the case of the 320-ft. diameter reservoir illustrated in *Fig.* 364, the walls were divided by permanent expansion joints into sections 60 ft. long, and steel shuttering was used on both faces of the wall. Purpose-made steel moulds were provided for the splay at the bottom of the wall, and by providing wire ties between the shuttering on opposite faces little bracing was required. Sufficient shuttering was supplied for two 60-ft. sections, each set of shuttering being used eight or nine times.

The bottoms of tanks do not present any unusual problems in construction unless they are domed. *Fig.* 365 illustrates the construction of a small dome. The whole of the shuttering for the soffit of the dome and the whole of the reinforcement are shown in position. The shuttering for the upper face of the dome is partly erected, enabling the lower part of the dome to be concreted. The illustration shows the circumferential collars which act as templates at the top and bottom of the shuttering and the method of cutting the radial boards to allow

FIG. 364.—STEEL SHUTTERING FOR CIRCULAR RESERVOIR.

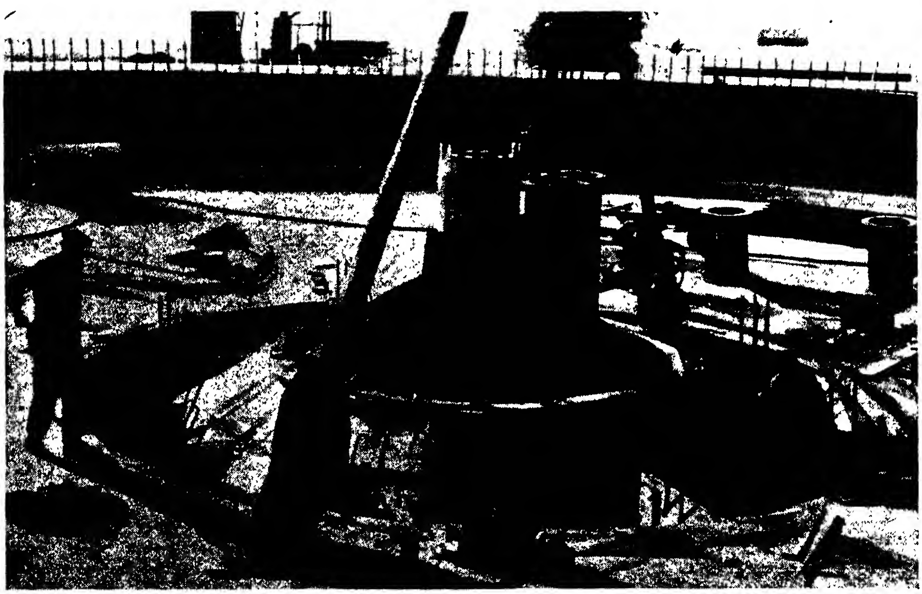


FIG. 365.—SHUTTERING DOMED BOTTOM.

for the changing diameter. *Fig.* 366 shows the completed shuttering for a domed bottom of a water tank, showing herring-bone arrangement of sheeting.

Internal shuttering for circular pits of small diameter or small holes in mass concrete can be constructed in a unit. Circular formers are made from 2-in.

FIG. 366.—SHUTTERING FOR DOMED BOTTOM OF WATER TANK.

or 3-in. timber, and on two or more of these narrow longitudinal boards are nailed with close joints as illustrated in *Fig.* 365. This method is suitable for work 1 ft. 6 in. to 4 ft. in diameter, the boarding varying from 1-in. by ¼-in. laths for the smaller diameters to 3-in. by ¾-in. lagging for the larger diameters.

Reinforcement.—In constructing circular tanks the recommendations for obtaining impermeable construction (p. 457) should be followed. Care must be taken to place the reinforcement exactly as shown on the drawings. This applies particularly to the circumferential bars. If these are in more than one length the laps should be staggered so that as few as possible occur at any vertical section. In no case should the laps in one layer of bars be vertically in line with the laps in the layer immediately above or below. If the contractor is free to arrange the reinforcement he should use small diameter bars at close centres rather than large bars at wider centres so as to save labour in bending the reinforcement. If the bars are $\frac{5}{16}$ in. in diameter or less they may be supplied in coils, in which case the material can be wound around the periphery of the tank in the form of a continuous spiral. The ring of vertical bars provides a template for the horizontal steel, the latter being wired to the verticals at the required spacing. Tanks up to 20 ft. diameter have been successfully reinforced by this method, while ¾-in. diameter bars in stock lengths of 30 ft. to 40 ft. have been used in a similar manner in conical tanks up to 40 ft. in diameter.

Rectangular Tanks.

Shuttering.—The walls of rectangular tanks are constructed on the same general lines as panel walls and retaining walls. Bolts are suitable for containers of dry materials, but wire ties are preferable if impermeability is required unless the inside of the tank is to be rendered. In the walls of shallow rectangular tanks ties or bolts can be avoided if an external means of support similar to that shown in *Fig.* 367 is adopted. The shuttering for the external face is strutted

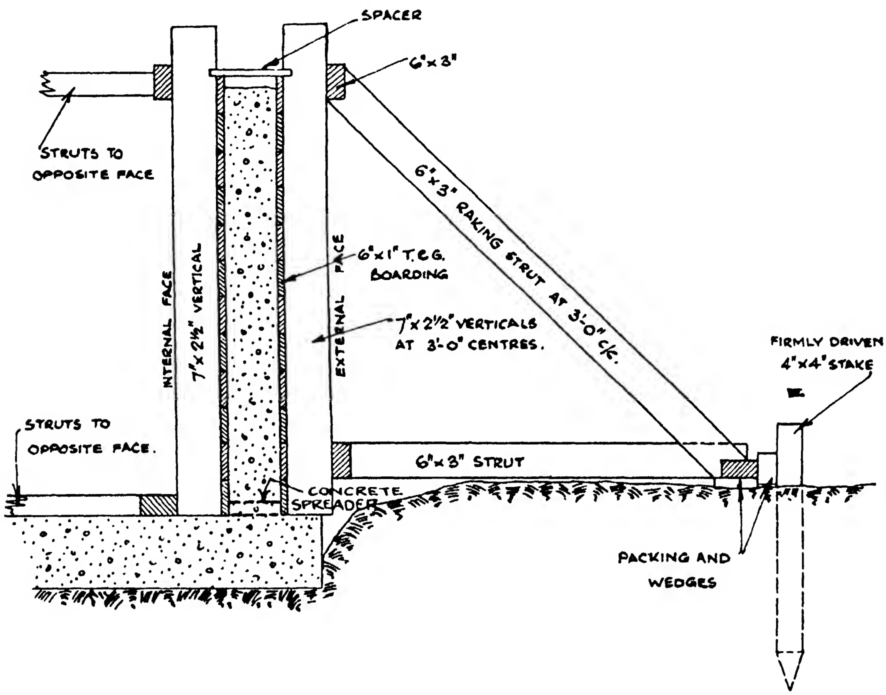


FIG. 367.—SHUTTERING FOR SHALLOW RECTANGULAR TANK.

from a longitudinal timber bearing against a series of stakes driven into the ground. If the tank is narrow the internal shuttering can be strutted off the opposite face, but if the width is so great as to prevent this, raking struts and longitudinal bearers similar to those on the outer face must be provided. In this case the bearers to the inner face should be of generous dimensions so that they will not deform if unsupported over considerable horizontal distances. Although their ends can be fixed in position by the corresponding bearers from the adjacent walls, intermediate supports are not readily arranged in wide tanks, owing to the fact that the concrete floor precludes the use of stakes. Stops formed by casting a short length of $\frac{3}{4}$-in. bar vertically in the floor provide intermediate support, the stops being subsequently cut off and the floor made good. The shuttering of opposite faces of the wall is kept at the correct distance apart at the top by a timber spreader nailed to the top of the sheeting; alternatively, the spacer can be attached to the tops of the vertical joists. Spreading at the bottom is prevented by the struts, and until the concrete is placed the two faces of the sheeting are kept apart by a concrete spreader which can be left permanently in position. A similar example of the shuttering for a section of a swimming-bath wall is illustrated in *Fig.* 368.

Wall shuttering with raking struts on the inside is illustrated in *Fig.* 369, which also shows a detail of the shuttering for a tank corner in which there is a horizontal splay at the junction of the walls. In this example both bolts and wire ties are used. In either of the examples shown in *Figs.* 367 and 369 the boarding for one face of the wall would be attached to the vertical members for the full height of the wall. If the concrete is to be placed in two or more lifts

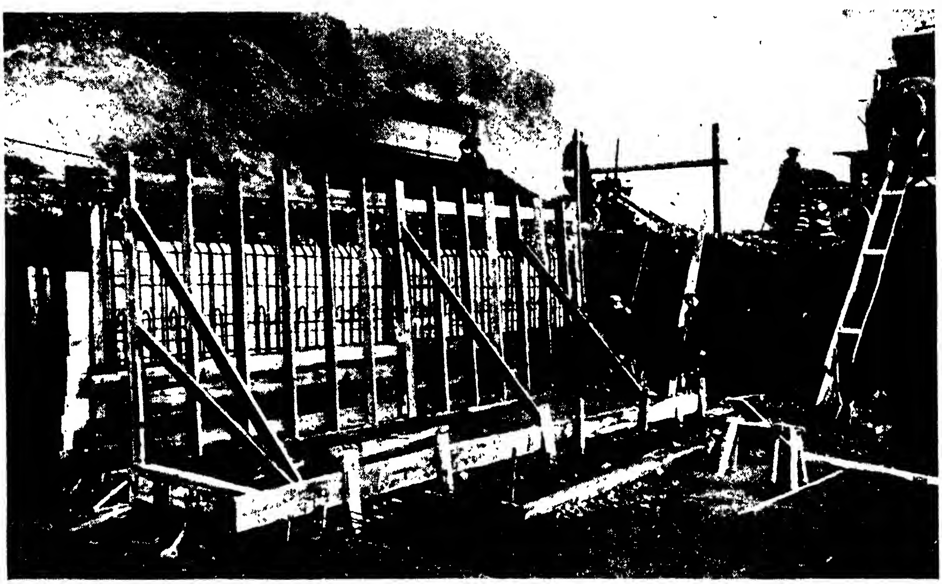


FIG. 368.—SWIMMING BATH WALL.

the other face would be sheeted for the first lift of concrete and the remainder boarded up as concreting proceeds.

Large rectangular tanks or reservoirs usually have counterforted walls; the details of shuttering for a typical counterforted wall are shown in *Fig.* 370. In this example bolts are used for tying in the sheeting on opposite faces, as it is intended that the tank shall be rendered. As the rendering will ultimately conceal the inner face, rough boarding is used on this side of the wall. The shuttering for the counterforts is erected for the full height of the wall and acts as a series of soldiers for guiding the panel units. The latter are made about 3 ft. deep and extend between two adjacent counterforts. The method of fixing these units is shown in *Fig.* 370. When a lift of concrete has been placed and hardened, the wedges are removed and each panel unit is slid upwards and wedged into position for the next lift. It is necessary to strut and guy the counterfort boxes to maintain them vertical.

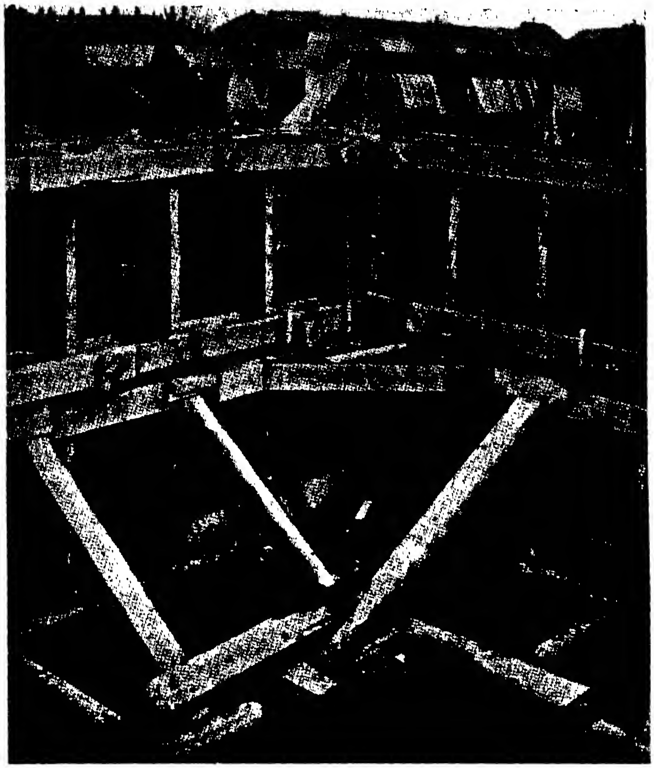


FIG. 369.—DETAIL OF TANK CORNER.

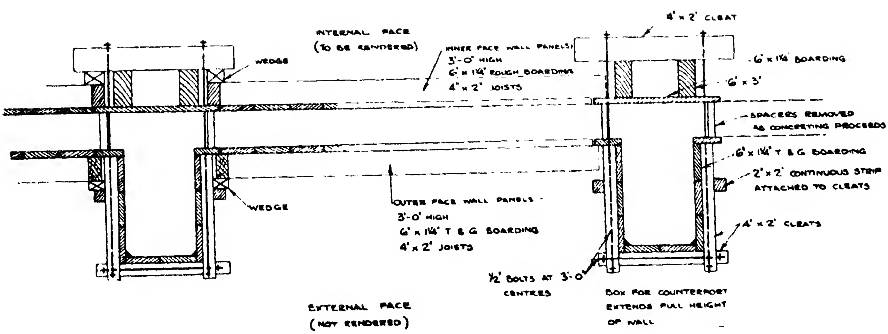


FIG. 370.—COUNTERFORTED WALLS.

SECTION III.—BUNKERS, SILOS AND CHIMNEYS.

Bunkers and Silos.

A bunker or silo may be a single compartment or may be formed by a nest of compartments, the bottom usually being in the form of an inverted cone or hopper.

Sloping Bottoms.—The slopes of the latter are usually of such a steepness that both faces of the bottom slab require shuttering. The shuttering for the underside is usually erected for the whole hopper in the first instance, the upper-face shuttering being built up as concreting proceeds. Since the hopper bottom is suspended from the walls or from beams along the top of the slopes, it is essential

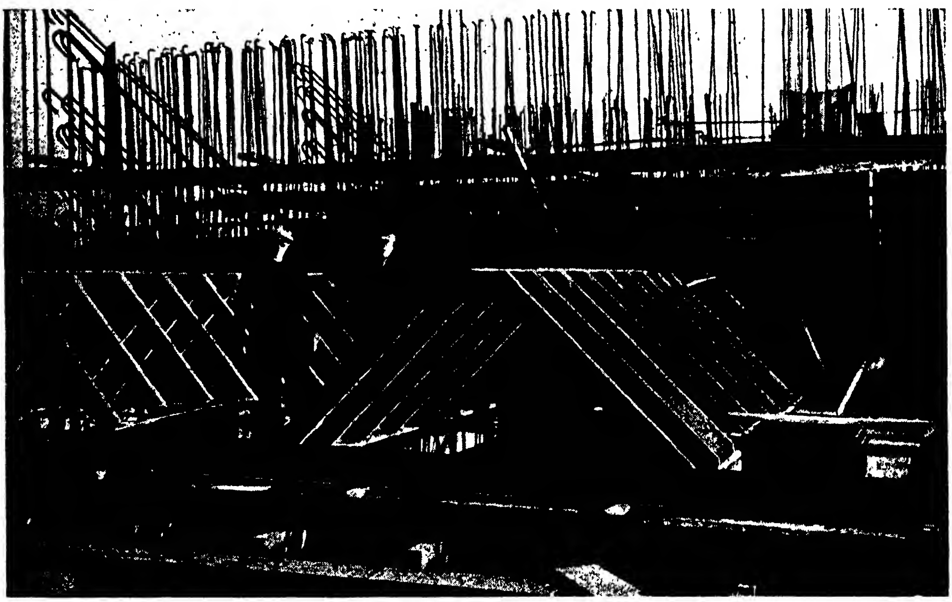


FIG. 371.—PROFILES FOR HOPPER BOTTOMS.

that the latter members should have been constructed and matured before the propping is removed from below the hopper bottom. How much of the walls should have been constructed and what age the concrete should attain before removing the props depend upon the method of design. Generally the wall concrete should have attained an age of 28 days, and at least a height of wall equal to half the distance between columns should have been constructed before the weight of the hopper bottom is transferred to the walls.

The first stage in the construction of hopper bottom shuttering is illustrated in *Fig.* 371 where the profiles for the bottoms are shown in position ready for the attachment of the sheeting, which may be either in timber or steel. The illustration is actually for a trough bottom, but this method is often used when a number of pyramidical bottoms are in line. The shuttering for the transverse slopes in the latter case are also supported on this framework.

With the conventional design of hopper bottom shown at (*a*) in *Fig.* 372, the relative amount of shuttering involved is above the average for normal slab construction and, owing to the pyramidical or conical shape, the joiner's work is more complex and is made more intricate by the introduction of corner splays.

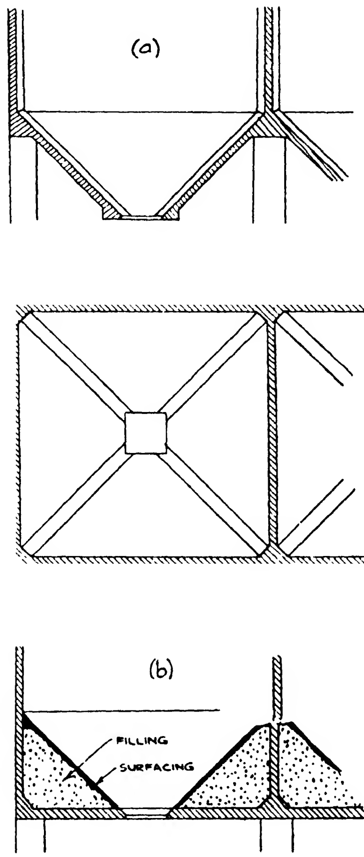


FIG. 372.—HOPPER BOTTOMS.

When good carpenters are scarce or their wage rates are high or in other circumstances it may pay the contractor to consider the alternative of constructing, with the client's approval, a flat-bottom container, making up the slopes subsequently as shown at (*b*). Although both the area of shuttering and volume of concrete may be increased, the greater ease of placing the latter and the much simpler shuttering may result in an overall saving. The wall shuttering, whether of panel type or continuously-lifting type, can be started at the outlet level instead of at the top of the slopes. The corner fillings can be a cheap concrete, say 1 to 8 breeze aggregate concrete, the fillets and surfacing being applied as a rendering or as an anti-abrasive, tiled, or other specified surface. If the filling concrete is made as dry as is workable, no shuttering is required for the sloping faces. An alternative method of construction is to pre-cast the hopper bottom with vertical suspension steel projecting. A hopper bottom of this type is shown in *Fig.* 373, being lifted into position before the walls of the bins are constructed.

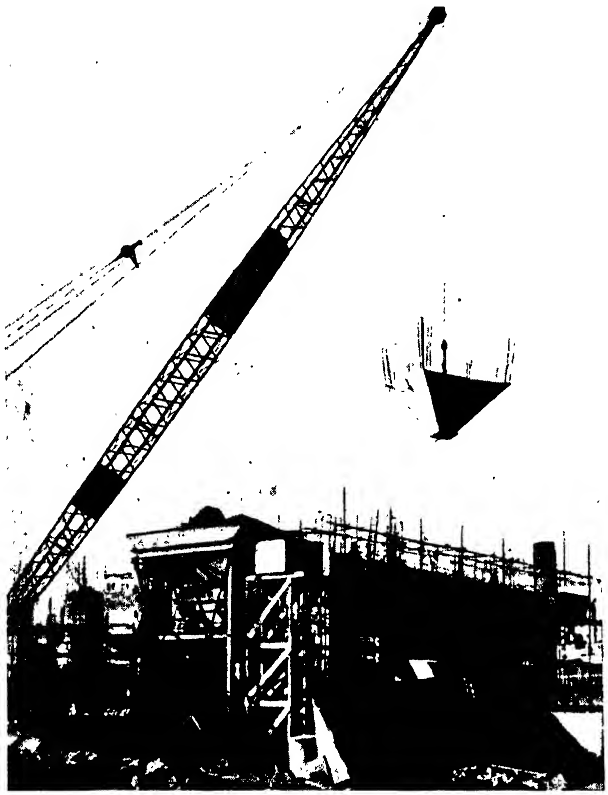


FIG. 373.—PRE-CAST HOPPER BOTTOMS.

Walls.—The construction of wall shuttering for bunkers usually follows the methods already described for circular and rectangular tanks, although the use of continuously-moving shuttering, as described later, is now common in the erection of the walls of silos and similar tall structures. When moving shuttering is not used the walls of bunkers or silos are constructed with panel shuttering of timber or steel.

For rectangular compartments a common type of timber wall shuttering consists of a series of panel units such as those illustrated in *Fig.* 374. A single lift is provided for the whole of the walls for the complete job or for a considerable section of the work. Each panel is about 2 ft. or 2 ft. 6 in. deep and is bolted at the top and bottom through the wall, a timber spreader being provided at the top, the hardened concrete in the section below acting as a spreader for the bottom. Attention is drawn to the method of constructing the splays and to the corner bracing, the latter being attached when the panels are in position. When the units are to be lifted, the lower bolts are withdrawn and the panel is slid upwards for the next lift. The lower bolts, when replaced, rest in the grooves in the concrete formed by the corresponding upper bolts and provide a support for the weight of the panel. Thus a series of bolt-holes is formed in the walls and provides a means of attaching bearers supporting the working platform at a convenient level for manipulating the panels and for the men

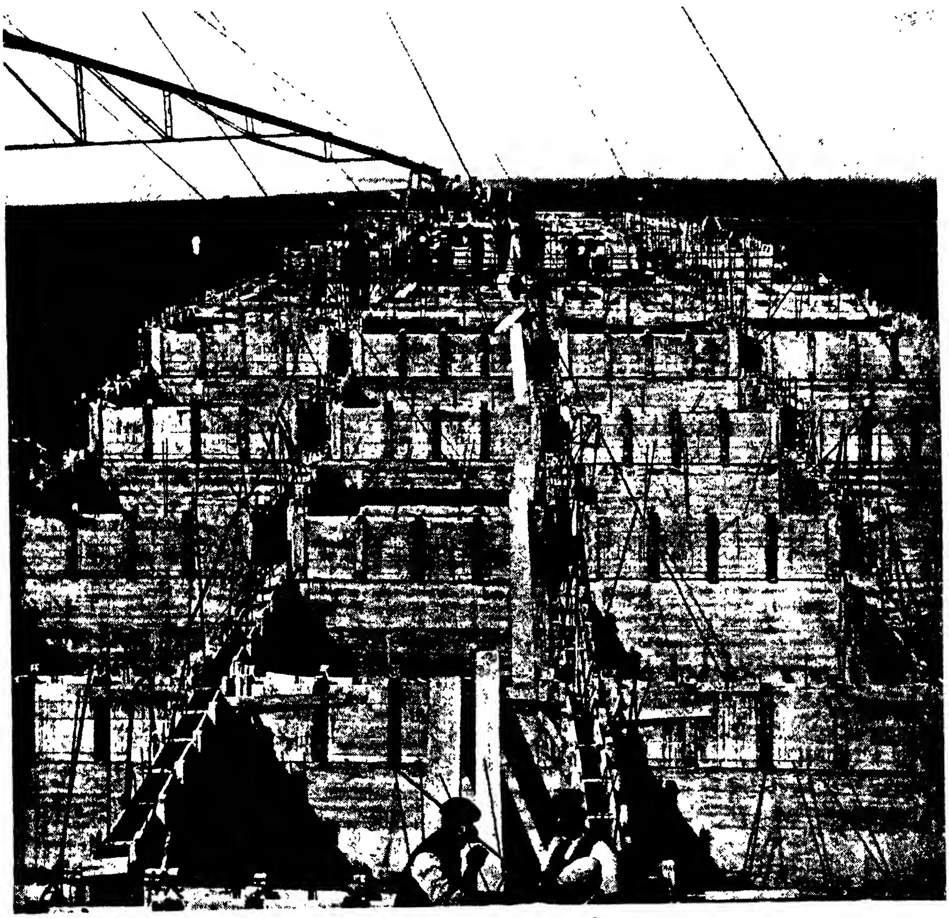


FIG. 374.—RECTANGULAR SILOS.

attending to steelwork and concrete. By this method about 2 ft. of wall can be constructed every two days, or each day if rapid-hardening Portland cement is used and temperature conditions permit. The extent of wall for which shuttering should be provided should be sufficient to keep the concreting plant fully employed throughout the working day. The work can be speeded up by multiplying the concreting units and providing additional shuttering. Once the successive stages of steel fixing, dismantling and re-erection of shuttering and concreting have been organised the construction proceeds smoothly.

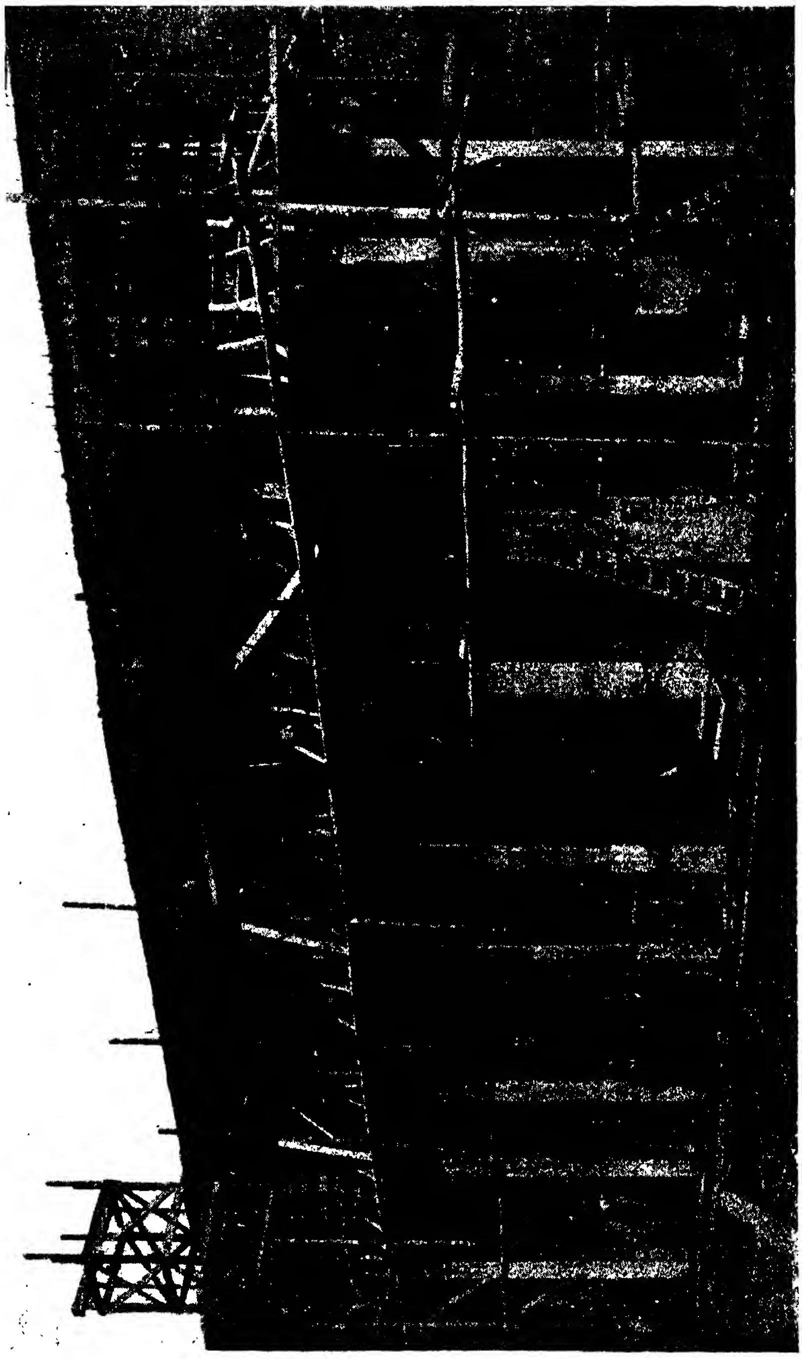


FIG. 375.—STEEL MOULDS FOR CIRCULAR BINS.

The vertical reinforcement is provided in short lengths, say 5 ft., in order that it shall present the minimum amount of obstruction to workmen and plant. The horizontal steel for each lift is fixed immediately before the panels of shuttering are raised.

If continuously-moving shuttering is not used for the walls of circular bins, the latter are often more conveniently built with steel moulds, timber shuttering being used for the supporting columns and bottoms. An example is illustrated in *Fig.* 375 where, for a battery of four circular bins, sufficient shuttering is provided for two lifts of the complete circumference of each bin.

Continuously-moving Shuttering.

Continuously-moving shuttering consists of a belt of double sheeting about 4 ft. deep extending around the complete periphery of the interior and exterior

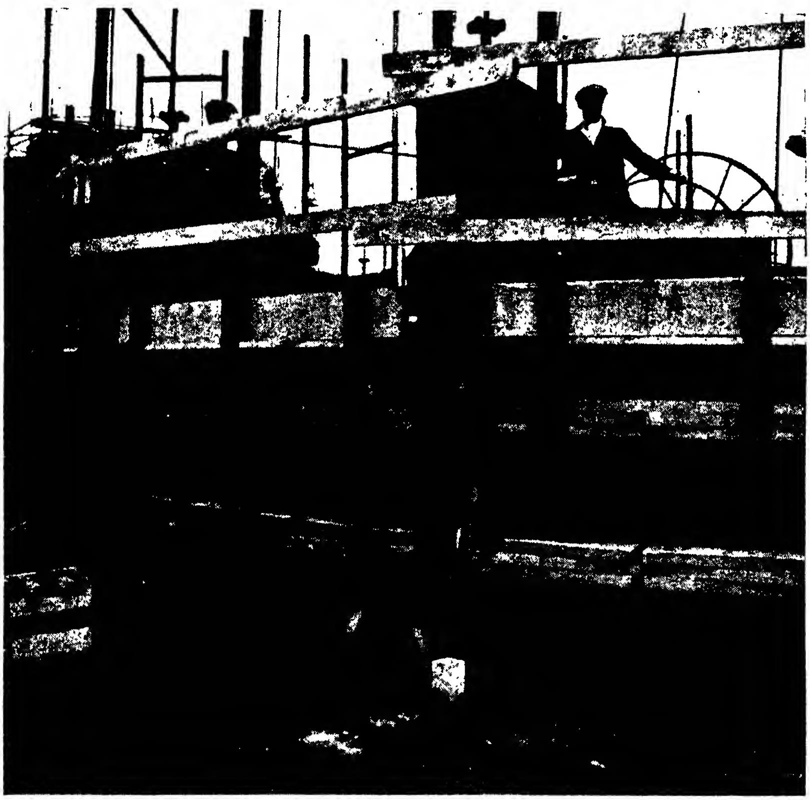


FIG. 376.—CONTINUOUSLY-MOVING SHUTTERING.

walls of the structure. The sheeting consists of vertical boarding attached to horizontal ribs which are suspended at intervals from timber yokes. The climbing operation is controlled by a number of jacks attached to the crosshead of the yoke and supported by a jaw that bites on a vertical rod. *Fig.* 376 illustrates the general arrangement of the shuttering, while a detail of a jack is shown in *Fig.* 377.

The jacks, which are placed at about 5-ft. centres (but not exceeding 7 ft.), are usually of screw type. The principles are the same for all types containing screw jacks. The jacking rods pass through the screwed sleeve of the jack, and at their lower end are buried in the concrete wall below the shuttering. Since the whole weight of the shuttering is transferred through the jacking rods to the portion of wall already constructed, these rods must have appreciable stiffness. They are usually about 1 in. in diameter and are made from steel containing 0·2 to 0·25 per cent. of carbon. These rods cost 25 to 30 per cent. more than ordinary mild steel rounds.

FIG. 377.—JACKS FOR MOVING SHUTTERING.

For the shuttering itself, two horizontal ribs formed by triple segmental timbers bolted together act as staves and templates. A suitable size of timber from which to cut the ribs is 8 in. by 2 in. Diagonal bracings as shown in *Fig.* 376 prevent racking of the shuttering between the jack centres. The thickness of the boards should be 1 in. to $1\frac{1}{4}$ in. For normal curved wall work a suitable size of sheeting is 4 in. by 1 in. The boarding when attached to the ribs should not be close-jointed, a slight space between adjacent boards permitting the swelling that takes place when the timber is continuously in contact with wet concrete.

The yokes should be substantially made, all members being bolted together and the whole frame cross-braced or otherwise strengthened laterally. The hole in the bottom transverse member of the yoke through which the jacking rod passes should not be larger than will just allow the rod to pass freely ; if it is too large a valuable point of lateral support to the jacking rod is lost.

Openings for windows, doors, beam connections, etc., can be formed by building timber frames into the thickness of the wall as shown in *Fig.* 376, enabling the shuttering to slide past without interruption. Within the boundary of the outer walls and over the whole area of the job, a working platform is constructed. This consists of timber boarding carried on joists supported on the top rib of the wall shuttering.

It is essential that the reinforcement should be designed in accordance with the design of the shuttering to ensure that the bars can be easily manipulated and fixed when the jacks and yokes are in position. It is advantageous if the laps in the vertical bars do not all occur at one level; by staggering these laps as well as the joints in the jacking rods, the operation of fixing vertical bars is spread evenly over the whole time the job is in progress instead of being concentrated at intervals throughout the height of the walls. A maximum length of bar of 10 ft. to 12 ft. is best suited for the vertical reinforcement, and a timber frame is built up off each yoke as shown in *Fig.* 377; the tops of adjacent frames being connected by a horizontal timber assists in preventing the vertical bars from drooping.

A working detail of a typical moving shuttering assembly is shown in *Fig.* 378, where A is the turnhead and B is a 2-in. diameter screw-jack having a $\frac{1}{2}$ in. pitch, the diameter at the bottom of the thread being $1\frac{3}{4}$ in. One complete turn of the turnhead corresponds to a lift of $\frac{1}{2}$ in. The clutch, C, is a rotatable cast-steel member fitting the lower end of which is a pair of hinged tool-steel jaws which bite into the 1-in. diameter jack-rod D under the action of two conical springs. The joists E and boarding F form the working platform supported on the walings N, to which also is attached the wall sheeting M. The walings are hung by bolts J from the yoke H to which is also attached the finishers' scaffold K and L.

The cycle of operations, commencing from the stage when the shuttering is filled with concrete, is to raise the shuttering by giving every jack a succession of quarter turns until the belt of shuttering has been evenly lifted 3 in. to 6 in. The horizontal reinforcement for this height is then placed in position and concrete deposited almost to the top of the shuttering. This complete cycle may take almost an hour. The sequence in which the jacks are operated must be such that the shuttering lifts evenly. One man can usually attend to four jacks, giving a quarter turn to each in the proper order and repeating the cycle a sufficient number of times to give the necessary total lift. All men operating jacks should work in unison, so that no portion of the shuttering lags behind or gets ahead of the general level. The cycle of operations, once commenced, must be carried on continuously day and night until the end of the job, for if the concrete is allowed to harden in the shuttering while the latter is stationary it is extremely difficult to resume movement and almost impossible to attain uniform lifting.

It will be appreciated that the use of continuously-moving shuttering involves careful planning to ensure that each operation is carried out in its correct sequence and without delay to succeeding operations. Thus reinforcement for the complete work should all be bent before commencing jacking. The amount of concrete in each lift of 3 in. to 6 in. should be determined accurately, and the mixing and distributing plant should be easily able to cope with this amount in the limited time available for placing. The concrete plant should be arranged as near the middle of the work as possible, and on extensive jobs two or more

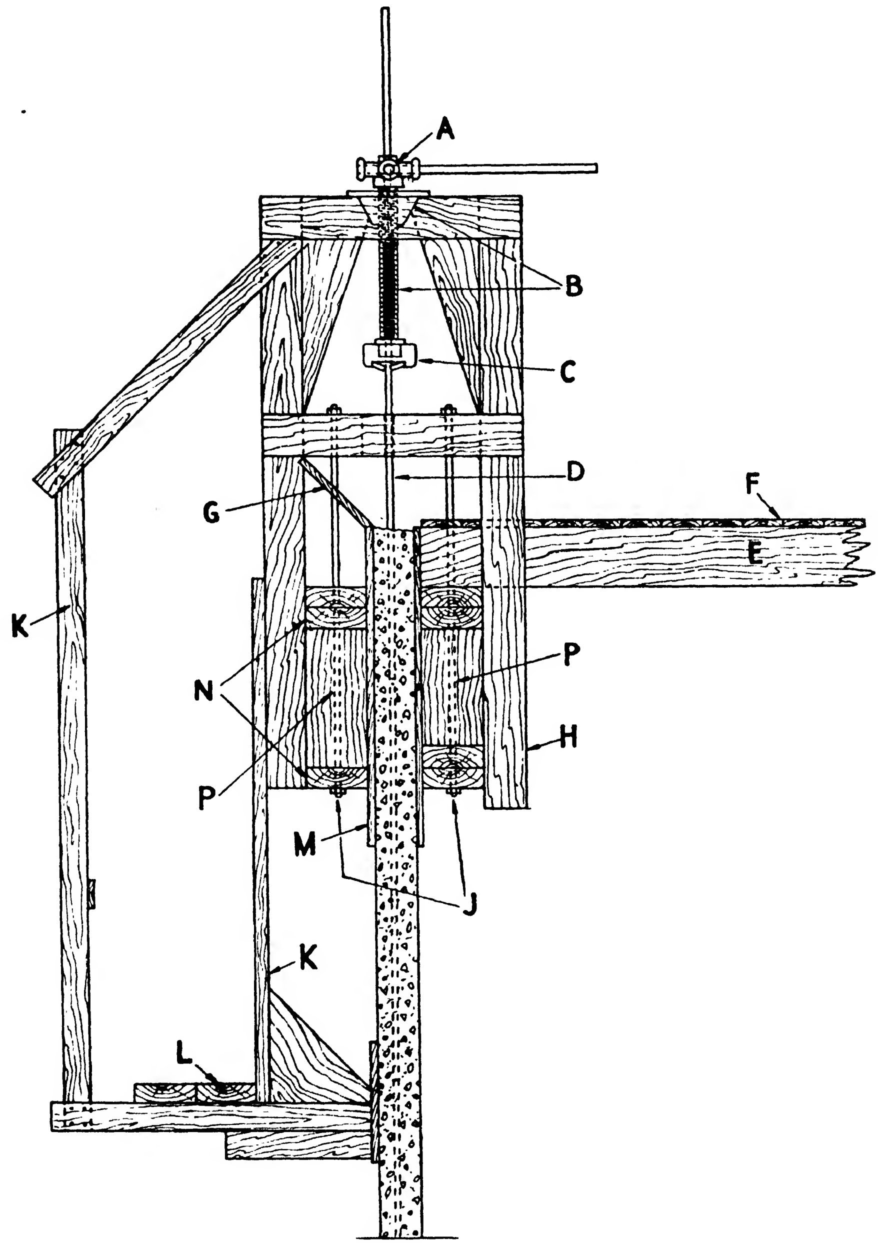


FIG. 378.—DETAILS OF MOVING SHUTTERING.

plants should be placed along the length of the work. In view of the seriousness of stoppages, duplicate mixing and hoisting equipments are advisable, and as a further safeguard one set may be electrically driven and the other driven by a petrol engine.

The steel-fixing and concreting gangs, who will be working simultaneously, should be arranged so that the latter follows the former without getting in each other's way. The workmen should be fully instructed in their various duties and the whole job should be under the charge of a man experienced in this type of work and whose organising and controlling abilities are well-established. As he will be unable to be on the job continuously throughout the progress of the jacking, it is equally important that a competent second-in-command should be available for reliefs.

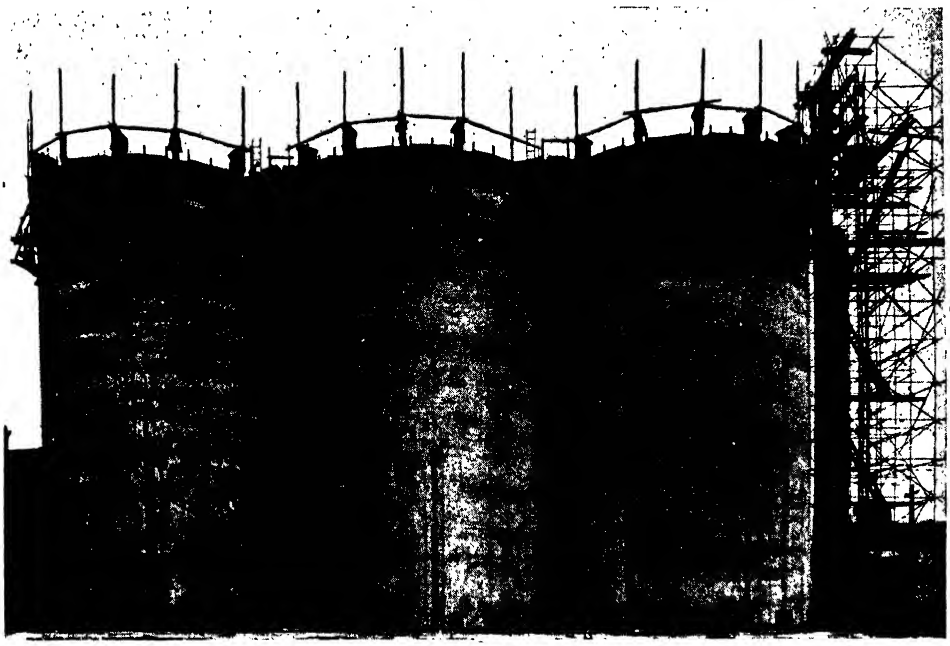


FIG. 379.—PLATFORM SUSPENDED FROM MOVING SHUTTERING.

The rate of vertical progress may be about 10 ft. in 24 hours, but lower at the beginning of the work and until the men have obtained experience. In bad weather the speed is also greatly retarded. Nevertheless, on chimney and bunker construction an average rate of 4 ft. to 5 ft. per day has been maintained through severe winter conditions, while in favourable conditions an average of nearly a foot an hour has been achieved.

Although continuously-moving shuttering involves the provision of such extras as efficient lighting equipment and a large labour gang, who will generally receive extra pay for night work and Sundays, the economy of this type of construction is undoubted when a structure of considerable height and constant horizontal cross-section has to be erected. It is thus applied to the construction of silos or deep bunkers, chimneys and similar structures. Even if the walls vary in cross section, for example, if they taper from bottom to top, the jacking arrangement can be modified to suit.*

Since there are usually few surface blemishes to make good, finishing is limited to brushing the external wall faces with cement grout and rubbing down with a wooden float. These operations can be carried out from a gantry suspended from the yokes. An external hanging scaffold of this type is illustrated in *Fig.* 379 where the shuttering is shown approaching the top of the work.

Chimneys.

The erection of tall reinforced concrete chimneys presents peculiar problems to such an extent that this work is often undertaken by specialist contractors.

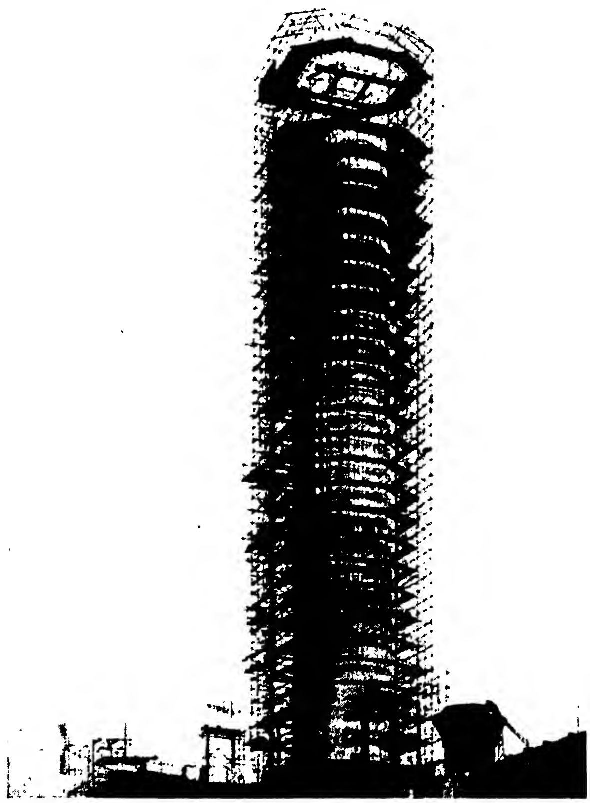


FIG. 380.—CHIMNEY CONSTRUCTION WITH EXTERNAL SCAFFOLDING.

Chimneys of 3 ft. diameter or less are usually correspondingly limited in height and are erected with the aid of an external scaffold. Since the quantity of concrete to be placed in each vertical lift of about 3 ft. is only small, this can be hoisted by hand-tackle operating within the shaft.

* For a description of a successful method the reader is referred to the paper read by Mr. G. P. Bridges before the Institution of Structural Engineers and reproduced in the "Structural Engineer," April 1936.

An internal scaffold replaces the external scaffold in the case of chimneys whose diameters afford a convenient internal working space, say, 6 ft. to 10 ft. in diameter. In such cases a power-hoist for workmen and materials can be installed to operate in the centre of the shaft. The scaffolding may be supported independently of the walls of the chimney, or a series of internal working platforms can be supported on putlogs passing through the walls.

For larger chimneys of modern power-houses an external scaffold is often erected. From the examples illustrated in *Figs.* 380 and 381 it will be seen that

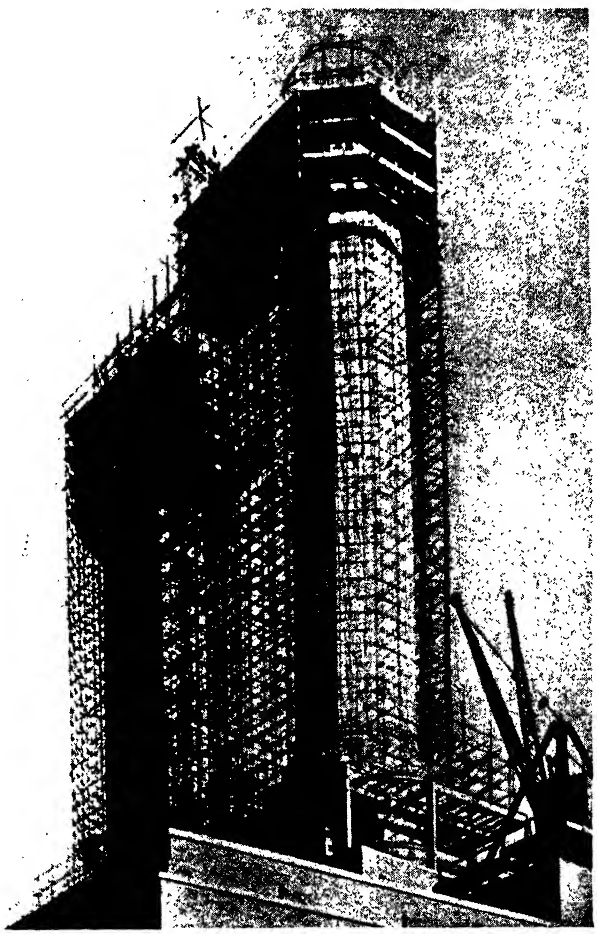


Fig. 381.—Twin Chimneys with External Scaffolding.

the amount of scaffolding required is very considerable. The adoption of continuously-moving shuttering eliminates the necessity of providing an external system of scaffolding, but may require an internal structure to accommodate the hoist and other equipment.

Timber and steel shuttering are both used in chimney construction. Where steel moulds are used for the main portion of the shaft, it is usually necessary to revert to timber for decorative portions. *Fig.* 382 shows the erection of a chimney using lifting panels of timber shuttering.

Continuously-moving shuttering, as already described (p. 475), is specially suitable for tall chimney shafts of uniform wall thickness and constant diameter,

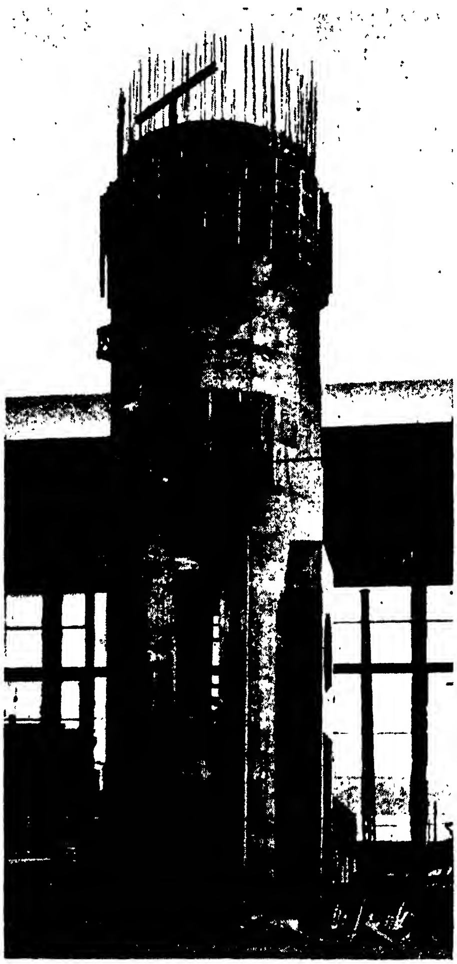


FIG. 382.—TIMBER PANEL SHUTTERING FOR CHIMNEYS.

and has also been successfully devised to allow for varying wall thicknesses and shaft diameters. With moving shuttering, rates of vertical progress of 8 ft. daily are not uncommon, working day and night, compared with 3 ft. to 4 ft. per day shift using ordinary panel construction. Moving shuttering requires more time for preliminary work, but when suitable reduces the overall time of a contract. The exposed nature of chimney structures renders their speed of construction very susceptible to inclement weather.

The proportions, consistencies, mixing and tamping of concrete in chimneys require special attention in order to obtain a dense concrete, as a dense skin is particularly essential to limit the effects of sulphur fumes on the free lime in the cement.

Many chimneys are provided with internal linings for a part or for the whole height and some specifications require the internal surface, where not lined, and the external surface for the topmost 20 ft. or 30 ft. to be coated with an acid-resisting paint. This operation can be carried out from internal and external scaffoldings where such have been erected. Where no external scaffolding is erected, the men can work from suspended platforms or cradles encircling the shaft and hanging from the top of the chimney. The application would proceed from the top downwards. Protective paints should not be applied until the concrete has dried out, that is, at least seven days after it has been deposited. The platform that is usually suspended from the yokes of moving shuttering for inspecting and finishing the outer surface cannot be used very conveniently for applying protective paints, as the concrete surface within working reach of such platforms at any given time will only be a few hours old. Thus an independent cradle must be provided and should not be overlooked in estimating the cost.

Provision for attaching internal linings must usually be made when constructing the shaft. If the lining is separated from the concrete by an air cavity, the lining may be tied back at intervals to the concrete by metal ties. The latter would be inserted in the concrete as construction proceeds and will present little difficulty whatever type of shuttering is employed. If the lining is directly in contact with the concrete, the former can act as the shuttering for the internal face and is built up a few courses in advance of concreting.

In many cases, linings are supported on corbel rings provided at vertical intervals of about 30 feet. Such corbels offer no constructional difficulties when the shaft is built with ordinary shuttering, but if the contractor proposes to use continuously moving shuttering, it is necessary to collaborate with the engineer so that the design will allow the corbels to be attached to the shaft at intervals around the circumference, instead of being constructed monolithically and continuously. The supports for the ring could be allowed for by leaving pockets as described in the foregoing section on continuously-moving shuttering.

CHAPTER XI

MISCELLANEOUS OPERATIONS

Pre-cast Work.

THE notes in this Section apply primarily to pre-cast work that may be incidental to a general building contract or to the manufacture of mass concrete blocks or monoliths. The extensive subject of pre-cast work as relating to such products as blocks, posts, ornaments, windows, lintels, etc., requires specialist treatment and is outside the present scope ; it is fully considered in " Manufacture and Uses of Concrete Products and Cast Stone," by H. L. Childe, and in companion books dealing solely with this subject.

Pre-cast work has advantages over in-situ work in certain circumstances. It is often easier to pre-cast members for awkward positions where concrete placing or the provision of supports for shuttering may be difficult. Also, where unskilled labour and only a minimum of supervision are available pre-casting may allow the manufacture of the members to be localised and carried out under more convenient conditions. The operations of pre-casting a number of units may also offer an opportunity of working under shelter for concretors and other trades on wet days and in the intervals of work on the major portion of in-situ structures. Stairs, steps, and other building details, girders for railway overbridges where the shuttering for in-situ work might foul the loading gauge (see Chapter IX), over-water work when it is desired that as much construction as possible should be carried out on land, and concrete ties behind wharf walls, are a few of the uses of pre-casting that may be profitably considered. Although in certain circumstances the claims of cast in-situ bearing piles should not be ignored, the pre-casting of bearing piles and sheet piles offers obvious advantages. Pile making has been considered in Chapter VII.

It is essential that when it is proposed to pre-cast any part of a structure the design should have been made with this in view, and consideration should be given to the weights of the finished units relative to the capacity of available lifting tackle. For the latter purpose, a round figure of 15 cu. ft. of concrete per ton can be taken when estimating the finished weight of a reinforced concrete member, although with a dense concrete, heavy aggregate, and much reinforcement, as little as 13 cu. ft. to the ton has been obtained in pre-cast concrete beams. When casting the units, provision should be made for lifting by embedding mild steel eyes in the upper face of the member. These eyes may be $\frac{3}{4}$-in. or 1-in. reinforcing bars, bent to the shape of a U, with the two arms buried in the concrete. Alternatively gas-pipe ferrules inserted horizontally in the member

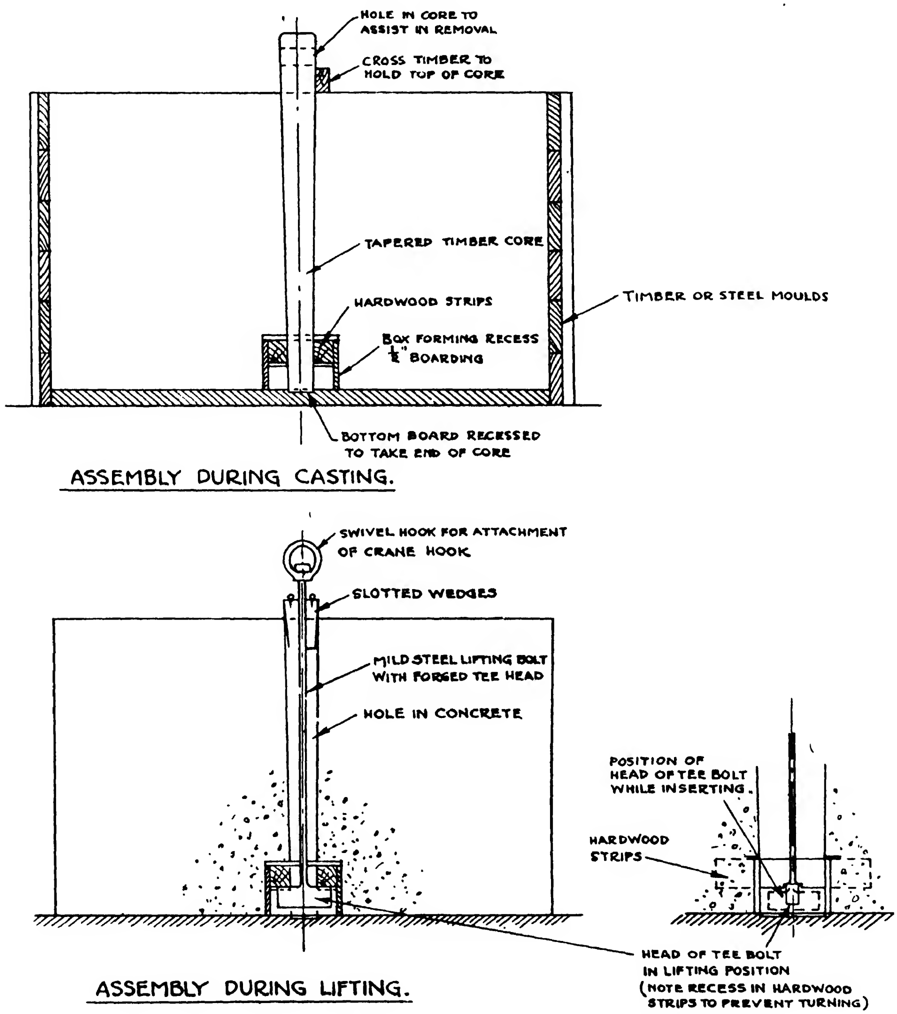


FIG. 383.—LIFTING TACKLE FOR PRE-CAST CONCRETE BLOCKS.

and projecting therefrom as described for piles will serve the purpose. If eyes or ferrules are not provided and it is proposed to lift the member by means of chain slings, timber packings should be placed between the chains and the concrete to preserve the latter from superficial damage. In the case of heavy units the points of lifting should be approved by the engineer. If the member is of uniform concrete section throughout, with a constant area of reinforcement, the lifting points specified for piles can be adopted, as these will produce minimum bending moments.

Where the reinforcement is not symmetrically disposed in such units as lintels or slabs, the units should be plainly marked when they are cast to show which face is to be placed uppermost in the finished work. Some specifications call for lintels to be made at least six weeks before they are built into the work and require loading tests to be made before they are incorporated in the structure.

Pre-cast plain concrete blocks are common in bridge piers, jetties, and similar massive construction. With such blocks the design and arrangement of lifting devices is important. *Fig.* 383 shows a method of forming a hole for lifting heavy blocks with a tee bar. In this example the hole extends right through the block, but in deeper units it is sufficient if the penetration is 2 to 3 ft. deep. Steel plates may be substituted for the hardwood strips, but some form of bearing cannot be dispensed with except with very light blocks, when the tee head can bear directly on the concrete. If the blocks have to be placed to a batter, the position of the lifting hook can be arranged off centre so as to cause the block to tilt when lifted. A simple calculation or graphical construction will show at what point to lift the block to obtain the desired amount of tilt.

When a large number of blocks has to be cast, steel moulds may be more economical than timber. These are constructed from thin gauge steel plates stiffened with light angles, any necessary chamfers, grooves or other markings being formed by attaching hardwood fillet pieces by screws to the steel units.

In the case of big blocks, a lean mix is often used for the body of the block and a richer mix for the facing. For example, the body may be in the proportions of 1 : 3 : 6 with a 1 : 2 : 4 facing, or the body may be in the proportions of 1 : 8 with a 1 : 4 facing, the two concretes being placed simultaneously to ensure a good bond. Specifications for large blocks usually stipulate that every 8-in. or 12-in. depth of concrete should be tamped as it is placed. If a special facing is used it should not be thinner than 1 in. If a coloured or other special mixture is to be used for one face of the block, whenever possible this should be cast at the bottom of the mould. If a special mixture is to be used on more than one face, it may be mixed to a stiff consistency and placed against the sides of the mould a few inches at a time and followed by the wetter " body " concrete, the facing being kept ahead of the body ; alternatively, separating plates as described for walls in Chapter VIII may be used.

Connecting to Existing Work.

When new concrete work is to be joined to existing concrete the procedure described for construction joints in Chapter VI should be followed. The operations of cleaning the old surface is of great importance if a good bond between new and old concrete is to be effected. When concrete is to be bonded to existing brickwork, the bond can be improved if the joints between the bricks are scraped out ¼ to ½ in. deep.

When vertical work, such as walls, is built up before horizontal work, such as an intermediate floor, is constructed, it is necessary to leave chases for slabs and pockets for beams. Recesses in concrete walls can be formed as illustrated in *Fig.* 384 ; this method is applicable to work such as constructing an intermediate floor supported on the retaining walls of a two-story basement. The chase or pocket is conveniently formed by lightly attaching to the wall shuttering a piece of ¾-in. steel mesh. The concrete placed around the mesh should be

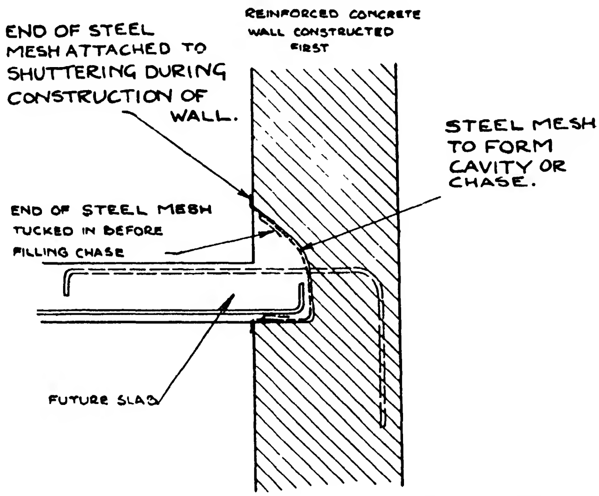


FIG. 384.—CHASES OR POCKETS IN CONCRETE WALL.

fairly dry to prevent loss of mortar through the meshes of the netting. When the shuttering is struck, the mesh will remain in position, there being no need to remove it; in fact it would be very difficult to do so. Before concreting the

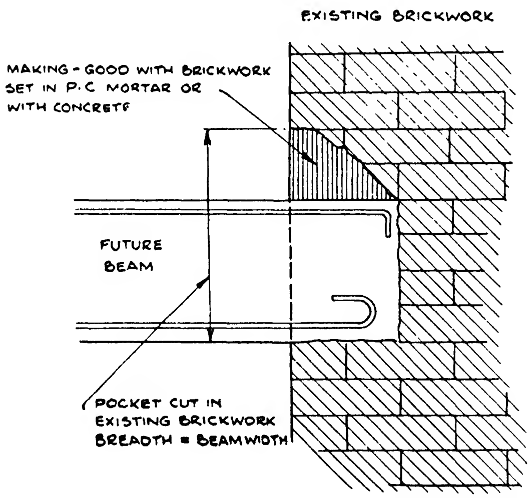


FIG. 385.—BEAM POCKETS IN EXISTING BRICKWORK.

slab and consequently filling the chase, the projecting ends of the mesh should be tucked into the chase or cut off to prevent rust stains appearing on the face of the completed work. This method of forming a chase is more convenient than timbering, leaves a rougher surface as a key for the subsequent concrete, and the mesh is more easily adapted to accommodate vertical reinforcing bars passing through the chase. Chases should not be formed or cut in existing work without the assurance of the engineer that the strength of the work will not be impaired or without taking precautions to ensure stability.

When concrete work is supported on brickwork, the pocket or chase should be shaped as in *Fig.* 385 as this allows the concrete at the end of the beam or slab to be well rammed into position and the assembly of the reinforcement may be facilitated. The triangular cavity in the brick wall should be made good by packing with concrete or by brickwork set in cement mortar. In beam pockets cut in existing brickwork the vertical sides should be made as smooth as practicable to ensure the correct side cover being given to the reinforcement.

Marine Construction.

More than ordinary care to ensure high grade workmanship and in the selection of materials is required in connection with marine construction as defects may arise from a number of causes, primarily chemical attack and corrosion of reinforcement resulting in disintegration. When concrete is to be immersed in sea-water provision should be made to provide resistance to attack by the magnesium and sodium sulphates present in the water, the attack being more pronounced in warm and tropical waters.

The concrete should be dense and impermeable ; there should be not less than 550 lb. of cement in each cu. yd. of set concrete in general work or less than 650 lb. per cu. yd. in work within tidal range or when placing under water is unavoidable. Hard aggregates should be used and ample cover given to the reinforcement. The concrete should be properly cured in air or in fresh water where practicable. Excessively wet or very dry mixes should be avoided and uniformity of mix and consistency should be aimed at. The number of day's-work or other construction joints should be reduced to a minimum and each should be carefully cleaned and prepared before continuing concreting. Joints should be avoided if possible in work between low and high tide levels, where deterioration is usually most evident. Where joints are unavoidable and the concrete is placed in batches between tides, the surface of the wet concrete should be protected against washing by canvas securely attached to the shuttering. The net labour cost for placing only for concrete placed between tides may be more than double that for normal work, including the cost of the necessary protective measures.

Aluminous cement concretes are more resistant to sulphate attack than are Portland cement concretes, although the hardening of the former may be retarded when in contact with sea-water during the early maturing stage. Some regulations, therefore, limit the amount of the alumina content to 8 per cent. when the concrete is in such positions as will necessitate hardening in the presence of sea water. Aluminous cement is very suitable for pre-cast work or work done in the dry within cofferdams.

The addition of trass or treatment with oxalic acid or fluorides also offers some measure of protection, as do natural bituminous coatings (and to a less extent tar derivatives) if renewed at intervals. Some authorities contend that the admixture of powdered sodium silicate has no advantageous effect on resistance to attack by sea-water. A recommended quantity of trass is 40 per cent. by weight of the cement, while the addition of one part of calcined gaize (which contains a large proportion of active silica) to two parts of cement clinker is reported to be effective.

Cracking of concrete in marine construction may be due to over-stressing due to ordinary bending, especially if the concrete surface has been corroded by sulphate attack, or to excessive shrinkage, or careless handling of pre-cast members. Once cracks have been formed the water may gain access to the reinforcement and result in consequent expansion causing the concrete cover to spall off. Spalling may also be caused by the freezing of water entering the surface pores or cracks. Disintegration may also result from blows from vessels or by abrasion from shingle or ice. Obvious precautions against these damaging actions include the provision of ample cover of concrete over the reinforcing bars (say not less than 2 in.), dense concrete, and protection by timber rubbing pieces, piles, fenders or similar attachments at points liable to external blows or abrasion.

Observations made in earlier chapters in connection with cofferdams, culverts and sheet pile walling, together with those in the present chapter on pre-cast blocks, have a bearing on the practical side of marine and river work. Much of this class of construction is carried on piles, the permanent level of the heads of the piles being below high water-level. It is sometimes advantageous and economical to cast the piles long enough to project above high water, thus providing a support to whatever temporary staging is required until the end of the job. When the work is completed the excessive length of pile can be cut off. With such an arrangement it is possible to continue driving piles at all states of the tide and the piling stage can be conveniently carried on the piles already driven.

Permanent Joints.

The growing appreciation of the necessity of introducing permanent joints in reinforced concrete construction to allow for shrinkage, temperature changes, and settlement has led to the design of various types of joints to suit different conditions. Joints in retaining walls, reservoirs, swimming pools, long buildings, bridges, roads, and floors laid on solid ground are usually fully specified by the engineer, who should describe each operation. It is essential for the success of the joint that its function should be understood by the constructor and scrupulous care should be taken throughout every operation. Joints are usually specified in floor slabs laid directly on the ground and covering large areas. The positions of these joints are preferably determined from considerations of the amount of slab that can be concreted in one operation, a joint being formed at the end of each period of concrete placing. Joints should also be placed where a marked reduction in width of the slab occurs, since such restrictions lead to weak sections for resisting contraction stresses. The simplest type of joint, such as occurs between panels of slabs laid directly on the ground, may be a dry joint, the stopping-off board at the end of one period of concreting being removed and a strip of waterproofed paper laid against the edge of the slab before proceeding to concrete the adjacent panel. In place of waterproofed paper, a strip of tarred paper, felt, bitumen sheeting, or similar material may be used, or the face of the concrete may be treated with bituminous paint. In these instances the two concrete faces are only separated by the thickness of the inserted strip. Sometimes a gap of $\frac{1}{4}$ in. to $\frac{1}{2}$ in. in width is left between adjacent panels, these gaps being left open or filled with asphalt or sand and asphalt. Bituminous filling is preferable when there is any likelihood of damp finding its way upwards through the joint. Where asphalt or similar fillers are used the concrete should be allowed to dry out before applying the filler. The edges of joints on the upper surface of floor slabs subject to heavy traffic should be protected by steel angles as illustrated in *Fig.* 386. These angles are built in when the slab is concreted.

It is essential that joints in roofs should be watertight and copper-strip waterbars, similar to those described later for reservoir walls, are sometimes specified. Normally a less expensive type of joint is found to be satisfactory. In each of the alternative designs illustrated in *Fig.* 387, the joint is filled with a soft-grade bitumen or mastic. To prevent fracture of the asphalt over the

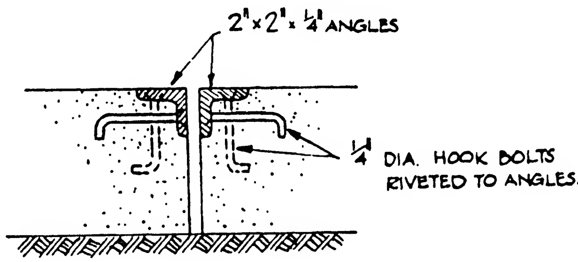


FIG. 386.—PROTECTION OF FLOOR JOINTS.

joint, a piece of canvas is laid on the slab. This reinforces the asphalt and, by reducing the adhesion to the concrete, increases the flexibility of the asphalt over the joint. The paper lining at the faces of the joint may be replaced by bituminous joint filler to allow for expansion, which may be greater in the roof than in other parts of the building. The line of the joint will appear on the ceiling below unless it is concealed by arranging the joint over a wall or by constructing a mock beam below the joint as illustrated in one of the designs in *Fig.* 387. The plaster on ceilings must be jointed if the slab above is jointed, and this is most conveniently done by separating the two panels of plaster by a wooden strip Rawplugged to the concrete. This is also shown in *Fig.* 387.

Joints are frequently provided in the stems of retaining walls at intervals not exceeding 60 ft. along the length of the wall. The spacing depends on the height of the wall. More frequent joints are required in low walls with thin stems, where it may be sufficient to provide a simple dry joint. One section of the wall would be completed with a vertical stopping-off board, and on removal of the latter a separating layer of waterproofed paper would be applied to the vertical edge of the wall stem before concreting the adjacent section. The principal objection to this type of joint is that any unequal deflection or tilting of one section compared with the adjacent section will show at the joint. Keyed joints are therefore preferable for walls more than 4 ft. in height. The key is formed as shown in *Fig.* 388 by nailing a shaped batten to the stopping-off board.

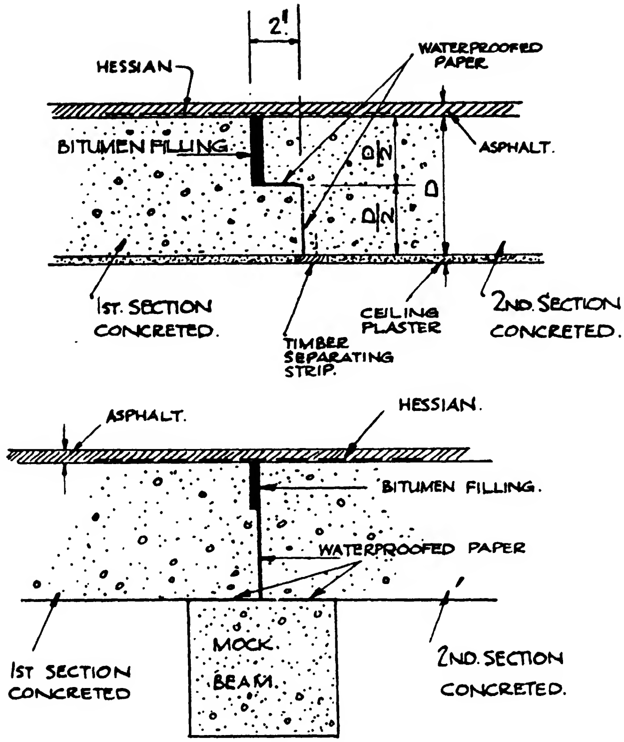


Fig. 387.—Joints in Roof Slabs (Alternative Designs).

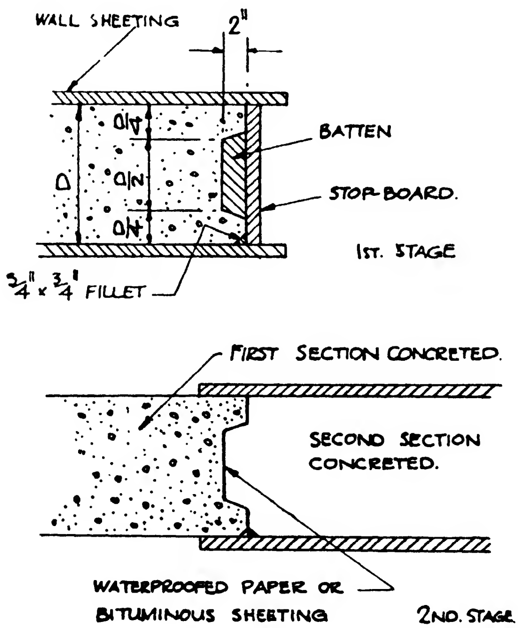


Fig. 388.—Joint in Retaining Wall.

The batten should be well oiled so that it can be readily removed and leave a clean cut groove. Before the next section is concreted, strips of tarred or greased paper, or bituminous sheeting, are inserted against the vertical edge of the concrete to prevent the surfaces from adhering. This construction also reduces percolation of moisture through the joint. The formation of a small **V** on the exposed face of the joint improves the appearance by obviating the ragged line which may be produced if the edge of the concrete is broken while stripping the shuttering from the first section of the wall.

The type of joint illustrated in *Fig.* 388 allows contraction of the wall-stem due to shrinkage and fall in temperature. A rise in temperature causing an expansion not greater than the shrinkage is also allowed for, and it is found in practice in most cases in temperate climates that this provision for movement

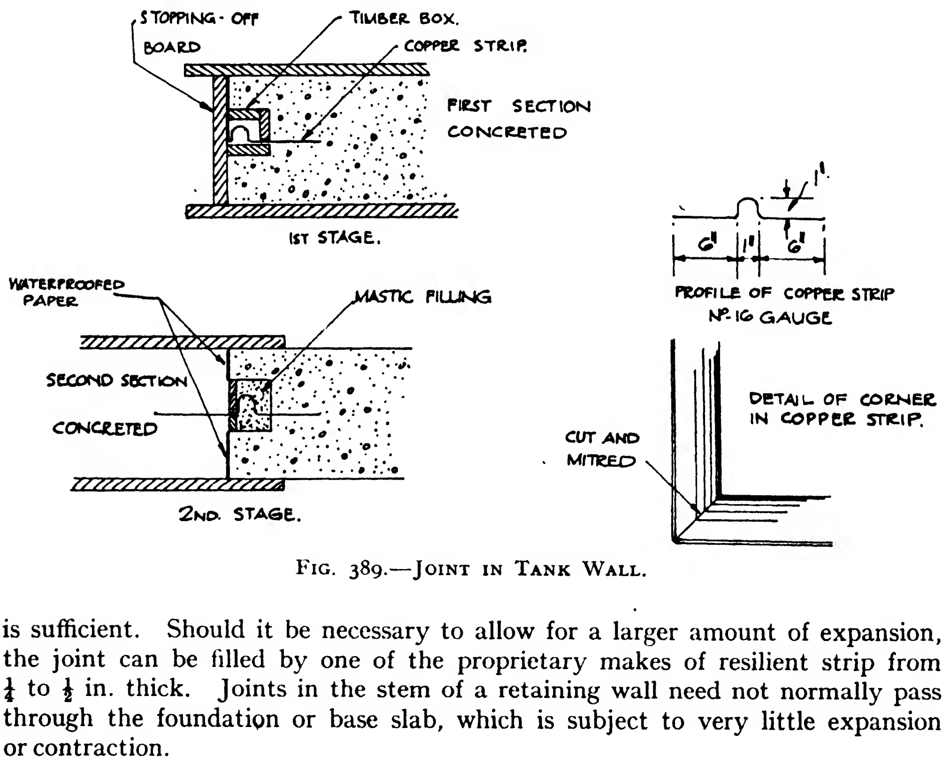


FIG. 389.—JOINT IN TANK WALL.

is sufficient. Should it be necessary to allow for a larger amount of expansion, the joint can be filled by one of the proprietary makes of resilient strip from $\frac{1}{4}$ to $\frac{1}{2}$ in. thick. Joints in the stem of a retaining wall need not normally pass through the foundation or base slab, which is subject to very little expansion or contraction.

A suitable joint for the walls of tanks, reservoirs, swimming baths, and similar water-containing structures incorporates a waterbar in the form of a copper strip, usually No. 16 gauge, shaped as shown on *Fig.* 389 to allow movements to be taken up in the wall. The strip is held in position between a divided stopping-off board while the first section of the wall is concreted, the loop in the strip being encased in a wooden box. Alternatively, the stopping-off board can be in one piece, with the projecting end of the strip bent back against the face of the board as illustrated. The strip is straightened out upon removal of the stopping-off board, after which the timber box can be extracted. Waterproofed paper is applied to the face of the concrete of the first section, thus separating the concretes placed in the first and second sections. It is preferable to fill the cavity around the loop of the strip with a soft mastic to prevent dirt, water, or other undesirable materials collecting in the joint. This is most conveniently done after extraction of the wooden box and before concreting the second section. Where appreciable expansion is anticipated, resilient bituminous sheeting should be used in place of paper. If the copper strip cannot be obtained in a sufficiently long piece to extend the full height of the wall, one or more standard lengths should be soldered together. Where the vertical section of the joint is bent into the floor, it is necessary to cut and mitre the loop section of the strip, sub-

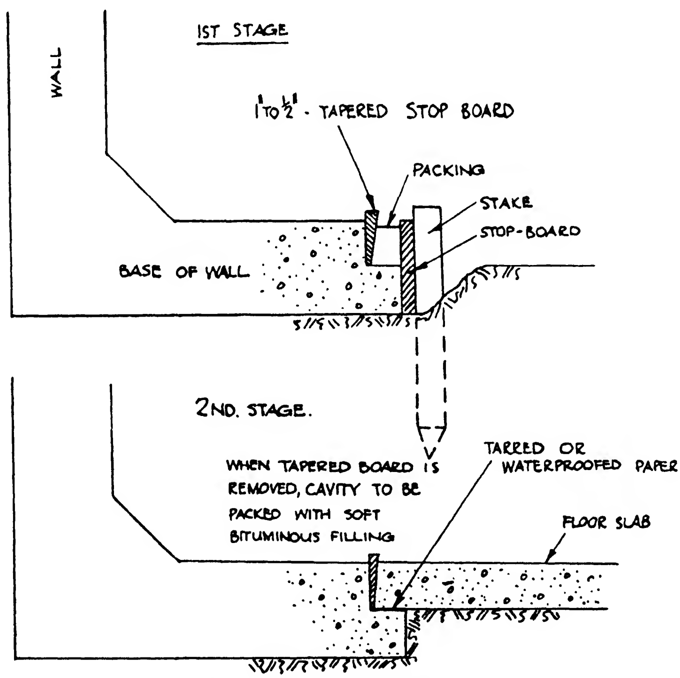


FIG. 390.—JOINT IN RESERVOIR FLOOR.

sequently soldering the cut portion. The loop should be kept on the inside of the bend as shown in *Fig.* 389, otherwise a cover strip is required to make a satisfactory joint. All joints in the copper strip should be carefully made to ensure that it will act as a complete water-bar.

In the usual sequence of reservoir construction the wall-base and stem are constructed in advance of the floor. It is therefore advisable to make a permanent joint between the wall-base and the floor, since an ordinary construction joint would be of relatively long standing and difficult to make good. A suitable joint is shown in *Fig.* 390, where a tapered stopping-off board is used against the upper part of the wall base. This board is left in position until the floor is laid. It is advisable to ease it away from the base slab before depositing the concrete in the floor in order to prevent it sticking and becoming difficult to remove. The horizontal portion of the joint is covered with tarred or greased paper or bituminous sheeting. When the floor concrete has hardened, the tapered board is removed and the joint filled with a suitable material. The material used for joint filling is important as it must be plastic at all temperatures and not harden with age or when in contact with water. It must also effectively adhere to the concrete on each side of the joint. There is some advantage in permitting reinforcement to cross the joint, the tapered board being slotted accordingly, because this presents unrestricted opening of the joint and riding of the floor slab relative to the wall base. The presence of a joint, however, gives sufficient freedom to avoid uncontrolled cracks. Joints across the floors of reservoirs can be constructed in a similar manner.

Similar joints to those provided in large reservoirs are necessary in swimming-pools, although it is not necessary to provide joints in medium size pools if they

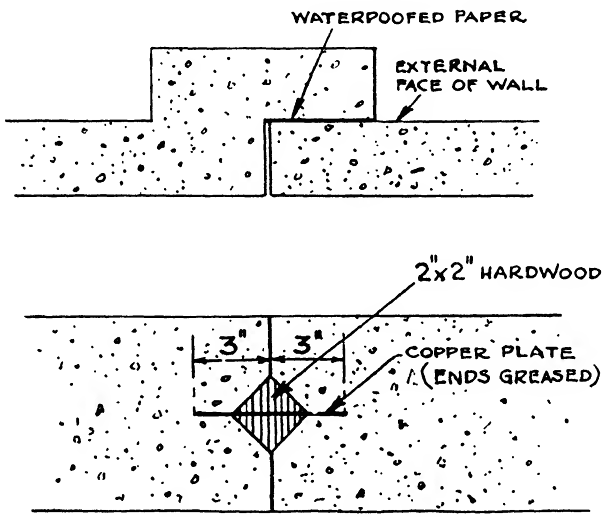


FIG. 391.—JOINTS IN EXTERNAL WALLS OF BUILDINGS (ALTERNATIVE DESIGNS).

can be maintained always full of water. The contraction of the concrete consequent upon the pool being left empty for a period may lead to cracking unless joints are provided. Since joints will show on the floor of the pool, where convenient they should be placed along the swimming lines or along any decorative features incorporated in the treatment of the bath bottom. In all cases where rendering or other surfacing is applied to the constructional concrete it is essential that the joint extend through the finish. Failure to do so will result in the formation of uncontrolled and disfiguring cracks in the latter. Where the appearance of the black line of a bitumen-filled joint is objectionable, the joint filling can be concealed by covering it with a strip of aluminium foil which will adhere to the filling, or a coloured jointing material may be used.

Joints in the external walls of buildings must be weatherproof, and this is frequently accomplished by arranging a simple joint behind an external pillar or pilaster as shown in *Fig.* 391. An alternative and more effective joint is also illustrated, incorporating a copper plate the ends of which, being greased, are free to slip in the concrete when movements take place.

Cement-gun Work.

In cement-gun work, a sand and cement mixture is shot into position at a high velocity by means of compressed air, the material when in place being termed "gunite." The process is extensively used for reconditioning existing structures as well as building-up new work and placing concrete in awkward situations. The plant comprises an air compressor maintaining a pressure of not less than

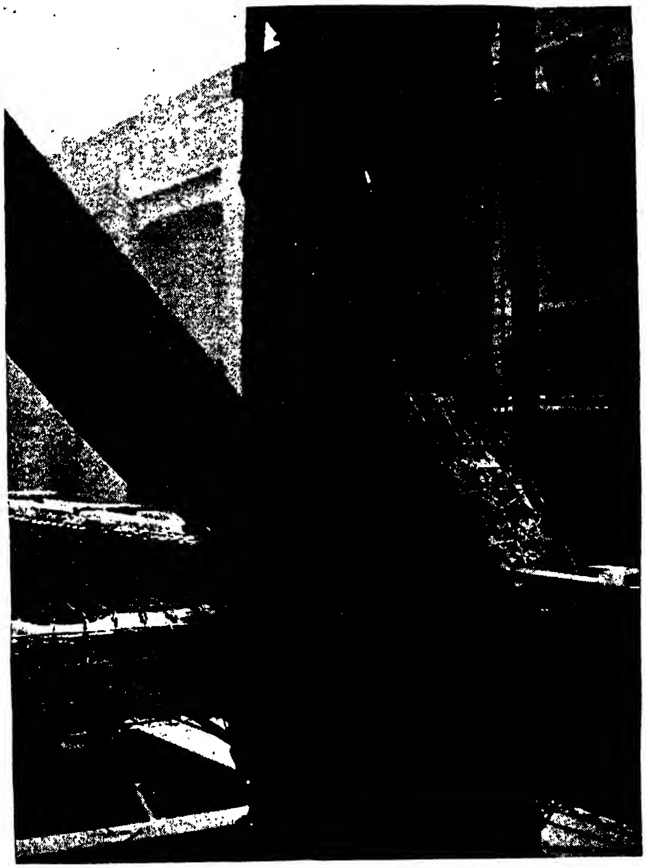


FIG. 392.—WORK PREPARED FOR "GUNITING".

30 lb. per square inch, and a cement gun together with the necessary hose for conveying water, air, and material to the gun.

The material usually consists of 1 part of Portland cement to 3 parts of sand (by volume), a richer mix not being advisable. A mixture leaner than 1 to 4 may be too permeable. The sand should be graded from $\frac{3}{8}$ in. downwards, and particular care should be taken to ensure freedom from deleterious matter.

The cement gun contains two chambers connected by a cone valve. The upper chamber, or material lock, contains the dry-mixed material which passes to the lower chamber where a rotary feed-wheel driven by a compressed-air motor regulates the supply of dry sand and cement mixture to the material hose along which it is carried in the air-stream to the nozzle. The filling of the upper chamber can be taking place while the lower chamber is supplying material to the hose, so that continuous placing of the mixture is ensured.

The mixture is maintained in a dry state until it reaches the nozzle, where it passes through a ring of water-jets directed towards the centre of the nozzle. The amount of water can be varied by the man operating the nozzle. By this method the cement and sand mixture, with only as much water as is required for the hydration of the cement, is placed with considerable force. Consequently the gunite will bond well with the masonry, brickwork, or concrete surface to which it is applied so long as the old surface has been thoroughly cleaned previous to spraying. In preparing the surface of existing work to take gunite, the whole of the defective concrete should be hacked away and the reinforcement exposed. The latter should be wire brushed to remove loose rust and scale. It is recom-

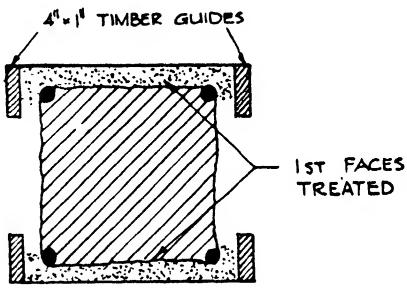


FIG. 393.—GUNITING COLUMNS.

mended that a layer of gunite should be applied completely around the member being repaired although the defective portion may be only on one face. Smooth concrete faces should be hacked to form a key. A mesh of reinforcement should be wrapped round the affected member as shown in the illustration of the typical column and brace joint in *Fig.* 392, the mesh being kept about ¼ in. clear from the face of the old concrete.

Timber screeds to control the finished face of the coating are erected, as illustrated in *Figs.* 392 and 393, at each corner of the member being treated. The gunite is shot on to two opposite faces first, the screeds being used for screeding to a smooth finish. After the coating has set the screeds are removed and the remaining faces gunited, the corners already hardened forming guides for screeding these faces.

The gunite is applied with the nozzle about 2 ft. 6 in. to 3 ft. away from the work, the gun being held as shown in *Fig.* 394. As the mixture is shot against the surface to be covered, the first grains of sand striking the surface rebound but the cement adheres to the surface, thereby building up a cement matrix. Succeeding grains of sand embed themselves into this matrix and force the cement into the interstices of the surface to be covered, thus ensuring that the gunite will form a good bond with a surface. Owing to the force with which it is applied, gunite has great density, and hardens more rapidly than ordinary concretes.

After the main coat has attained its initial set, a finishing coat about $\frac{1}{16}$ in. thick may be applied. The thickness of the main coat used in practice may vary from $\frac{1}{4}$ in. to 4 in., but 2 in. is about the maximum thickness that can be applied in one operation. If greater thickness is required it is usual to apply two coats, both of which would normally be reinforced.

The impermeability and strength of gunite have led to its use in many types of structures, including buildings, bridges and reservoirs and other structures where a thin layer of concrete having high strength and impermeability is required. Reinforced gunite walls $1\frac{1}{2}$ to $2\frac{1}{2}$ in. thick spanning up to 10 ft. between steel or concrete frames may be used in building work, such walls being flexible enough

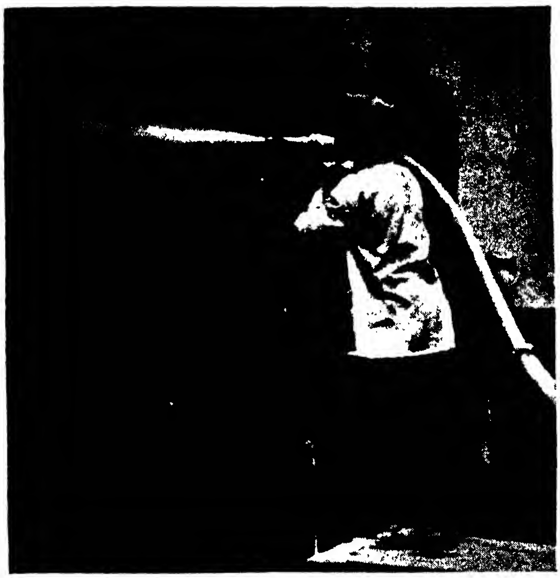


FIG. 394.—APPLYING GUNITE.

to allow for movement in the main members and sufficiently reinforced to prevent temperature cracks. Reinforced gunite is also used for the protection of structural steelwork, having an advantage over ordinary concrete since, due to its greater density, a thinner cover is required which results in less dead weight on the structure. In tunnels, culverts, pipes, and similar work reinforced gunite is applied to provide a hard, dense, waterproof face on the inside.

The labour required for cement-gun work consists of an experienced man to operate the nozzle, one man to work the gun and attend to the compressor, three men mixing and handling the dry materials, and a labourer for odd jobs. Little movement of the cement-gun and compressor about the site is necessary if sufficient length of hose is available, as it is possible to feed the material through distances that in extreme cases have exceeded 500 ft.

The overall cost of cement-gun work in connection with reconditioning existing structures depends upon the thickness and the type of structure; for example, treating frameworks of columns and braces will be more costly than treating plain walls. The amount of cutting away and preparation necessary to the old surface will also affect the cost. The following representative examples of overall cost apply to reinforced concrete structures. Reconditioning columns and braces as in pier construction may cost up to 20*s*. per square yard for 2-in. coatings, while a similar thickness applied to columns and beams in general types of buildings may cost 15*s*. per square yard. The average cost for lining the walls of bunkers, reservoirs, and similar structures ranges from 8*s*. per square yard for 1-in. work to 18*s*. per square yard for 2-in. work, $1\frac{1}{2}$-in. coatings costing from 10*s*. to 15*s*. per square yard depending upon the nature of the work. Lining chimneys with 2-in. of gunite may cost up to 23*s*. per square yard, while reconditioning culverts with a $\frac{3}{4}$-in. coating may cost as little as 6*s*. per square yard.

Testing Completed Structures.

Upon completion of most major structures, a loading test is sometimes specified before the structure is put into service. Such tests, which are at once a proof of the sufficiency of the design and the soundness of the construction, may also be required for minor structures or for one or more members of a building, for one of the following reasons:

(1) As a routine test to comply with the requirements of the purchaser or the local authorities;

(2) When for reasons of faulty materials, workmanship, or design the structure is known to have less strength than was expected.

(3) When the structure will be called upon to sustain a load different from that assumed in the original design.

In the design of most structures a factor of safety of four is usually allowed on both the dead (or permanent) load and the live (or incidental superimposed) load. That is to say, the calculated maximum stresses on the materials do not exceed one quarter of their presumed ultimate strengths. When stressed to a figure of over half their ultimate strength, however, most materials show signs of distress, possibly by excessive deflection or, in the case of reinforced concrete, by the occurrence of cracks in the concrete due to diagonal tension failure or to overstressing the reinforcement in tension. These, or similar, indications would occur generally at a stress considerably below that at which the material would break down and at which probable collapse would consequently occur. While in many cases it is desirable to test the structure with a load in excess of the design load, it is equally important to guard against applying a load so high that a permanent defect in the structure may result. Thus many specifications and regulations call for a 50 per cent. overload for testing floors and similar constructions. Thus, if a floor has been designed for a superimposed load of 100 lb. per square foot and the permanent weight of the floor construction is 80 lb. per square foot, the total design load would be 180 lb. per square foot. A 50 per cent. overload for testing would be interpreted as a superimposed test load of $100 + (\frac{1}{2} \times 100) = 150$ lb. per square foot, thus the actual total load is $150 + 80 = 230$ lb. per square foot. This is a net increase of 28 per cent. In all cases where the full ultimate dead load is not operating at the time of testing, the applied test load should be augmented accordingly. Cases have been recorded where floors and roofs of buildings have been tested to twice the design load when the structure is to be used for a higher superimposed load than originally contemplated.

Although an overload may be specified for buildings, for structures of the warehouse class where the superimposed load designed for equals or exceeds 200 lb. per square foot, it is recommended that the test load should equal the design load. Similarly for bridges, the test load should not exceed the design load and water-containing structures under test should not be filled beyond the design level.

The age of the construction at the time of test should in no case be less than six weeks. With rapid-hardening Portland cement the minimum period can be reduced to three to four weeks. A reasonable period should elapse between removing the shuttering and applying the test load. If on any days during the specified minimum period, hardening of the concrete has been delayed by low temperatures, the test should be postponed for an additional period equal to the period of delay.

The members to be tested would usually be selected by the engineer representing the clients and the actual magnitude, disposition and method of applying

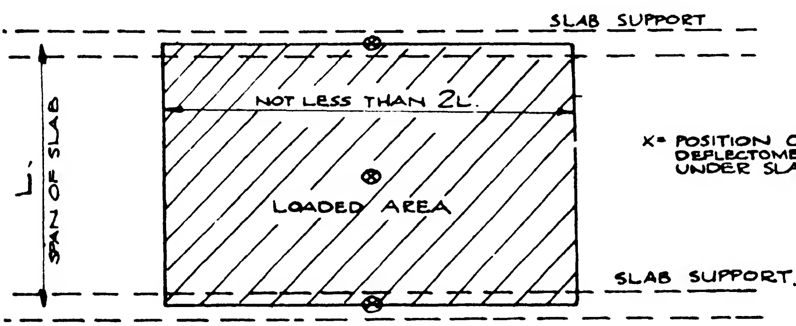


FIG. 395.--LOADED AREA FOR SLAB TEST.

the load, and the extent of the loaded area would be decided in consultation with the engineer responsible for the design. Tests on floors of buildings are conducted by loading a portion of the floor slab and observing its behaviour under loading and unloading. In some cases the beams may also be subject to test. The area of floor loaded for a slab test should extend in length over the whole span and have a width parallel to the supports of at least twice the span, as shown in *Fig.* 395. There is some difficulty in providing a satisfactory arrangement of loading for testing a floor beam. If the amount of load required can be provided in the form of weights or pig iron, it should be distributed uniformly along the line of the beam as shown in *Fig.* 396. When a lighter material only is available, it is better to load the floor uniformly over the whole of the panels on either side of the beam under test, as is also illustrated in *Fig.* 396. In either method of loading, the distributing effect of the slab may be such that it partly relieves the beam under test by transferring some of the load to adjacent beams.

It is often difficult to obtain sufficient weight of loading material to test a floor adequately, but the areas mentioned should be considered the least necessary to obtain results of any value. The applied load may be bricks, cement in bags, sand, weights or blocks of pig iron, these or similar materials being laid directly on the top of the floor. Alternatively tanks of water, placed on the floor, may also be suitable for beam tests. In the case of open frameworks of beams single beams can be loaded by suspending tanks of water from the member under test. If the member is of small section, suspended bags of cement may provide sufficient load. Whatever material is used, precautions must be taken to ensure that the member is not relieved of any load due to arching within the stacked material itself. For this reason, bricks, cement in bags, or similar loadings, must be arranged in independent vertical piles with clearances between each pile.

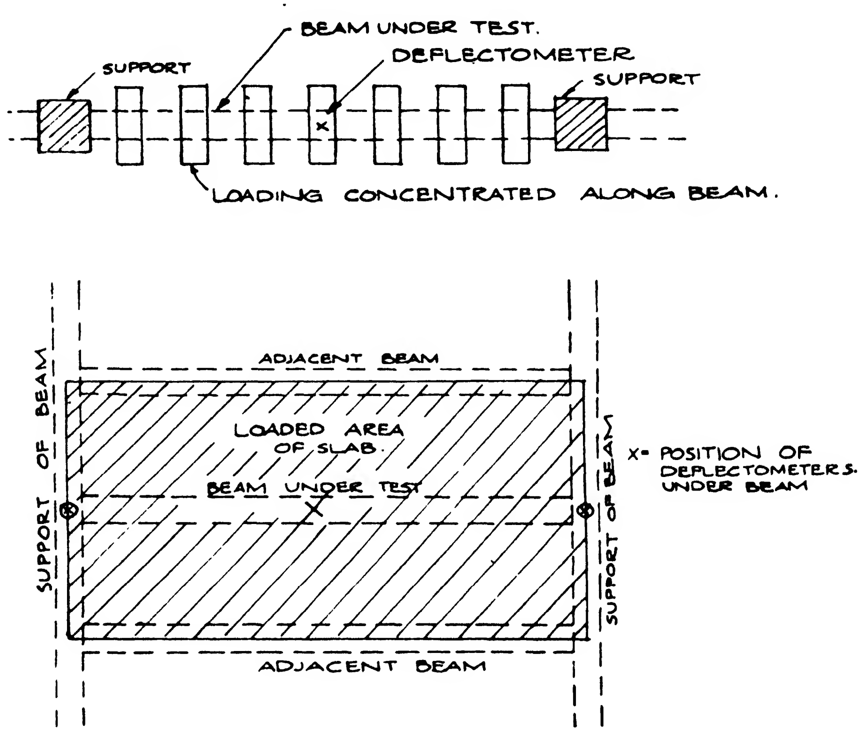


FIG. 396.—LOADING FOR BEAM TESTS.

Dry sand, spread in an even layer, is very suitable for floor tests as it provides a non-arching loa l and is usually available in sufficient quantities on a building site.

Railway bridges are tested by running locomotives over the bridge, and in the case of road bridges, steam rollers, loaded lorries or other available vehicles, as illustrated in *Fig.* 397, are used. In some bridge tests, a stationary load provided by a deep layer of sand spread on the carriageway and footpath, has been combined with moving vehicular loads to simulate the design loading. As the available loading may often differ from that assumed in the design, it is usual for the engineer to specify precisely the number and disposition of the vehicles. *Fig.* 398 illustrates the test load on a 112-ft. clear span double cantilever bridge designed for Ministry of Transport loading. The load, consisting of four lines of steam rollers, loaded lorries, etc., is shown on one half of the structure, and each

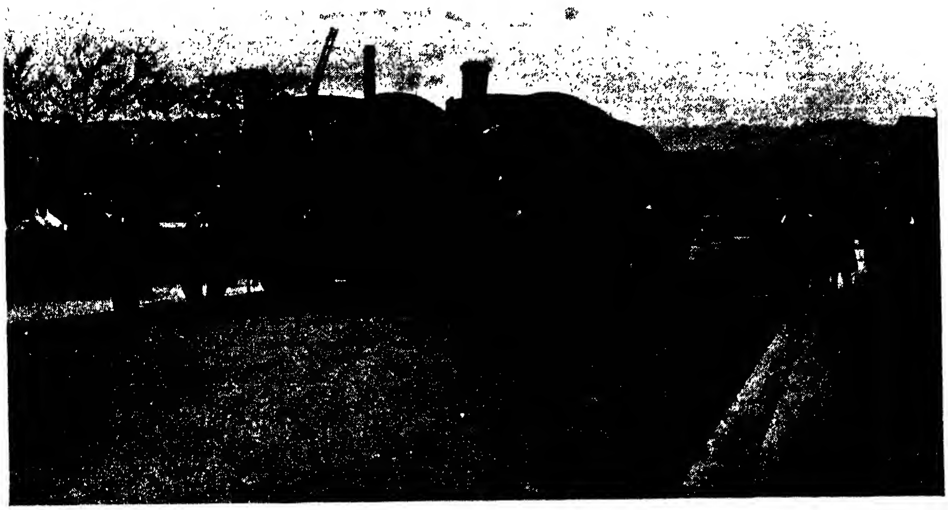


FIG. 397.—TEST LOAD ON BRIDGE.

FIG. 398.—TEST LOAD ON BRIDGE.

line consists of a 12-ton steam roller, a 15-ton steam wagon, and a 13-ton lorry, giving a total load of only 153 tons as compared with 184 tons required for the Ministry of Transport loading, it being found impossible to provide sufficient lorries. Readings of deflection were taken below the centre of the span and on the surface of the roadway at the centre of the span, the maximum deflection for the latter being only $\frac{1}{18.000}$ of the span.

Whatever type of loading is adopted, it should be applied to the structure gently, thus protecting the members under test from shock, unless, as in the case of railway bridges, the nature of the normal working load involves impact. Thus, structures such as grandstands are usually tested by a dynamic load representing the worst conditions of loading likely to be applied in service. A party of men is first marched in step across the span under test. They then walk across in broken step and finally jump in unison upon the structure. During each of these tests, observations of the maximum vibrations and deflections are made.

The effect of the test load on any member of a structure is observed by examination of the member during and after the test and by the measurement of the deflections produced by the loading. Deflectometers for measuring the movement are mounted on a timber staging built up to within a foot of the slab or beam soffit, it being recommended to have one instrument at midspan and one under, or as close as possible to, each support. The net midspan deflection of the member is the difference between the reading of the central deflectometer and the mean of the readings of those at the supports. Before any loading is applied, the instruments should be fixed and adjusted by packings to their zero positions. The load should be applied steadily and symmetrically and the deflectometer reading taken at intervals and at least when half the design loading is reached and at the full design loading. Additional readings should be taken if an overload is applied, when the maximum load is on the member.

The full load should be left in position for some hours before final readings of the maximum deflection are taken. After the test load has been removed, the residual or permanent deflection should be recorded. Recommendations differ regarding appropriate times for taking these readings. A minimum of six hours of full loading before recording the maximum deflection is recommended, while the permanent deflection should be taken twelve hours after the removal of the load. The recommendation for a Code of Practice require the test load to be maintained in position for twenty-four hours, at the end of which period the maximum deflection should be recorded.

The maximum deflection of a beam or slab to be satisfactory should, with a 50 per cent. overload, not exceed $\frac{1}{600}$ of the span when (i) the load is uniformly distributed throughout the span and (ii) the ends of the member are freely supported. For a continuous span, the maximum satisfactory deflection should not exceed $\frac{1}{1,600}$ of the span. In most well designed and soundly constructed work, whether beam and slab construction or arch construction, the measured deflections will be considerably less than these values.

The permanent deflection should not exceed 25 per cent. of the maximum deflection; that is there should be a 75 per cent. recovery upon removal of the load. If the recovery is less than this amount, the Code of Practice recommends that the member be reloaded, the increase in deflection being recorded. Upon removal of the load the second time, the recovery should be at least 75 per cent. of this increase.

The deflectometer may be either an instrument as illustrated in *Fig.* 399

which is manufactured for the purpose of testing structures, or a simple site-made pointer moving over a graduated dial can be used if it is substantially and accurately constructed. If dumpy level readings are taken, they should be checked against deflectometer registrations. In all cases, the staging upon which the deflectometers are mounted and the datum to which levels are referred, should be such that they are unaffected by the application or renewal of the load. Precautions should be taken to ensure that the instruments are not interfered with during the testing period.

While the test is in progress, careful observations should be made to detect any cracking of the concrete. The soffits of slabs should be inspected for this purpose and beams should be examined for cracks on the soffit at midspan and over the top of the supports, and for diagonal tension cracks near the supports.

Should the maximum deflection exceed the specified amount or should the member under test fail to regain the specified amount or exhibit any other indications of weakness, the test might be considered as proving the work to be unsatis-

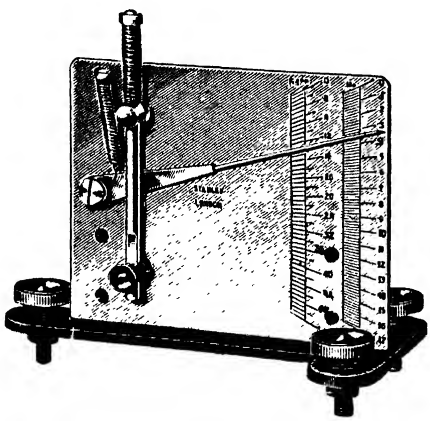


FIG. 399.—DEFLECTOMETER.

factory. The engineer is then generally within his rights to order the contractor to carry out at his (the contractor's) expense any demolition, making-good or additional work that may be considered essential for security. Similarly the cost of testing, including provision of any scaffolding or staging, and usually the provision and handling of load, supply of recording instruments and other equipment are chargeable to the contractor who ordinarily will allow for these items in his original tender. In some cases, the client will provide the loading material delivered on to the site, although the contractor is usually responsible for transferring the material to the floor under test and for subsequent removal. In many cases, also, the engineer lends the deflectometers.

When a test is carried out because the work is obviously defective or where there are other reasons for suspecting that the member will not carry the full test load, precautions must be taken to prevent, in the event of a collapse, injury to the men and equipment, and damage to other parts of the structure. Adjacent beams and the floor (if any) below that being tested may need temporary propping and a substantially constructed platform, strong enough to carry the weight of the portion of the floor under test together with the superimposed test load, should be erected as close as possible to the soffit of the floor. Instead of mounting the deflectometers on this staging they can be placed on the floor below and movements of the loaded floor transmitted to them through light, stiff rods passing through the platform as illustrated on *Fig.* 400. Observations of the behaviour of the floor should be particularly rigorous throughout the test and overloading should not be applied until the engineer in charge has satisfied himself that the floor may be expected to carry additional loading.

In cases where the concrete is known to be defective, further information may be obtained by testing in compression cores taken by drilling from the actual work. This test, however, cannot take into account honeycombing, lack of consolidation around the reinforcement, or similar local defects.

Reservoirs, water towers or other liquid containers, are tested to prove not only sufficient strength, but watertightness. A general specification for a test

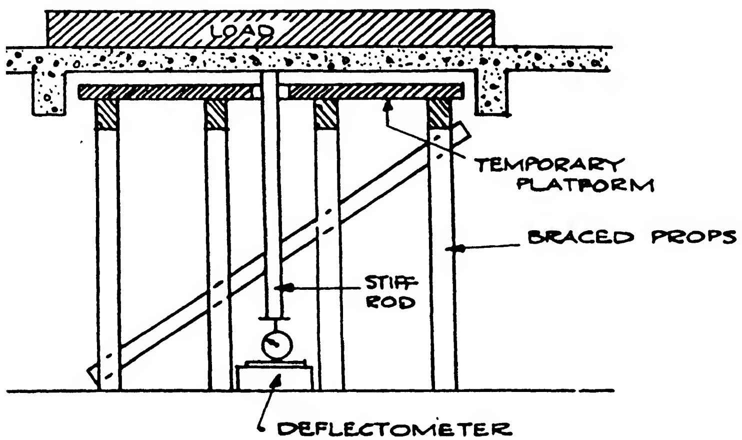


FIG. 400.—ARRANGEMENT FOR TESTING DOUBTFUL FLOOR.

would be to fill the reservoir or tank to a given level and at the end of a given time the level of the water should not have fallen more than a specified amount. In assessing an allowable amount, consideration should be given to such factors as evaporation of water in uncovered reservoirs due to the sun and wind, and leakages through valves and pipes that may not be within the concrete contractor's contract. Thus for a covered reservoir the maximum allowable fall in water level might be specified as $\frac{1}{4}$ in. in eight hours after a test maintained for three days, while for a covered water tower tank, the test may be required to be in progress for three weeks during which time the water level should not fall more than $\frac{1}{8}$ in. in a period of forty-eight hours.

If the test is unsatisfactory, the contractor is usually called upon to line the reservoir with asphalt, rendering, or other material, at his own expense, until impermeability is secured. In the case of earth-covered reservoirs it is preferable to make the test before the earth banking is placed against the walls, thus enabling any points of leakage to be readily detected. Before testing under these conditions it is necessary to determine whether the designer has allowed in the design of the walls that the effect of the water pressure will in part be offset by the pressure from the earth outside the walls. This point must be given consideration before filling any underground tank work, since serious overstressing may result if the containers are filled without the anticipated counter-pressure from the earth.

The testing of some types of industrial structures where air- or gas-tightness is essential, may include an air pressure test in addition to a hydrostatic test. Thus, a set of gas purifier-boxes may be specified to sustain a pressure of 48 in. of water followed by an air test for forty-eight hours at a pressure equivalent to 36 in. of water. The success of such tests may be more dependent upon the close fitting of covers, valves and other fittings than on the quality of the concrete construction.

Testing of the load-carrying capacity of piles and methods of testing the bearing resistance of ground have been dealt with in Chapter VII.

ILLUSTRATIONS.

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