BUILDING DESIGN AND CONSTRUCTION

VOLUME I

BUILDING DESIGN AND CONSTRUCTION

CONTENTS OF VOLUMES II AND III

BUILDING DESIGN AND CONSTRUCTION

WITH REFERENCE TO THE NEW L.C.C. REGULATIONS

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FOREWORD

By H. Berry, Esq., *Chairman of the Town Planning and Building Regulation Committee, London County Council.*

THE L.C.C. is sometimes erroneously described as a very slow-moving body; but, however that may be, in recent years it has moved to some purpose in the direction of modernising London's Building Laws and Regulations. Possibly the most striking advance has been the London Building Act (Amendment Act) of 1935 and the up-to-date By-laws, produced under that Act, which came into operation on 1 January, 1938. These By-laws cover the whole range of building, and for the first time include timber. A Code of Arc Welding made its debut on the same date. For the convenience of the London building public, these, together with much more allied information, have been published in book form, under the title of " Construction of Buildings in London," and this carefully indexed volume should be a hand-book to all who handle the many phases of building in London. In some respects the book is self-explanatory: but in many others a commentary, written by persons with a good working knowledge of the subject or in professional practice, would be of great additional value. This commentary is supplied by the present volume, the authors of which have both contributed, in varying directions, useful criticisms of the By-laws and Code in their draft stages.

While a commentary generally adheres closely to its text, it is realised that sometimes a departure from and criticism of that text is useful. The authors have, from time to time, made their contribution in this direction, and while their volumes are bound to be of the greatest value to the designer and the student, they are also of value to those whose duty it is to administer.

Nor is the value confined to those operating in the County of London, for the London Standard of Building has been adopted in many parts of the British Commonwealth of Nations. It is to be devoutly hoped that most places in this country will adopt a similar standard, to the benefit of those who occupy buildings. A low standard of building is a crime against the community, and the endeavour of the London Building Code is to prevent such crimes. The present volumes will help all who wish to build in a self-respecting manner.

H. BERRY

Feb., 1939

INTRODUCTION

IMMEDIATELY after the publication of the recent and far-reaching revisions of London's building regulations we found it necessary to prepare certain explanatory notes, tables, and sketches for reference in our own drawing-office. The suggestion was then made that these data sheets might well find a wider field of use, and therefore the explanations were expanded, additional information in the form of notes, drawings, and tables was prepared, sections on design treating both of theoretical and of practical considerations were written, and finally the whole work was collected into the form in which it is now presented.

Of the first of the three volumes into which it was found necessary to divide the work for publication, the greater part has been devoted to technical explanations of the By-laws, and these have been accompanied by sketches, tables, and graphs; but, in order that the legal position might also be understood, a short resumé of the several Parliamentary and Council enactments, to which buildings in London must conform, has been included, together with extracts from operative Sections of the London Building Act and of dependent enactments. Finally, a chapter on monolithy (framework and continuity) has been included in the first volume, because it treats of certain questions of principle arising out of some of the By-laws. That chapter is of general application, being based on the Council's "Memorandum—Computation of Stresses," which in its turn follows very closely the Code of Practice of the Building Research Board (Department of Scientific and Industrial Research).

In the other volumes, chapters have been given on the design of structures composed of such materials as structural steel, including welded steel, timber, reinforced concrete, and brickwork, and notes have been included on foundations. Also notes have been made throughout on the economic arrangement and use of structural materials, and separate chapters on this matter have been included in addition. Further, over a thousand formulæ of general mathematical and structural use have been given.

It is not intended that this work should be, in any way, a text-book on structural mechanics. Our aim has been two-fold: first, to produce a work of reference to which architects, engineers, surveyors, builders, etc., can turn to find the requirements of the authorities on a point of construction; and, secondly, to assist a designer, who has already some knowledge of structures, by means of guiding rules, tables, and graphs.

Owing to the different standards of knowledge or ability in structural design which obtain, the information has been set out in two ways: first, in formulæ and tables, which can be used by one without a working acquaintance with higher mathematics; and, secondly, for designers with such knowledge by the use of processes involving the calculus.

One may rightly maintain that in general London's building regulations are now up to date and in line with developments in structural science, and that consequently their observance leads to sound and rational construction. However, there are points which, in our opinion, could be disputed, and where it would appear that unnecessary work would be involved by a strict adherence to a particular requirement, it is suggested that the designer apply for a waiver or relief of that requirement. On the other hand, we have sometimes recommended standards which are higher than those specified. Our aim has been to suggest economic construction which is consistent with stability.

Although the work has been based primarily on the London County Council regulations, most of the explanations given and the general chapters on design, treating of principles and of methods rather than of specific requirements, are in no way limited to London. This applies particularly to the data tables and graphs which are given to facilitate the actual design and calculations of concrete, steel, and timber. For this reason and for the reason that the London County Council standards are being adopted very widely throughout this country and in other British possessions, the use of the book is not restricted to the few miles surrounding Charing Cross. Whilst naturally every care has been taken to ensure that the statements and information in this book are correct, we cannot of course accept any responsibility in that connection.

In order to facilitate the finding of any information numerous cross references have been made both in the text and in the comprehensive index which has been included at the end of each volume.

There is yet another word which must be said. It is one of gratitude to the several gentlemen who have assisted us by their advice and criticism during the preparation of the work. Particularly would we thank Mr. P. J. Black, L.R.I.B.A., Vice-President of the Institution of Structural Engineers and Chairman of the District Surveyors Association.

A word of thanks to Mr. H. Berry must be added for his part, as chairman of the Town Planning and Building Regulation Committee of the London County Council, in removing many of the obsolete building regulations which tended to hamper rational design and construction.

FELIX J. SAMUELY.

CONRAD W. HAMANN.

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VOLUME I

LIST OF DESIGN TABLES, GRAPHS AND FORMULÆ

PART I

REGULATIONS, GENERAL

CHAPTER 1

SURVEY OF LONDON BUILDING REGULATIONS

In 1935 Parliament passed the London Building Act (Amendment) Act, by which the London County Council were empowered to make and modify their own regulations, which govern, amongst other matters, the construction of buildings, and the modifications and new regulations were to be made in the form of by-laws. Under this Amendment Act of 1935 various sections of the London Building Act, particularly those relating to building construction, have been replaced by new by-laws made recently. All previous Acts and regulations controlling buildings and streets within the area of the London County Council had been collected in the London Building Act, 1930, of which the sole purpose was to consolidate all previous enactments.

In 1937 by-laws for the construction and conversion of buildings and furnace chimney-shafts were passed by the Council, and came into operation on the first day of January, 1938. These by-laws relate principally to Part VI (construction of buildings), to the 2nd and 3rd Schedules of the London Building Act, 1930, and to the Reinforced Concrete Regulations made by the Council under the Act, and they apply to all buildings of brick, steel and concrete construction and of composite constructions such as brick and timber, with the exception of buildings erected by such bodies as the railway companies (and public authorities, as for instance, the London County Council itself) who have different powers under their own Acts of Parliament.

In addition, separate by-laws for the use of timber in the construction and conversion of buildings were passed by the Council in 1937, and these also came into operation on the first day of January, 1938. The Council have also prepared three documents to assist designers and builders in applying for waivers of certain of the building By-laws.

The first of these is entitled "Welding—Application for Modifications or Waivers," and regulations are given for applications for waivers of By-laws 63, 64, 69, 70, 72, 74, 75, 78, 81, 85, 86, 87, 90, 91, 104 and 114 so as to permit the use of electric metallic arc welding in the fabrication and erection of structural steelwork.

The second is entitled "Steel—Reinforcement (for Reinforced Concrete). Application for Modifications or Waivers." These regulations refer specifically to By-laws 15, 92, 94, 96, 97, 99, 100 and 103 to 114 inclusive, so as to permit the use of steel reinforcement for reinforced concrete, other than the reinforcement required under the By-laws. Application for the use of special deformed bars or of high tensile steel, or of any other special steel, can, therefore, be made.

The third group of regulations has been given the title "Structural Steel, Applications for Modifications or Waivers." Applications in accordance with these regulations may be made to permit the use of structural steel other than the structural steel which is required by the By-laws. They refer specifically to By-laws 15, 63, 69, 73, 81, 82, 83, 84, 85, 87, 90 and 91.

These three documents, relating to welding, to special steel for reinforced concrete, and to special structural steel, have been prepared by the Council to serve only as indications to the designer or builder of what conditions and materials the Council may request before any waiver of the By-laws is granted, and in themselves they have no legal significance. They have, however, been discussed in detail in this volume, and although they do not actually form part of the By-laws, they have been introduced at places in the explanations of the By-laws where it has been considered they would be most appropriate and of greatest assistance.

A document has also been prepared, published in April 1938, entitled "Building By-laws—Computation of Stresses," with a sub-title of which an abstract reads: "Memorandum prepared by the Superintending Architect . . . issued by the London County Council to afford guidance in the computation of stresses in the construction of buildings and chimney-shafts."

This document sets out the methods by which it is recommended that structures should be calculated; and whilst, obviously, it is wise that one should follow the methods as far as possible, the Memorandum is in no way binding to the designer.

On 11 April, 1935, the Advisory Committee on the Amendment of the London Building Act, 1930, published a report which was handed over by the London County Council to its Building Acts Committee for consideration. On 9 May and 4 and 11 July, 1938, the Town Planning and Building Regulation Committee (which the former Building Acts Committee had since become) reported back to the Council setting out, in a very comprehensive report, how the several recommendations of the Advisory Committee had been dealt with and at the same time giving a very clear survey of all the legislation which had been passed and actions which had been taken since 1935. This report of the Town Planning and Building Regulation Committee has been published as an Extract from Minutes of Proceedings of the Council on 19 July, 1938. This report should be read by all who are interested in or affected by the building regulations in London, or, indeed, in any other part of the country, for many of the conclusions of the Committee are on points of general interest.

In the following chapters of this book, these various documents will be referred to by the abbreviated titles which are set out in the second column immediately below.

The London Building Act (Amendment) Act, 1935.	"Amendment Act."
The London Building Act, 1930.	"London Building Act" or "Act."
By-laws for the construction and conversion of buildings and chimney-shafts made by the London County Council in pursuance of the London Building Act (Amendment) Act, 17 December, 1937. Publication No. 3319.	"By-laws."
By-laws made by the London County Council in pursuance of the London Building Act (Amendment) Act for the use of timber in the construction and conversion of buildings, 3 December, 1937. Publication No. 3317.	"Timber By-laws."
Welding—Application for Modifications or Waivers. Regulations made by the London County Council, 1937. Publication No. 3318.	"Welding Regulations."
Steel Reinforcement (for Reinforced Concrete). Application for Modifications or Waivers. Regulations made by the London County Council, 1938. Publication No. 3329.	"Reinforcement Waiver Regulations."
Structural Steel. Application for Modifications or Waivers. Regulations made by the London County Council, 1938. Publication No. 3371.	"Structural Steel Waiver Regulations."

London County Council Building By-laws—Computation of Stresses. Memorandum prepared by the Superintending Architect, published April, 1938. Publication No. 3333.	" Memorandum."
Report of Town Planning and Building Regulation Committee. Minutes of Proceedings of London County Council, 19 July, 1938. Publication No. 3370.	" T.P. and B.R.C. Report, July, 1938."

It is recommended very strongly that the user of this book read carefully the individual documents, particularly the London Building Act, having first noted those sections which have been replaced by the By-laws.

The By-laws, Timber By-laws, Memorandum, Welding Regulations, and Reinforcement and Structural Steel Waiver Regulations have all been published now, within the one cover, where also extracts from the British Standard Specifications referred to in the By-laws, etc., are given.

In order to facilitate the understanding of the general position and scope of all these documents, it is proposed, before investigating them in detail, to give a brief summary of the London Building Act and of the By-laws.

THE ACT

The London Building Act is divided into seventeen parts.

Part I. Ss. 1–5. Part I, which is introductory, comprises a description of the Act, and its scope together with some fifty-four definitions of terms which occur in later parts.

Part II. Ss. 6–21. These sections govern the formation and widening of streets.

Part III. Ss. 22–32 control the lines of building frontage.

Part IV. Ss. 33–41 cover the naming and numbering of streets.

Part V. Ss. 42–56 set out the governing conditions for the heights of buildings and the open spaces required round them.

All these parts require the attention of the designer or builder. However, as it is the purpose of the authors to confine this volume, as far as possible, to constructional requirements only and to the alterations which have been made in these requirements, it is not proposed here to enlarge on any of these preceding sections of the Act.

Part VI contains thirty-two sections on the construction of buildings; but as the majority of these sections have been replaced by the By-laws, it has been considered advisable at this stage to enumerate these sections and to give the numbers of the By-laws by which they have been superseded.

S. 57 and 2nd Sch.—Section 57, subsection (1), together with the whole of the 2nd Schedule of the Act which is invoked by clause (*b*) of subsection (1) (which 2nd Schedule sets out the requirements for walls and footings such as thicknesses and heights, etc., in walls of domestic buildings and warehouses, but not of public buildings), has been replaced by Parts I to VII of the By-laws and by By-laws 150 and 157.

Section 57, subsection (2), has been retained, and under this clause open sheds not higher than 16 ft. and not greater than 400 sq. ft. in area are exempt from the requirements of the Act; but they must nevertheless be constructed to the satisfaction of the district surveyor.

S. 58, 3rd Sch., S. 59, and Reinforced Concrete Regulations.—The whole of Section 58, together with the 3rd Schedule to the Act covering metal skeleton construction, loads, and working pressures on brickwork and on the ground, have similarly been superseded by Parts I to VII of the By-laws and by By-laws 150 and 157, as have also Section 59 and the Reinforced Concrete Regulations (allowed on 8 September, 1915, and 30 July, 1937) made under subsection (2) of Section 59.

S. 60.—Subsections (1), (2), (3) and (5) of Section 60, relating to the recesses and openings in walls, have been replaced by the requirements of Parts I to VII of the By-laws and By-laws 150 and 157. Subsection (4) has been retained, stipulating that no opening may be made in a party wall unless in accordance with the appropriate conditions of the Act.

S. 61 : Timber in external walls. Replaced by By-law 142.

S. 62 : Bressummers. Replaced by Parts I to VII of the By-laws and By-laws 150 and 157.

S. 63 : Parapets to external walls. Replaced by By-law 145.

S. 64 : Walls deemed to be party walls. Retained.

S. 65 : Height of party walls above roof. Replaced by By-law 146.

S. 66 : Chases in party walls. Replaced by Part IV of the By-laws.

S. 67 : Construction of roofs. Replaced by By-law 147.

S. 68 : Storeys in roofs. Replaced by By-law 148.

S. 69 : All but one (Subsection (19)) of the twenty-two subsections of Section 69, relating to chimneys and flues, have been replaced by By-law 132. The retention of Subsection (19) makes it illegal to cut away a chimney breast or shaft built with or in a party wall unless the district surveyor certifies that it can be done without injuriously affecting the stability of the building.

S. 70 : Furnace chimney-shafts. Replaced by Parts I to VII of the By-laws and By-laws 150 and 157.

S. 71 : Close fires and vapour pipes. Replaced by By-law 136.

S. 72 : Floors over furnaces and ovens. Replaced by By-law 138.

S. 73 : Has been modified and retained in a form which requires that all floors of lobbies, corridors, landings, etc., in public buildings shall be constructed of and carried on supports of fire-resisting materials. This requirement is applied to certain other buildings by By-law 140.

S. 74 : Ventilation of staircases. Retained.

S. 75 : Requirements for habitable rooms. Replaced by By-law 141.

S. 76.—Subsections (1), (2), and (3) of Section 76, relating to windows in habitable rooms, are replaced by By-law 149.

Subsection (4) of Section 76: Windows in habitable rooms. Replaced by By-law 141.

S. 77 : Arches over public ways. Replaced by By-law 137.

S. 78 : Arches over passages or public ways. Replaced by By-law 139.

S. 79.—Subsection (1), the first sentence of Subsection (2), and Subsection (4) of Section 79, governing the projection of or from buildings, have been replaced by By-law 143. The remainder of Subsection (2) and Subsections (3), (5), (6), (7) and (8) have been retained.

S. 80.—Subsections (1) and (2) of Section 80, setting out the requirements for separating walls, etc., between buildings, have been replaced by By-law 144.

The first paragraph of Subsection (3) of Section 80 has been replaced by By-law 140, and the retained paragraph refers to the district surveyor's fee for inspection of chambers not as separate buildings.

S. 81 : Cubic extent of buildings. Retained.

S. 82 : Uniting of buildings. Retained.

S. 83 : Division walls to be to party wall requirements. Retained.

S. 84 : Plans. Retained.

S. 85 : Construction of public buildings. Retained.

S. 86 : Conversion of buildings into public buildings. Retained.

S. 87 : Staircases in certain public buildings. Retained.

S. 88 : Buildings under railway arches. Retained.

Part VII. Ss. 89–94.—Part VII of the Act has been retained as a whole. This part refers to special and temporary and timber structures, and comprises Sections 89 to 94.

It should be noted here that the Timber By-laws do not replace any sections in the London Building Act, because the Act itself does not contain specific requirements for any timber constructions. Therefore, Timber By-law 1 to Timber By-law 24, together with their Schedule, give the requirements for strength and quality of all load-bearing timbers which may be used in any building governed by Parts VI and VII of the Act. They do not apply to any building which is exempt from the operation of these Parts VI and VII of the Act.

Part VIII. Ss. 95–112 : Means of escape in case of fires. Retained.

Part IX. Ss. 113–127 : Rights of building and adjoining owners. Retained.

Part X. Ss. 128–142 : Dangerous and neglected structures. Retained.

Part XI. Ss. 143–145 : Dangerous and noxious businesses. Retained.

Part XII. Ss. 146–148 : Dwelling-houses on low-lying land. Retained.

Part XIII. Ss. 149–150 : Skysigns. Retained.

Part XIV. Ss. 151–183 : Superintending Architect and District Surveyors. Retained.

Part XV. Ss. 184–186.—These sections, retained without modification, set out the powers of the Council to make by-laws for certain purposes, which purposes are enumerated in clauses (*a*) to (*q*) of Subsection (1), Section 184.

Part XVI. Ss. 187–210 : Legal proceedings. Retained.

Part XVII. Ss. 211–236.—One alteration has been made to Part XVII—Miscellaneous—in that Subsections (6) and (7) of Section 222 have been replaced by Parts I to VII of the By-laws and By-laws 150 and 157.

The 26 sections of this part of the Act treat of such matters as historic buildings, expenses, plans and documents, offences against the Act, Crown exemptions, etc.

Six Schedules to the Act are then attached.

1st Schedule : names various building materials which may, for the purposes of the Act, be deemed to be fire-resisting. This Schedule has been retained.

2nd and 3rd Schedules.—As has been shown under Sections 57 and 58, the 2nd and 3rd Schedules have been superseded.

4th Schedule.—In the 4th Schedule are given the fees which are payable to the Council in respect of dangerous or neglected structures. This Schedule has been retained.

5th Schedule, in seven parts, has also been retained. Part I covers fees payable by the Council to the district surveyor on means of escape in case of fire; Part II, fees payable by the Council to the district surveyor on dangerous structures; Part III, fees payable by builder, owner or occupier to district surveyor; Part IV, fees payable by builder, owner or occupier to the district surveyor in respect of buildings wholly or partly of reinforced concrete or metal skeleton construction; Part V, fees payable by builder, owner or occupier to the district surveyor on means of escape in case of fire; Part VI, general regulations regarding fees; and Part VII, interpretation of "building erected after the commencement of the Act" for the purpose of the 5th Schedule.

6th Schedule : gives a list of all previous enactments which have been repealed by the Act. This Schedule has been retained.

Reinforced Concrete Regulations, made under Section 59, Subsection (2), and allowed on 8 September, 1915, and 30 July, 1927, have been replaced entirely by Parts I to VII of the By-laws and By-laws 150 and 157.

Metropolis Management Act, 1855.—By-laws made by the Metropolitan Board of Works on 17 March, 1857, relating to the formation of new streets in the Metropolis, have been retained; but by the Local Government Act of 1888 the powers of the Board were transferred to the London County Council and any notices required to or from the Board must therefore be given to or received from the Council.

Metropolis Management and Building Act Amendment Act, 1878.—By-laws 1 to 7, made by the Council under Section 16 of the Metropolis Management and Building Act Amendment Act on 13 October, 1891, and confirmed on 19 October, 1891, relating to repeal of previous by-laws, foundations and site, materials for walls, duties and fees of district surveyor, plans and penalties, have been superseded by Parts I to VII of the By-laws and By-laws 150 and 157.

L.C.C. (General Powers) Act, 1890.—Under Section 31 of the London Council (General Powers) Act, 1890, five by-laws were made by the Council on 13 October, 1891, and confirmed on 19 October, 1891. These by-laws, regarding materials for plastering, excavation and filling materials, duties and fees of the district surveyor and penalties, have all been replaced by Parts I to VII of the By-laws and By-laws 150 and 157.

The following remaining enactments are retained in operation :

By-laws. 30 June, 1914.—Under Sections 184 and 236 (2) (*a*) of the Act, By-laws made by the Council on 30 June, 1914, and allowed on 20 February, 1915, have been and are still retained in operation. They refer to the regulation of lamps, signs or other structures overhanging the public way not being within the City of London.

L.C.C. Rules for Application under the Act. 1 October, 1930.—On 1 October, 1930, the Council issued a document entitled "Rules governing applications under the Act." These rules and regulations still apply, and they set out details of the type of plans which are required and give information regarding the naming and numbering of streets, etc.

L.C.C. Regulations for Applications under Part III of the L.C.C. General Powers Act, 1908 : October 1928.—Regulations dated October 1928 have been retained. They have been adopted under Section 236 (2) (*a*) of the London Building Act and cover the applications under Part III of the London County Council General Powers Act, 1908, for additional cubic extent of a building.

There are also attached resolutions of the Council dated 18 December, 1928, regarding similar regulations governing buildings to which Section 17 of the London

County Council (General Powers) Act, 1908, applies : or, as Section 17 of the London County Council (General Powers) Act, 1908, was re-enacted as Section 81 of the London Building Act, to which Section 81 of the Act applies.

L.C.C. Regulations made under Section 8 of the London County Council (General Powers) Act, 1920, on 25 January, 1921.—These regulations refer to dwelling houses in low-lying lands and set out details of the information and plans which must be given to the Council. They remain in force.

L.C.C. Resolutions under powers of 1st Schedule (Part 3) to London Building Act (Amendment) Act, 1905 : 23 June, 1925, 2 March, 1926, 13 July, 1926, March 1938.—These resolutions approve as fire-resisting materials certain hollow floors and roofs, wired glass, and fire-proofed soft woods, provided in each case that certain specified conditions are observed. The resolutions remain in operation.

London Building Acts : Regulations made by the Tribunal of Appeal 23 April, 1914.—The requirements for the hearing of an Appeal are given and the scale of fees to be paid to the district surveyor is attached.

London Building Act. Section 172.—A scale of fees payable to the District Surveyor under Section 172 of the Act is set out.

Sections of Various Acts Relating to Building in London.—Sections of the following Acts operate in relation to building in London :

Metropolis Management (Section 6) Amendment Act, 1890.
Public Health (London) Act, 1891 : Sections 96, 97 and 98.
Housing, Town Planning, etc., Act, 1909 : Section 43.
Town and Country Planning Act, 1932.
City of London (Various Powers) Act, 1911 : Sections 35, 36 and 38.
Education (Administrative Provisions) Act, 1911 : Section 3.
Metropolitan Water Board Act, 1913 : Section 83.
Factories Act, 1937.

They concern the laying out of roads ; underground rooms ; back-to-back dwellings ; wooden structures and hoardings ; exemption of schools under certain conditions from any by-laws made by a local authority under Section 157 of Public Health Act, 1875 ; exemption of Metropolitan Water Board structures ; provision of fire escapes for factories ; doors to workshops ; offences and penalties ; definitions of factories, etc. ; laundries and institutions.

London County Requirements (under Factories and Workshops Act, 1901 and 1907, and London Building Act (Amendment) Act, 1905) for means of Escape in Case of Fire.—The matters covered by these requirements include means of access to roof, conversion of buildings, exemption of certain buildings, arbitration, plans, staircases, maintenance, materials, lighting, storage of inflammable liquid, windows, etc.

Public Authorities Protection Act, 1893.—This Act would not normally affect buildings ; but if legal action is to be brought against any person carrying out statutory or other public duty in respect of any building, then attention would have to be given to the Act.

Waivers.—The Council draws special attention to the fact that Section 9 of the Amendment Act gives them power to modify or waive any regulation which is contained in the By-laws. It will be of interest to Architects and Engineers to know what kind of waiver will be given.

Before the By-laws came into operation, waivers of a number of constructional requirements could be obtained, and sometimes, such waivers contradicted the old regulations. This, of course, was due mainly to the fact that some parts of the Act, especially the reinforced concrete regulations, were hopelessly out of date.

Where a new by-law has been so drafted to replace a specific requirement of the Act, of which relief would previously have been given, it is not to be expected that the by-law would be waived. Where, however, a by-law is a repetition of a requirement of the Act, of which relief would previously have been given, but which, under the limitations of the Amendment Act the Council had not been empowered to modify, then a waiver could be expected to be given.

There are two classes of waiver which will have to be considered:

First, there are those waivers which are already provided for by the published general regulations, in which publications the actual By-laws that can be waived are enumerated together with information as to the alternative requirements which the Council would be prepared to allow, provided that any special conditions which they might think necessary in a particular case would be observed. These regulations give the necessary information for making applications for the use of welding in structural steel and for the use of high tensile or other steels either for reinforced concrete or structural steelwork.

Secondly, there are the waivers which will be given where, in particular cases, a strict interpretation of the wording of the By-laws would impose unnecessarily hard conditions on the work.

The second type will apply particularly to existing buildings which may have been erected under conditions less stringent than those which the new By-laws require. This will be observed especially with regard to fire-proofing. They will apply also to small and unimportant buildings where lower standards can be justified.

The majority of waivers granted by the London County Council refer to By-laws 43, 91, 132, 133, to 137, 140, 141, 143 and 144.

By-law 43 sets out the requirements for external walls and gives two exceptions, for which the thickness of the external walls can be reduced to 4½ in. A number of waivers have been granted giving permission to use brickwork not less than 4 in. in thickness for small outbuildings, lobbies, sheds, etc., and in one or two cases for bay windows; but the condition that such 4 in. should be specially protected by tiles against weather has been imposed. By-laws 137, 140, 141, 143 and 144 very often apply to the rebuilding of existing structures and refer mainly to the question of the fire-proofing of buildings of different ownership. In such cases waivers have been granted which make the conditions less stringent than those of the By-laws. By-law 91 requires that where structural steel is used in any building, that steel must conform to the same requirements as in steel-frame buildings. As this request has never been made before, alterations in existing buildings by means of steelwork would sometimes be made impossible if special waivers were not granted. The Council has in many cases allowed the standard of fire protection in such buildings to be lower than that of steel-frame buildings.

Apart from those by-laws mentioned, of which waivers have been granted several times, there are certain others of which one waiver has already been granted and may be granted again in the future. They refer mainly to cases where the conditions assumed in the By-laws do not actually obtain. Waivers of the loads set out in By-law 4 may be granted if, for example, in a workshop it could be proved that the actual load would be smaller than the required 150 lb. per sq. ft., but a condition would probably be made in this case that a notice be put up to prevent the actual load exceeding the reduced design load. The same condition refers to all stresses which are prescribed for special materials: e.g. concrete designation IA refers to a certain mix which must have an ultimate strength in preliminary tests, after 28 days, of 5,626 lb. per sq. in. and 3,750 lb. per sq. in. in the works test. The Council, however, will be willing to permit greater stresses than those specified if the concrete gives higher strengths on the preliminary or works tests. In a similar manner waivers are obtainable for high tensile structural steel where especially high yield-points can be guaranteed. In most cases the Council will grant waivers only on the provision that the work be carried out under special conditions that guarantee good results. The foregoing remarks are intended to give the reader an idea of the waivers that can be expected to be given by the London County Council. Such waivers, however, are in no case to be taken as precedents for any other case, although it is best to ascertain if a waiver has been granted in a special instance and to find out under what conditions it was granted. If the same conditions obtain and if the same standard of work will be adopted, it is reasonable to assume that the same waiver will be given.

This, however, does not restrict applications to cases for which waivers have previously been given, and one is always at liberty to apply for a waiver of any by-law whatsoever. Obviously, before doing this one should have sufficient and clear evidence that the alternative to which one desires to work will still ensure a high standard of stability and workmanship. These standards should be equal to those required by the By-laws or to the standard of any existing work, for in the latter case, a chain being no stronger than its weakest link, it clearly is illogical in making alterations in an existing building to provide certain structural units of very much greater strength than others on which they depend for the ultimate stability of the building.

A register is kept, at County Hall, of all waivers which are granted by the Council, and this register is open to inspection by the public. Further, at intervals—generally half-yearly—a list of waivers is published in the Council's Minutes.

Buildings Exempt from the By-laws

For the buildings enumerated in the succeeding notes, the By-laws and the Timber By-laws are not legally binding either on the Council or on the builder; but in the majority of cases it can be assumed that any design which adheres to the By-laws requirements will be approved. For ease of reference in the notes these buildings have been divided into four classes:

A: Buildings which come under the heading "special buildings" and which are, therefore, to be approved by the London County Council.

B: Buildings to be approved by the district surveyor.

C: Buildings to be approved by authorities other than the London County Council.

D: Buildings which do not require any approval as far as the construction is concerned.

The sections of the Act to which reference is made in the following notes are reprinted on pp. 212 *et seq.*

Group A.—Buildings which come under the heading "special buildings" and which, therefore, are to be approved by the London County Council.

(1) "Where a local authority, or a company, has statutory powers for the supply of electricity in any district in London, the buildings of that local authority or company, used as a generating station, or for works, shall be deemed to be special buildings to which the general provisions of Parts V, VI and VII of this Act and the 1st and 2nd Schedules thereof, do not apply; and plans thereof shall be submitted to the Council for their approval; and the Council shall have power to authorise the buildings to be erected of greater dimensions than 250,000 cu. ft., and in other respects to exempt the buildings from any of the provisions of this Act if they think fit." (Section 227 of the Act.)

(2) Any building for which special reasons can be given why the By-laws, or some of the By-laws, should not be applied. (Section 89 of the Act.)

(3) Buildings which are only temporary and for which a licence is desired from the London County Council. (Section 90 of the Act.)

Group B.—Buildings to be approved by the District Surveyor.

(1) Public buildings are included in this class and, defined under Section 5 :
" ' public building ' means :

" (*a*) a building used or constructed or adapted to be used as a church, chapel or other place of public worship, or as a school, college or place of instruction (not being merely a dwelling-house so used) or as a hospital, workhouse, public theatre, public hall, public concert-room, public ball-room, public lecture-room, public library or public exhibition room, or as a public place of assembly ; or used or constructed or adapted to be used for any other public purpose ; or

" (*b*) a building used or constructed or adapted to be used as an hotel, lodging-house, home, refuge or shelter where the building extends to more than 250,000 cu. ft. or has sleeping accommodation for more than one hundred persons ;

" and includes the buildings and premises of the Stock Exchange within the city." (Section 85 and Sections 86 and 87 apply also to such buildings.)

(2) Buildings to which the requirements of Section 57 (2) of the Act apply.

Group C.—Buildings to be approved by authorities, other than the London County Council.

(1) Bridges, piers, jetties, embankment walls and wharf and quay walls, and all other buildings referred to under Section 223, (1) to (9).

(2) " Every building, structure, or work vested in and in the occupation of his Majesty, either beneficially or as part of the hereditary revenues of the Crown or in trust for the public service or for public services." (Section 226 of the Act.)

(3) " Every building, structure or work vested in and in the occupation of any department of His Majesty's Government or of the Metropolitan Police, or of the trustees of the British Museum for public purposes or for the public service." (Section 226 of the Act.)

(4) " Every building, structure, or work vested in and occupied for the service of the Duke of Cornwall for the time being." (Section 226 of the Act.)

(5) " The lands, buildings, and property of :
(1) the Honourable Society of the Inner Temple ;
(2) the Honourable Society of the Middle Temple ;
(3) the Honourable Society of Lincoln's Inn ;
(4) the Honourable Society of Gray's Inn."
(Section 228 of the Act.)

Group D.—Buildings which need no approval as far as the construction is concerned.

(1) All buildings referred to under Section 223, (10) to (16).

(2) Wooden buildings which are exempted from the requirements of the Act by the provision of Section 91.

Any building exempt from the By-laws due to its purpose is only exempt as long as that purpose obtains. This means, for example, that buildings belonging to a railway company built for the purpose of the railway, but not in accordance with the By-laws, will have to be altered to agree with them if at any time they are used for a purpose other than that for which they were originally erected.

If any building is exempt, due to its being the property of the Crown or of any corporation mentioned before, and if, at any time, it passes into other hands, it will cease to be exempt.

CHAPTER 2

INDEX TO : BY-LAWS ; WELDING REGULATIONS ; TIMBER BY-LAWS ; SECTIONS OF LONDON BUILDING ACT, 1930

INDEX TO BY-LAWS

Schedule I. Slump test for consistence of concrete.
Schedule II. Preliminary cube tests of concrete.
Schedule III. Works cube tests of concrete.

Part III. Foundations and Sites

BY-LAW

26. Foundations:
    Piling.
27. Site treatment and filling.
28. Site covering.
29. Exterior filling.
30. Foundation bearing pressures.
31. External earth load.
32. Protection of metal in foundations.
33. Concrete foundations to walls or piers.
34. Pressure on plain concrete filling.
35. Pressure on plain concrete foundations.
36. No tension in plain concrete.
37. Site covering as floor.
38. Site precautions.

Part IV. Walls and Piers

Section 1: *General Requirements*

39. Walls and proportion of openings.
40. Materials of walls.
41. Restriction on hollow bricks or blocks.
42. Restriction on timber, etc., in walls.
43. Thickness of walls.
44. Thickness of walls.
45. Cavity walls.
46. Damp proofing.
47. Copings.
48. Notice to district surveyor re existing work.
49. Notice to district surveyor re thickening.
50. Walls to be to calculations or to Section 2 of Part IV.

Section 2: *Rules for Walls not determined by Calculations*

51. Materials.
    Oversailing walls.
    One floor only in roof.
    Storey heights.
    Distributed loads only.
    Ground levels.
    Recesses.
52. Length of walls.
53. Thickness of walls.
    Thickness of partition walls.
    External cavity walls.
    Required increase in wall thickness may be confined to piers.
54. Thickness of external and party walls in domestic buildings.
55. Thickness of external and party walls in warehouse buildings.
56. Separate or panel walls.
57. Underpinning.

Section 3 : Rules for Walls to Calculations

BY-LAW

58. Walls and piers in combination and on adjacent storey heights.
59. Storey heights for slenderness ratios.
60. Pressures on brick or block walls or piers.
    Pressures on plain concrete walls or piers.
    Pressures on other materials not designated.
61. Restriction on shear and tensile forces in walls and piers.
    Resistance of walls to wind pressure.
62. Underpinning.

PART V. STRUCTURAL STEEL

63. Structural steel to comply with By-laws.
64. Supports for steelwork.
65. Foundations for steelwork.
66. Protection of structural steel.
67. Scope of structural steel by-laws.
68. Casing of structural steel.
69. Column bases.
70. Ends of columns.
71. Bearing stresses for columns and widths of packing pieces.
72. Column splices.
73. Caps and bases to solid round columns.
74. Fabrication.
75. Erection.
76. Bolts.
77. Spacing of rivet or bolt holes.
78. Unsupported flange widths.
79. Tacking rivets.
80. Minimum thickness of plates in columns.
81. Permissible working stresses.
82. Working stresses in grillage beams.
83. Working stresses in filler floor beams.
84. Span of beams.
85. Column stresses for axial loads from slenderness ratios.
86. Effective column lengths for slenderness ratio.
87. Permitted stresses for eccentric loads.
88. Bending moments in columns proportional to stiffnesses.
89. Dissipation of column bending moments.
90. Increase in permissible stresses for wind loads.
91. Any other structural steel in a building.

PART VI. REINFORCED CONCRETF

92. Reinforced concrete to conform to By-laws.
93. Supports for reinforced concrete.
117. Construction of reinforced concrete work.
118. Tests.
115. Deflection, etc.
116. Restraint of compression members.
94. Foundations for reinforced concrete.
95. Protection of metal reinforcement.
96. Reinforcement :
    (i) Material.
    (ii) Uses.
    (iii) Preparation.

PART VII. CHIMNEY-SHAFTS

PART VIII. MISCELLANEOUS

PART IX. GENERAL

INDEX TO WELDING REGULATIONS[1]

CLAUSE

1. Plans and calculations.
   Conditions imposed by Council.
   Approval for each building necessary.
   Preliminary approval.
2. Structural steel.
3. Provision for effects of rigidity.
4. Nomenclature to be used.
5. Information required by Council.
6. Permissible stresses.
   Details of connections.
7. Restriction on square butt welds.
8. Permissible stresses in J and bevel butt welds.
9. Types of butt welds.
10. Gaps for butt welds.
11. Root faces for butt welds.
12. Angles for V butt welds.
13. Angles and gaps for bevel butt welds.
14. Shape of V butt welds.
15. Shape of J butt welds.
16. Butt welds in plates of different thicknesses.
17. Back run on single butt welds.
18. Reinforced and flush butt welds.
19. Throat thickness of butt welds.
20. Size and throat thickness of fillet welds.
21. Calculation of fillet welds.
22. Minimum effective length of fillet welds.
    Angles between surfaces of plates joined.
23. Definition of side fillet weld.
24. Definition of end fillet weld.
25. Definition of diagonal fillet weld.
26. Definition of T fillet weld.
27. Weld symbols.
28. Effective cross-section of welds.
29. Effective section modulus of welds.
30. Calculation of direct stress.
31. Calculation of bending moment stress.
32. Calculation of combined stresses.
33. Stresses at connections to be calculable.
34. Connections and welds to be accessible.
35. Welds on nominal members.
36. Joints to be sealed against weather.
37. Intermittent fillet welds permitted.
38. Welds in slots or holes.
39. Preference for side and end fillets in combination.
40. Spacing of side fillets in end connections.
41. Restriction on end fillets in end connections.
42. Minimum laps of plates.
43. Continuous fillets at ends of members.
44. Unsupported width of plate.

[1] In the explanations of the Welding Regulations given on pages 112 to 121 this arrangement, as it is given in the official publication, has not been retained, and clause numbers have been omitted from the explanations.

INDEX TO TIMBER BY-LAWS

INDEX TO EXTRACTS FROM THE LONDON BUILDING ACT, 1930

PART II

REGULATIONS, EXPLANATORY

CHAPTER 3

BY-LAW 1. DEFINITIONS. PART I. LOADINGS. BY-LAWS 2 TO 8

By-law 1. Definitions.—In this by-law there are set out the meanings of various expressions which are used in subsequent by-laws. These interpretations or definitions are to be taken throughout unless the context of any by-law indicates otherwise. Further definitions are given in Section 5 of the Act.

"Aggregate": all materials, stone and sand, etc., but not cement and water used for making concrete.

"Base" of a wall or pier: (see Fig. 1).

(1)

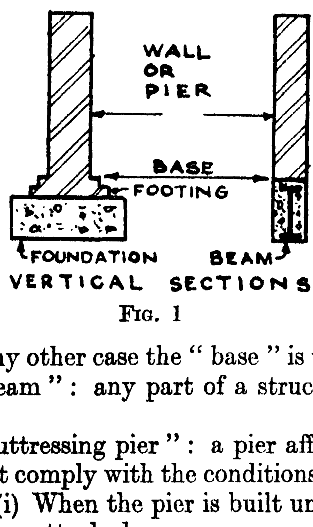


Fig. 1

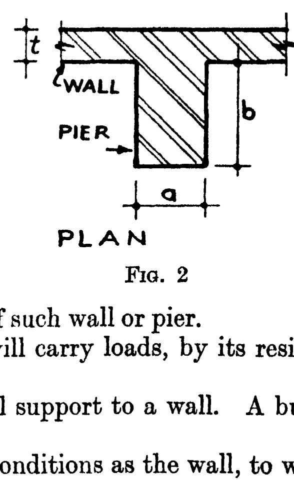


Fig. 2

(2) In any other case the "base" is the bottom of such wall or pier.

"Beam": any part of a structure which will carry loads, by its resistance to bending.

"Buttressing pier": a pier affording lateral support to a wall. A buttressing pier must comply with the conditions that:

A. (i) When the pier is built under similar conditions as the wall, to which it is attached,
   (ii) when the wall is built in accordance with the requirements of Section 2, Part IV of the By-laws, and
   (iii) when the pier carries only its own proportion of the distributed wall load, then:

$$\left.\begin{array}{l} a \text{ must not be less than } 2t \\ b \quad ,, \quad ,, \quad ,, \quad ,, \quad 3t \end{array}\right\} \text{(see Fig. 2)}$$

unless calculations show that a pier of less dimensions affords adequate lateral restraint.

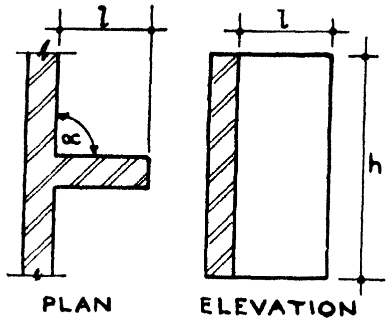
Or

B. (i) when the wall is built in accordance with the requirements of Section 2, Part IV of the By-laws,
   (ii) when the pier carries a load greater than its own proportion of the distributed wall load, or
   (iii) when for any reason the pier is not constructed under similar conditions as the wall, then:

the pier must be to dimensions determined by calculations, taking into consideration the vertical and lateral loads which it has to resist.

"Buttressing wall": A wall affording lateral support to another wall, such that
A. When built to the requirements of Section 2, Part IV of the By-laws:

FIG. 3



l must not be less than $\frac{1}{6}$h
$\propto$,, ,, ,, ,, 90°

unless calculations show that a wall of less length or having $\propto$ less than 90° affords adequate lateral restraint.

Or

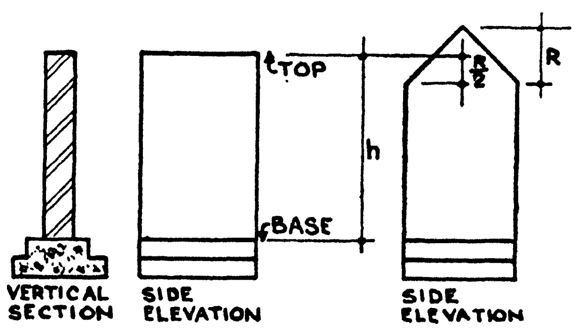
B. When not built to the requirements of Section 2, Part IV of the By-laws, the wall must be to dimensions determined by calculations, taking into consideration the vertical and lateral loads which it has to resist.

"Column," steel or reinforced concrete: any part of a structure which will carry loads by its resistances to compression in the direction of its length, and to bending due to such compressive loads being eccentric.

"Dead loading": the weight of permanent constructions such as floors, walls. partitions, etc.

"Height" of wall or pier:

FIG. 4



h = height.

"Incombustible material": material which neither burns nor gives off sufficient inflammable vapour to ignite when heated in accordance with British Standards Specification No. 476—1932.

"Lateral support" to a wall or pier: support which prevents movement of the wall or pier at the level of and in the direction of the support.

"Length" of any part of a wall: the greater horizontal dimension. If the

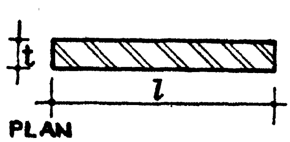


FIG. 5

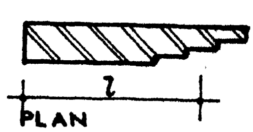


FIG. 6

horizontal section at any level is not a rectangle, the "length" is the average length at that level.

"Lintel":

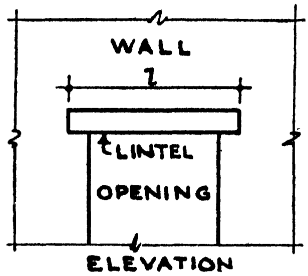


FIG.

l must not be greater than 8 ft.

"Load-bearing" in relation to any part of a building, including foundations:

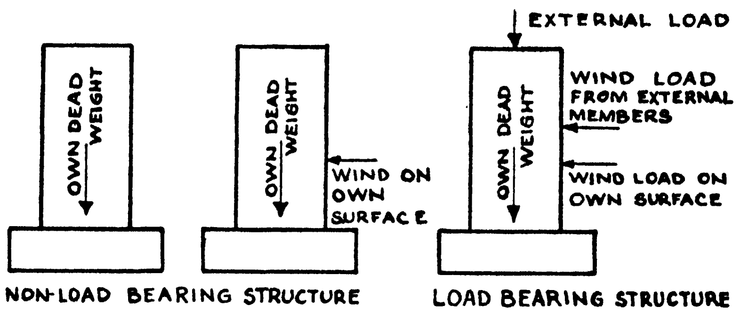


FIG. 8

"Mesh" in relation to measurement of materials: the mesh of a sieve complying with the British Standards Specification No. 410—1931.

"Partition wall": an internal wall which is not a division nor a party structure.

"Pier": see "Wall."

"Plain concrete": concrete not reinforced for the purpose of complying with the By-laws.

"The Principal Act": the London Building Act, 1930.

"Reinforced concrete": concrete reinforced for the purpose of complying with the By-laws.

"Required thickness" in relation to a wall: the thickness specified by the By-laws.

"Storey height" of a wall or pier:

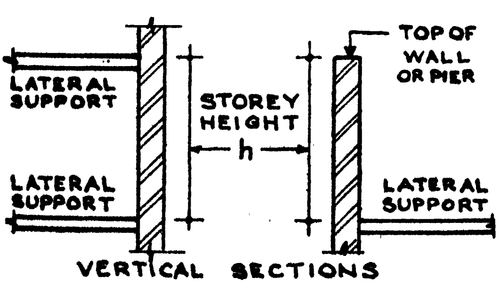


FIG. 9

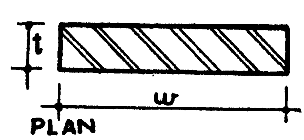


FIG. 10

h = storey height.

"Superimposed loading": all loading other than dead loading.

"Thickness" of a wall or pier: the less horizontal dimension, or if the horizontal section at any level is not a rectangle the " thickness " is the average thickness at that level.

" ' Wall ' . . . implies a length greater than six times the thickness." If this condition is not satisfied, the section is a " pier."

Or (see Fig. 5.)

" Wall " if l is greater than $6t$.

" Pier " if l is less than $6t$.

" Width " of any part of a pier : the greater horizontal dimension. If at any level the horizontal section is not a rectangle, then the " width " is the average width at that level.

See also the definitions given in Section 5 of the Act on p. 212.

PART I. BY-LAWS 2–8. LOADS

This part of the By-laws is devoted entirely to the question of loads. The amount of dead and superimposed loads to resist which the building must have adequate strength are set out with their governing conditions.

By-law 2. Permitted Stresses not to be exceeded.—The formal demand is made that every part of a building must be constructed to support and transmit safely all the dead and superimposed loads without stressing any of the materials beyond the limiting stresses specified by the By-laws.

This by-law is particularly important because it entitles the authorities to ask for stresses to be calculated in every part of a building, without exception. That is to say that, even in those cases in which the dimensions of a member may be determined by the specified empirical rules, the dimensions must not be such that the calculated stress in the member would exceed that which is permitted for the appropriate case.

For example, it will be seen that the thicknesses of walls can be determined either by Section 2 or Section 3 of Part IV. By-law 2 empowers the District Surveyor to insist on the determination of stresses by calculations as under Section 3, even when Section 2 has been adopted for the determination of the dimensions of the members.

By-law 3. Weights of Materials.—In order to calculate the dead loads, the weights of materials given in British Standards Specification No. 648—1935 are to be taken. However, should the district surveyor agree to other weights, this specification would not be used.

By-law 4. Superimposed Loadings.—This is one of the most important by-laws from the designer's point of view, in that it specifies in detail the superimposed loads against which provision must be made. The presentation of these loads has been re-arranged and set out in Table 1. There are many provisos and qualifying clauses in the actual by-law ; but in Table 1 index figures have been added giving references to the qualifying notes for any particular load.

It will be noticed that alternative loads have been specified for beams and slabs, and to assist the designer, criteria for the adoption of either have been given in Notes 11 and 12.

It will be seen also that reductions of superimposed loads transmitted by beams and slabs may be made in arriving at the loads which must be borne by columns, piers, walls and foundations. It is suggested that this reduction should not be made in any preliminary calculations because it frequently happens in the case of tall buildings, where the effect is greatest, that wind load stresses, which also are greatest, can just be accommodated (without any increase of members) by the permitted increased working stresses for wind effects plus this margin of reduced superimposed loads. Advantages of economy can usually be achieved for tall structures by applying this reduction in the final calculations.

TABLE 1

BY-LAW 4. SUPERIMPOSED LOADINGS FOR BUILDINGS [7, 8, 9, 10]

The small numbers refer to the Notes which follow

Class No.	Type of Building or Floor	SLABS: Superimposed Load, lb. per sq. ft. of floor area	SLABS: Special Load [6, 11]	BEAMS [4]: Superimposed Load, lb. per sq. ft. of floor area	BEAMS [4]: Special Load [5, 6, 12]
1 [8]	Rooms used for residential purposes; and corridors, stairs and landings within the curtilage of a flat or residence.	50	¼ ton per foot width of floor uniformly distributed over whole span.	40	1 ton uniformly distributed over span.
2 [8]	Offices; floors above entrance floor.	80	⅜ ton per foot width of floor uniformly distributed over whole span.	50	2 tons uniformly distributed over span.
3 [8]	Offices; entrance floor and floors below entrance floor; retail shops and garages for private cars of not more than 2½ tons net weight.	80	Ditto.	80	Ditto.
4 [8]	Corridors, stairs and landings not provided for in Class I.	100 [1]	Ditto.	100 [1]	Ditto.
5	Workshops and factories; garages for motor vehicles other than private cars of not more than 2½ tons net weight.	150 [1, 13]	⅜ ton per foot width of floor uniformly distributed over whole span, except garage floors [2].	120 [1]	2 tons uniformly distributed over span, except garage floors [2].
6	Warehouses; bookstores, stationery stores, etc.	200 [1, 13]	⅜ ton uniformly distributed per foot width.	200 [1]	2 tons uniformly distributed over span.
7 [8]	Any purpose not specified above.	Loading to be provided for to be ascertained to the satisfaction of the district surveyor.[13]			
Roofs 8	Flat roofs and roofs [3] inclined at an angle with the horizontal of not more than 20°.	50	—	30	—

Note 1.—Loadings to be to the satisfaction of the district surveyor, but not to be less than the values given in Table 1.

Note 2.—For garage floors load to be 1·5 times maximum possible combination of wheel loads; but each wheel load to be taken as not less than 1 ton.

Note 3.—Live load on roof in which α is greater than 20° (live load includes wind load): (See Fig. 11.)

(*a*) The *roof construction* shall be designed for P lb. per sq. ft. over AB on the windward side and P_1 lb. per sq. ft. over BC on the leeward side, such that
   (i) P and P_1 do not act simultaneously, but separately;
   (ii) P must not be less than 15 lb. per sq. ft.; and
   (iii) P_1 must not be less than —10 lb. per sq. ft.

(*b*) *All other construction* shall be designed for P_2 lb. per sq. ft. over AC, the horizontal projection of the sloping surfaces, such that P_2 must not be less than 10 lb. per sq. ft.

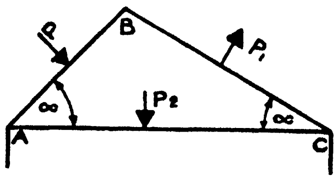


Fig. 11

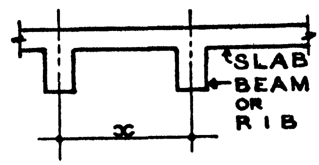


Fig. 12

Note 4.—If x is not greater than 2 ft. 6 in., beams or ribs shall be designed to resist the full superimposed load or the special load required for the slab. See Note 11, and Fig. 12.

Note 5.—The "special loads" do not apply to non-load-bearing beams, such as beams which tie columns, etc.

Note 6.—The reactions due to the special loads need not be applied to the columns, piers, walls or foundations, but the reactions of the normal superimposed loads must be taken into account.

Note 7.—In all cases of floors where the positions of partitions are not definitely located in the design, a uniformly distributed load sufficient to allow for them shall be added to the dead floor load. For all floors of rooms used for offices the minimum total allowance for internal partitions shall be at the rate of 20 lb. per sq. ft. of floor area.

Note 8.—For buildings over two storeys in height of the classes 1, 2, 3 (and 4 where the superimposed beam load is taken as 100 lb. per sq. ft.; and 7 where the superimposed beam load is not more than 100 lb. per sq. ft.) in which the total loads are transmitted through each storey to the foundation by either:

(*a*)		(*b*)
skeleton framework of structural steel or reinforced concrete	or	frame of structural steel or reinforced concrete and party wall or party walls

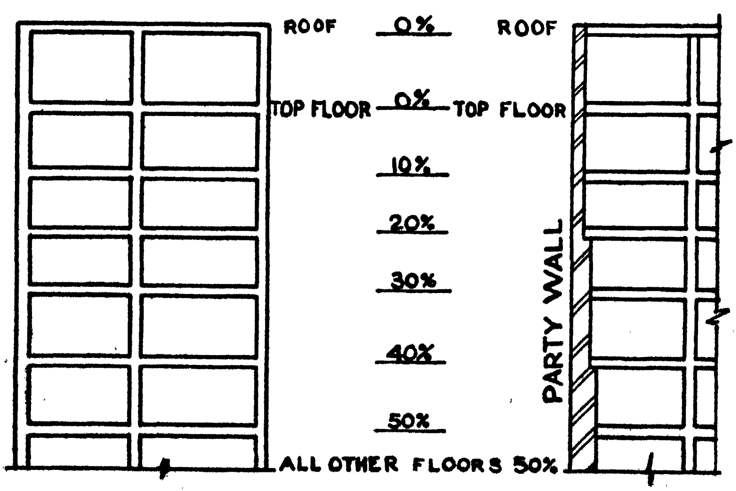


Fig. 13

then the reductions in Fig. 13 of superimposed loads (not of dead loads nor of partition loads, etc.) may be made in calculating the total loads which must be carried on columns, piers, walls and foundations. These reductions may be calculated from the proportion of floor area carried by each foundation wall, pier or column.

Note 9.—In any case where the actual superimposed load on any floor or roof is to exceed that specified for the floor or roof, such greater load shall be provided for in accordance with By-law 2.

In the case of any floor intended to be used for a purpose for which the superimposed load is not specified, the superimposed load to be carried on that floor, belonging to class 7, is to be agreed to by the district surveyor and shall be provided for in accordance with By-law 2.

The following superimposed loads will be approved, generally, by the district surveyor for certain purposes not specified in the By-laws, namely for churches, schools, reading-rooms and galleries and similar buildings :

TABLE 2

SLABS		BEAMS	
Distributed Load	Special Concentrated Load	Distributed Slab Load	Special Concentrated Load
80 lb. per sq. ft.	$\frac{3}{8}$ ton	70 lb. per sq. ft.	2 tons

Assembly halls, drill halls, dance halls, gymnasia, light workshops, public spaces in hotels, restaurants, hospitals, theatres and cinemas, and grand-stands belong to class 4.

Note 10.—In cases where a superimposed load may move, proper provision in accordance with these By-laws to the satisfaction of the district surveyor shall be made for all effects of such movement, including vibration, impact, acceleration and

TABLE 3

CRITERIA FOR SPECIAL AND SUPERIMPOSED LOADS ON SLABS

Building	Slab
Class 1	Special load for spans up to 11·2 ft. Superimposed ,, ,, over 11·2 ft.
Class 2	Special ,, ,, up to 10·5 ft. Superimposed ,, ,, over 10·5 ft.
Class 3	Special ,, ,, up to 10·5 ft. Superimposed ,, ,, over 10·5 ft.
Class 4	Special ,, ,, up to 8·4 ft. Superimposed ,, ,, over 8·4 ft.
Class 5	Special ,, ,, up to 5·6 ft. Superimposed ,, ,, over 5·6 ft. Except for garages for heavy vehicles—see Note 2.
Class 6	Special load for spans up to 4·2 ft. Superimposed ,, ,, over 4·2 ft.

deceleration. The loads for garages specified in Table 1 already provide for the effects of such impacts. In all other cases the impact due to a moving load can be taken into consideration by the use of a coefficient which will lie within the limits of 1·2 and 2·0. Normally acceleration and deceleration cause horizontal forces which can be computed.

Note 11.—The floor slab must be designed to resist the greater bending moment and the greater shear force which may be set up by either the "superimposed load" or the "special load." In general, the member should be investigated for both of the loads; but as a rule the special load is the determining one for short spans, and the limits to which this applies are set out in Table 3 on page 25.

Note 12.—All beams must be designed to resist the greater bending moment and the greater shear force which may be set up by either the "superimposed load" or the "special load."

In general the member should be investigated for both of the loads; but the following Table sets out the governing conditions.

In the table l_1 is the uniform width of slab supported by the beam, and l_2 is the span of the beam. If the width of slab which the beam supports is not uniform throughout the length of the beam, then these conditions are no longer applicable.

TABLE 4

CRITERIA FOR SPECIAL SUPERIMPOSED LOADS ON BEAMS

Building	Beam
Class 1	Special load for values of $l_1 \times l_2$ up to 56. Superimposed " " $l_1 \times l_2$ over 56.
Class 2	Special " " $l_1 \times l_2$ up to 89·6. Superimposed " " $l_1 \times l_2$ over 89·6.
Class 3	Special " " $l_1 \times l_2$ up to 56. Superimposed " " $l_1 \times l_2$ over 56.
Class 4	Special " " $l_1 \times l_2$ up to 44·8. Superimposed " " $l_1 \times l_2$ over 44·8.
Class 5	Special " " $l_1 \times l_2$ up to 37·3. Superimposed " " $l_1 \times l_2$ over 37·3. Except for garages for heavy vehicles—see Note 2.
Class 6	Special load for values of $l_1 \times l_2$ up to 22·4. Superimposed " " $l_1 \times l_2$ over 22·4.

Note 13.—See By-law 5.

By-law 5. Notice indicating Loads over 100 lb. per sq. ft.—In every room of which the superimposed floor load is required to be greater than 100 lb. per sq. ft. a notice stating the floor load must be fixed in a permanent and conspicuous position. The notice must read:

"London Building Act (Amendment) Act, 1935.

"Notice.

"The floor of this room is constructed for superimposed loading to an intensity not exceeding . . . pounds on any square foot of its surface."

By-law 6. Wind Loads.—A building which is actually exposed or which may become exposed to the wind must be capable of resisting the wind pressures.

The wording of this by-law does not represent correctly the intention of the authorities. The following explanation is, therefore, not in strict agreement with the wording of the by-law, but it represents the intention.

There are two different cases which must be distinguished. The first is that in which the height is greater than twice the base. In this case an exact calculation of wind loads is to be made, on the basis that the horizontal wind pressure is 15 lb. per sq. ft. on the upper two-thirds of the exposed surface, measured to the general roof level, whilst it is 25 lb. per sq. ft. on the total surface of all projections above. (See Fig. 14.)

The general roof level is the top of any horizontal roof or parapet which may extend above the roof, or the ridge of a pitched roof.

For a building of uniform elevation up to the general roof level the normal pressure is to be taken for the top two-thirds of the height. However, the by-law specifies that the pressure is to be taken on the upper two-thirds of the surface, not of the height. Therefore, in a building which is longer at the lower levels than it is near the roof, the height over which the pressure must be assumed to be applied will be greater than two-thirds of the actual height. Attention must be given to this fact.

Where a roof slopes at an angle of 20° or more, a vertical load due to wind pressure is to be considered as set out in Note 3 (*b*) to By-law 4.

In the second case an exact calculation is not required if the district surveyor is satisfied that the wind loads can be transferred safely to the ground. Such transference of wind loads to the ground can be made by external and internal cross walls if they are not further apart than set out in Section 2 of Part IV (By-laws 51 to 57), or by such cross walls in combination with stiff floors.

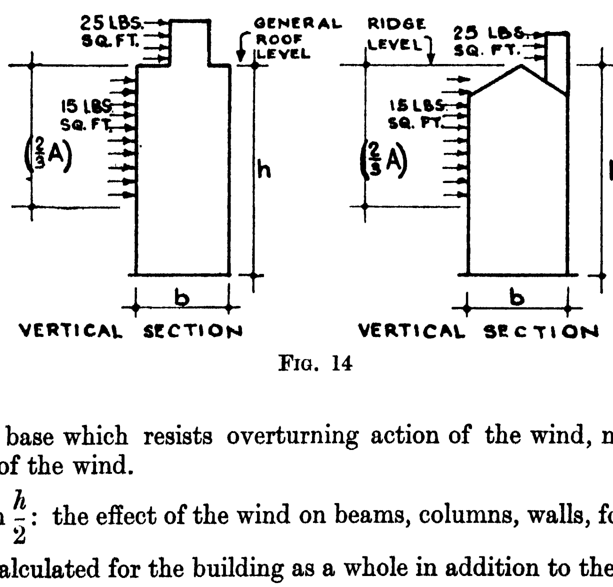


FIG. 14

b = width of base which resists overturning action of the wind, measured in the direction of the wind.

If b is less than $\frac{h}{2}$: the effect of the wind on beams, columns, walls, foundations, etc., must be calculated for the building as a whole in addition to the local effects.

If b is greater than $\frac{h}{2}$, and if the district surveyor is satisfied that the wind loads can be transferred safely to the earth, then :

Only local effects (for example, bending in wall panels between supports need) to be calculated.

Note :—By-laws 90 and 103 for structural steel and reinforced concrete construction respectively permit certain working stresses to be increased by $33\frac{1}{3}$ per cent. if loads and bending moments are increased only by the effects of wind.

By-law 7. Angle of Dispersion of Load.

α = angle of dispersion of load through the material from the direction of the imposed load.

α must not be greater than 45° for the specified materials.

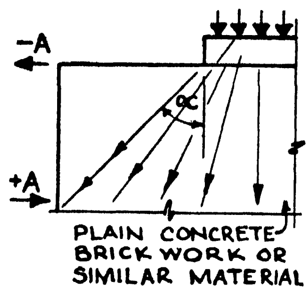


Fig. 15

In Fig. 15 if the material through which the load is being dispersed is arranged equally on both sides of the centre of gravity of the load, then the structural unit will be in equilibrium. But on the other hand, if the material is disposed only on one side so that it gets an eccentric load, then a dispersion up to 45° can be assumed only if there are two equal and opposite forces (—A and +A in Fig. 15) applied at the top and bottom of the block of material.

By-law 8. Loads not to exceed Those for which Building was constructed.—At no time may any building or part of it be subjected to greater loads than those for which it was constructed unless the district surveyor should require such loads for the purpose of testing.

CHAPTER 4

PART II. MATERIALS. BY-LAWS 9–25

THE requirements for all of the building materials covered by the By-laws are set out in By-laws 9 to 25, and in addition there are attached three schedules in which the standard methods of making concrete tests are described. The Reinforcement Waiver Regulations and the Structural Steel Waiver Regulations have also been included by the authors in this Part.

By-law 9. Aggregates for Reinforced Concrete.—For the making of reinforced concrete, the aggregate must fulfil the following conditions :

"Aggregate shall be sand and gravel or crushed natural stone. It shall be hard, strong and durable and shall be reasonably clean and free from clay, organic matter, coal and coal residues (including clinker, ashes, coke-breeze, pan-breeze, slag and other similar material), copper slag, forge breeze, dross (and other similar material), soluble sulphates (including gypsum and other similar material), porous material and other materials liable to reduce the strength of durability of the concrete or to attack the steel reinforcement."

Table 5 specifies the sizes of aggregate.

TABLE 5

PERCENTAGES OF AGGREGATE WHICH MUST BE RETAINED ON SIEVES OF SPECIFIED MESHES

	MESH		
	No. 100	$\frac{3}{16}$ in.	Minimum Clear Distance between Reinforcing Rods minus $\frac{1}{4}$ in.
Fine aggregate .	95 per cent.	0 per cent.	0 per cent.
Coarse aggregate .	100 per cent.	100 per cent.	0 ,, ,,

"Aggregate shall be so graded between the limits as to make a dense concrete of the specified proportions and consistence that will work readily into position without segregation and without the use of an excessive water content."

In cases in which the chemical composition of the aggregate is in any doubt, chemical tests must be made, and they should be repeated whenever the inspection of the aggregate indicates that there is inconsistency in the materials supplied.

Fine aggregate and sand can be tested by the two following standard methods :

Appendix II.[1]—

"*Standard Method of Test for Organic Impurities in Sands for Concrete.*

"This method of test is an approximate method of determining the presence of injurious organic compounds in natural sands which are to be used in cement,

[1] From the Report of the Reinforced Concrete Structures Committee of the Building Research Board, Department of Scientific and Industrial Research.

mortar or concrete. The principal value of the test is to furnish a warning that further tests of the sands are necessary before they are approved for use.

"A representative test sample, weighing about 1 lb., shall be obtained by quartering. A 12-oz. graduated clear glass bottle shall be filled to the $4\frac{1}{2}$-oz. mark with the sand to be tested. A 3 per cent. solution of sodium hydroxide in water shall be added until the volume of the sand and liquid indicated after shaking is 7 liquid oz. The bottle shall be stoppered, shaken vigorously and then allowed to stand for 24 hours.

"A standard colour solution may be prepared by adding 2·5 c.c. of a 2 per cent. solution of tannic acid in 10 per cent. alcohol to 97·5 c.c. of a 3 per cent. sodium hydroxide solution. This should then be placed in a 12-oz. bottle, stoppered, shaken vigorously and allowed to stand for 24 hours.

"After standing 24 hours, the colour of the clear liquid above the sand shall be compared with the colour of the standard colour solution prepared at the same time, and in accordance with the method described above, or with glass or other suitable standard of a colour similar to that of the standard solution."

Appendix III (A).[1]—

"*Standard Method of Decantation Test for Sand and Other Fine Aggregates*

"This method of test covers the determination of the total quantity of silt, loam, clay, etc., in sand and other fine aggregates.[2]

"The sample must contain sufficient moisture to prevent segregation and shall be thoroughly mixed. A representative portion of the sample sufficient to yield approximately 500 grams of dried material, shall then be dried to a constant weight at a temperature not exceeding 110° C. (230° F.). The dried material shall be placed in a pan approximately 9 in. (230 mm.) in diameter, and not less than 4 in. (102 mm.) in depth, and sufficient water added to cover the sample (about 225 c.c.). The contents of the pan shall be agitated vigorously for 15 seconds and then be allowed to settle for 15 seconds, after which the water shall be poured off, care being taken not to pour off any sand. This operation shall be repeated until the wash water is clear. As a precaution, the wash water shall be poured through a B.S. No. 200—mesh sieve and any material retained thereon returned to the washed sample. The wash sand shall be dried to a constant weight at a temperature not exceeding 110° C. (230° F.).

"The results shall be calculated from the formula: Percentage of silt, clay, loam, etc. =

$$\frac{\text{Original dry weight} - \text{weight after washing}}{\text{Original dry weight}} \times 100.$$

When check determinations are desired, the wash water shall be evaporated to dryness, the residue weighed and the percentage calculated from the formula: Percentage of silt, loam, clay, etc. =

$$\frac{\text{Weight of residue}}{\text{Original dry weight}} \times 100.$$

Appendix III (B).[1]—

"*Preliminary Field Method of Decantation Test for Sand and Other Fine Aggregates*

"This method of test is intended as a guide to the quantity of silt, loam, clay, etc., and where an aggregate passes the test it shall be deemed unnecessary to carry out the standard test given in Appendix III (A).

[1] From the Report of the Reinforced Concrete Structures Committee of the Building Research Board, Department of Scientific and Industrial Research.

[2] "This determination of the percentage of silt, clay, loam, etc., will include all water-soluble material present, the percentage of which may be determined separately as desired."

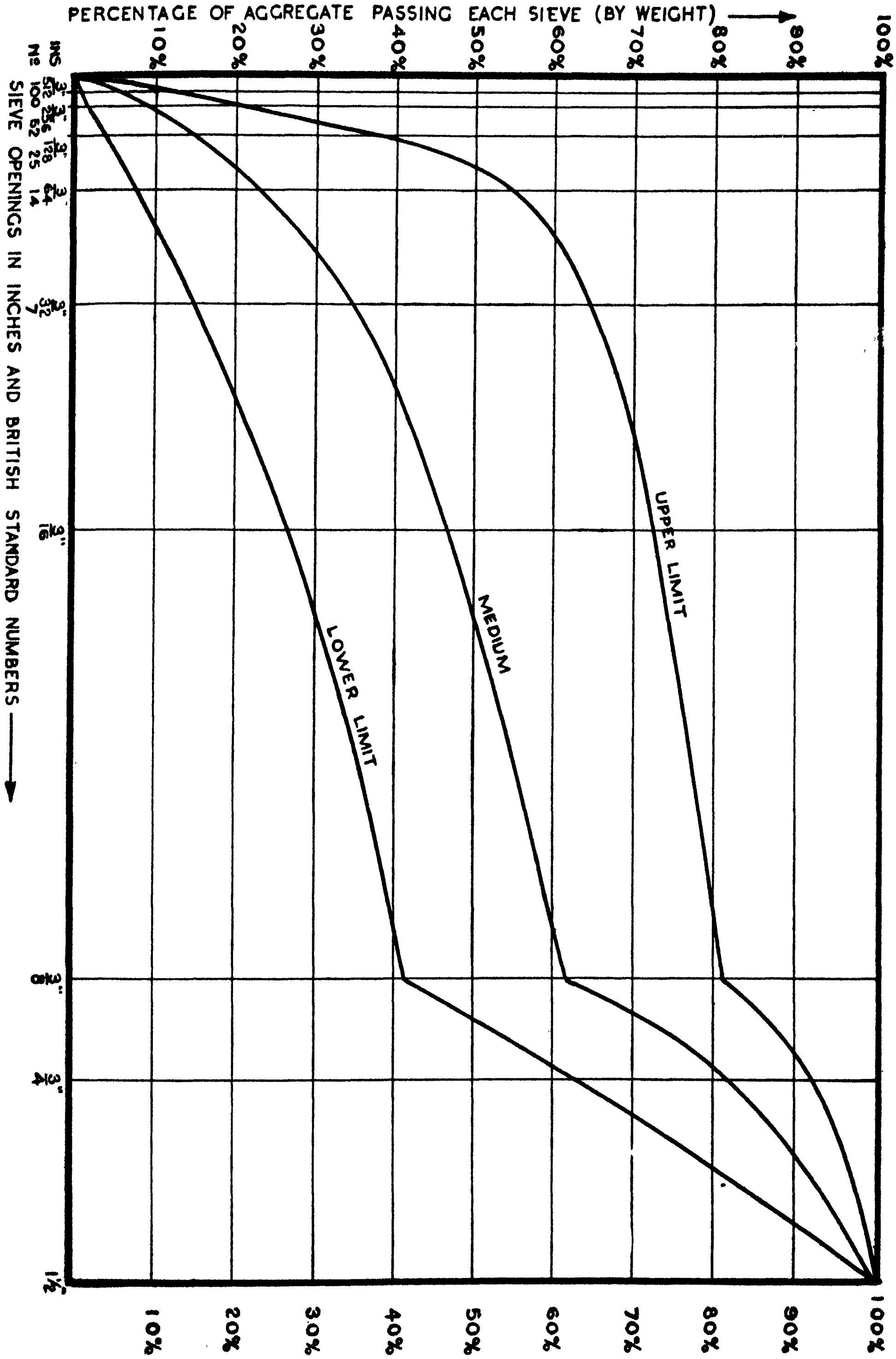

PERCENTAGE OF AGGREGATE PASSING EACH SIEVE (BY WEIGHT)
10% 20% 30% 40% 50% 60% 70% 80% 90% 100%
SIEVE OPENINGS IN INCHES AND BRITISH STANDARD NUMBERS
UPPER LIMIT
MEDIUM
LOWER LIMIT

FIG. 16

"A sample of the sand to be tested shall be placed in a 200-c.c. measuring cylinder, filling it up to the 100-c.c. mark.

"Clean water shall be added up to the 150-c.c. mark. The mixture shall be shaken vigorously and the contents allowed to settle for 1 hour.

"The volume of silt visible at the surface of the sand shall be noted and recorded as the percentage volume of silt in the sand.

"The aggregate shall be deemed satisfactory when this percentage volume does not exceed 6 per cent."

In order to ensure a dense concrete the aggregate is to be properly graded within the limits given in Table 5. Such grading is defined by the curves shown in Fig. 16.

By-law 10. Aggregate for Plain Concrete.—"Aggregate for plain concrete shall consist of such materials as are specified in By-law 9 or of hard well-burned brick, hard well-burned tile, pumice or other material which the district surveyor may approve as of like suitability and shall be so graded and contain sand in such proportion as to produce a dense concrete."

The limitations of size imposed by By-law 9 do not apply.

By-law 11. Sand.—"Sand shall be clean and shall be composed of hard, silicious grains reasonably free from clay or any animal, vegetable or bituminous matter. The grains shall be of such a size as to pass through a $\frac{3}{16}$-in. mesh."

Sand is an alternative to "fine aggregate," and all rules and tests given in By-law 9 for fine aggregate refer also to "sand."

By-law 12. Cement.—Cement may consist of Portland cement complying with British Standard Specification No. 12—1931, Portland blast-furnace cement complying with British Standard Specification No. 146—1932, or high-alumina cement; but no two such cements may be used in combination.

The By-law gives the following specification for high alumina cement:

"High-alumina cement shall consist of aluminous and calcareous materials which have been fused to a molten state and ground to such a degree of fineness that the cement will not leave residue of more than 12 per cent. by weight on a No. 170 mesh and not more than 1 per cent. on No. 72 mesh. The cement shall contain not less than 35 per cent. by weight of alumina and the ratio of percentage by weight of alumina to that of lime shall be not less than 0·9. When gauged with 22 per cent. by weight of water it shall not begin to set before the expiration of 2 hours, but shall begin to set within 6 hours of gauging, and the final setting shall take place within 2 hours of the initial setting. The strength of high-alumina cement shall be such that when a mortar is composed of one part by weight of high-alumina cement to three parts by weight of white Leighton Buzzard sand graded to pass No. 18 mesh and stop on a No. 25 mesh, and the whole is gauged with a weight of water equal to 8 per cent. of the dry materials, such mortar shall have a tensile strength of not less than 475 lb. per sq. in. within 24 hours after gauging, and within 7 days its tensile strength shall have increased and shall be not less than 550 lb. per sq. in."

For all cements there is a requirement that:

"Wherever cement is used it shall not be moved or disturbed after 1 hour from the time it has come into contact with water until it has set hard."

By-law 13. Water.—"Water used shall be clean and free from deleterious matter." Water taken from the London mains can be considered satisfactory, but in other cases it may be necessary to make chemical analyses.

By-law 14. Concrete.—Concrete is to consist of aggregate and cement mixed with water, the amount of water when Portland or Portland blast-furnace cements are used being just sufficient to bring the concrete to a uniform colour and suitability, and when high-alumina cement is used being just sufficient to produce a uniform and sound concrete. Concrete made from high-alumina cement must be kept wet for 24 hours after it has been mixed.

The following values, showing the strengths of vibrated and hand-rammed concretes in relation to the water contents, are based on published results of the Department of Scientific and Industrial Research tests and on tests made during the last 15 years on the Continent. It must be understood that the figures relate only to one typical concrete; but whilst a similar depreciation of strength with increased water content is to be expected for any concrete, the actual strength values depend in every case on the particular materials used.

TABLE 6

Gallons of Water per cwt. of Cement	Water Cement Ratio	Strength: lb. per sq. in.	
		Vibrated Concrete	Hand-rammed Concrete
3·92	0·35	4400	2500
4·48	0·40	6200	3500
5·04	0·45	5700	4300
5·60	0·50	4500	4400
6·16	0·55	3600	3300
6·72	0·60	2800	2500
7·28	0·65	2400	2200
7·84	0·70	2200	2000

The reason why the maximum strength is to be expected in vibrated concrete at 40 per cent. of water to cement and in hand-rammed concrete at 50 per cent. is that hand-rammed concrete with 40 per cent. water is still likely to develop voids, which diminish the ultimate strength.

The above figures for strength refer to 28-days tests on concrete of 1 : 2 : 4 mix. Actually the differences due to the amount of water diminish slightly if tests are taken after 4 or 6 months; but there is no ultimate convergence, and even after a period of 10 years the difference between the strength, say, of hand-rammed concrete with 70 per cent. water will be still only about 55 per cent. of that of similar concrete with 50 per cent. water.

This Table explains at the same time the advantage of vibrated concrete, which results mainly from the fact that a much smaller water cement ratio can be adopted; conversely there is little advantage in vibrating a very wet concrete. The average increase in ultimate strength by means of vibration can be estimated as approximately 28–30 per cent.

The requirements for "suitability" are of great importance for reinforced concrete work. The amount of water must be sufficient to ensure that the materials fill every void and that they may be deposited sufficiently well round any reinforcement to secure that correct and adequate bond will be developed.

Attention is drawn to the fact that the above-mentioned ratios refer to cement and water only, and are taken by weight. Under such conditions the proportions of the strengths remain practically the same for different mixes.

The water content can be suitably expressed in percentages, of volume and for this purpose 10 per cent. may be taken as 0·7 gallons for 112 lb. (one bag) of cement. This method of proportioning the water content, however, will be correct only if the cement and also the aggregate have been stored absolutely dry, a procedure which is possible only in large building operations.

In all other cases the measurement by means of the slump test is suggested. This is given in Schedule I. The slump, while not a scientific means of determining the water content, is sufficient for all practical purposes; but it must be kept in mind that the slump depends also on the size of aggregate, whilst to no great extent on the amount of cement used.

The authors suggest that the slump should be kept as small as possible, for shrinkage and other practical reasons. Avoidance of blemishes on the plaster or other outside finish, higher insulation and waterproof qualities, etc., all derive from low water content. The values given in Table 7 should not be exceeded.

TABLE 7

SLUMP VALUES

Size of Aggregate	Slump
$\frac{3}{4}$ in.	2 in.
$\frac{1}{2}$ in.	2·5 in.
$\frac{3}{8}$ in.	3 in.
$\frac{1}{4}$ in.	4 in.

There must be no segregation of the materials during the placing of concrete. It may be avoided, if the liquid concrete is not allowed to fall into its position from an excessive height. The vertical fall should be limited to 4–6 ft., and whenever it is necessary to drop concrete from a greater height it should be remixed. On a slope of 45°, however, the drop may be increased to 8–10 ft. No definite rules can be given for certain methods of transporting concrete, as for instance by pumping, and such methods can only be adopted on their merits.

Before the cement takes its " initial " set the concrete must be consolidated by punning, rodding, vibrating, or other similar means.

After consolidation the concrete must be protected from frost, heat, vibration, running water, etc., or from any other agency which would reduce its strength or cause voids ; and it must remain undisturbed. (The By-law does not state how long the concrete is to be left undisturbed, but it should remain so at least until the " final set " has taken place, and any formwork should be left securely in position for 3 weeks on the soffits of beams and slabs and for one week on the sides of columns and walls.) Struts should be left under the soffits of beams for 4 weeks.

All the foregoing figures are for concrete of ordinary Portland cement. They may, however, be reduced by 50 per cent. if rapid-hardening cement is used, provided that the minimum strength required after 28 days for the appropriate mix in Table 8 or 9 is actually obtained after 14 days.

Any formwork used must be sufficiently strong to withstand the depositing, punning and consolidating of the concrete without deformation. Concreting, i.e. mixing, placing or setting, must not be carried out when the temperature is below 40° F. Also, concreting should not be carried out even on days when the temperature may rise to over 40° F. but when it would remain so for only an hour or two. Where it is necessary to place concrete in such weather, the newly deposited concrete should be covered immediately by thick layers of sacks or other similar materials and such covering should be left in position for several days. Although a great amount of the setting actually takes place during the first day or so after the placing, during which time the greatest damage can be done to the concrete by the cold, this period of relatively rapid setting is considerably prolonged when the temperature is low. Therefore it is essential that concrete be left protected for as long as possible. Further, under such conditions and for the same reason shuttering should be left in position for a longer period. Another fact to which attention must be given, is that in the setting of concrete an appreciable amount of heat is liberated. Thus if the concrete is in massive units, there is a great tendency for the interior to remain at a considerably higher temperature than the surfaces which are chilled by contact with the cold, and unless proper protection is provided, cracks are likely to occur, or alternatively such disturbance of the partially set concrete may take place that considerable damage may result.

Depending on the proportions of aggregate to cement, and on the care in making, concretes have been given ten designations, namely I to VII and IA to IIIA.

Concretes IA, IIA and IIIA and I, II and III may be used for all structural purposes. Concretes IV and V may be used for all purposes other than for reinforced concrete; and concretes VI and VII may be used only as plain concrete filling material. No concrete of weaker mix than that of concrete VII may be used for any purpose in connection with the construction of a building or chimney-shaft.

The proportions of cement : fine aggregate : coarse aggregate and the required test strengths are set out in Tables 8 and 9 for concretes of each designation, together with some proportions for permissible intermediate concretes. For these and other intermediate mixes the volume of the coarse aggregate must be twice that of the fine, unless, however, in any particular case the district surveyor should allow or require a variation in the amount of coarse aggregate within the limits of 1½ times and 2½ times the volume of the fine aggregate. The ratio of the total volume of aggregate to that of the cement for any such special mix must be taken to find by its proportion to the corresponding ratios of the tabulated mixes, the test strength required under this by-law. For any normal intermediate mix the test strength will be found by direct proportion from neighbouring mixes.

Slump and works cube tests, to be carried out in the manners specified in Schedules I and III of this part, may be required by the district surveyor to prove the quality of any concrete.

Concretes I, II and III, given in Table 8, and concretes intermediate between I and III are defined as " ordinary concrete."

TABLE 8

DESIGNATION, MIX AND TEST STRENGTH

Concretes I to III are " ordinary concrete."

Designation of Concrete	Popular Reference	Cu. ft. of aggregate per 112 lb. cement		Minimum Resistance to Crushing in lb. per sq. in. within 28 days after mixing
		Fine	Coarse	
I	1 : 1·0 : 2·0	1¼	2½	2925
	1 : 1·1 : 2·2	1⅜	2¾	2850
	1 : 1·2 : 2·4	1½	3	2775
	1 : 1·3 : 2·6	1⅝	3¼	2700
	1 : 1·4 : 2·8	1¾	3½	2625
II	1 : 1·5 : 3·0	1⅞	3¾	2550
	1 : 1·6 : 3·2	2	4	2490
	1 : 1·7 : 3·4	2⅛	4¼	2430
	1 : 1·8 : 3·6	2¼	4½	2370
	1 : 1·9 : 3·8	2⅜	4¾	2310
III	1 : 2·0 : 4·0	2½	5	2250
IV	**1 : 6**	**7½**		**1480**
	1 : 7	**8¾**		**1295**
V	**1 : 8**	**10**		**1110**
	1 : 9	**11¼**		**925**
VI	**1 : 10**	**12½**		**740**
	1 : 11	**13¾**		**555**
VII	**1 : 12**	**15**		**370**

TABLE 9

DESIGNATION, MIX AND TEST STRENGTH.

" Quality A Concrete "

Designation of Concrete	Popular Reference	Cu. ft. of Aggregate per 112 lb. Cement		Minimum Resistance to Crushing in lb. per sq. in. within 28 days after mixing	
		Fine	Coarse	Preliminary Test	Works Test
IA	1 : 1·0 : 2·0	$1\frac{1}{4}$	$2\frac{1}{2}$	5625	3750
	1 : 1·1 : 2·2	$1\frac{3}{8}$	$2\frac{3}{4}$	5490	3660
	1 : 1·2 : 2·4	$1\frac{1}{2}$	3	5355	3570
	1 : 1·3 : 2·6	$1\frac{5}{8}$	$3\frac{1}{4}$	5220	3480
	1 : 1·4 : 2·8	$1\frac{3}{4}$	$3\frac{1}{2}$	5085	3390
IIA	1 : 1·5 : 3·0	$1\frac{7}{8}$	$3\frac{3}{4}$	4950	3300
	1 : 1·6 : 3·2	2	4	4815	3210
	1 : 1·7 : 3·4	$2\frac{1}{8}$	$4\frac{1}{4}$	4680	3120
	1 : 1·8 : 3·6	$2\frac{1}{4}$	$4\frac{1}{2}$	4545	3030
	1 : 1·9 : 3·8	$2\frac{3}{8}$	$4\frac{3}{4}$	4410	2940
IIIA	1 : 2·0 : 4·0	$2\frac{1}{2}$	5	4275	2850

In the same way, concretes IA, IIA and IIIA and their intermediate mixes are defined as " quality A concretes." In addition to the slump and works cube tests of quality A concretes, the district surveyor may require preliminary cube tests to be carried out before work begins and whenever any change in materials or proportions is made. Such preliminary tests are specified in Schedule II to Part II. A record of works tests for quality A concrete showing the strength of the concrete in any part of the structure must be kept on the work.

It will be observed that concretes I and IA, concretes II and IIA and concretes III and IIIA are of the same compositions ; but it is rightly assumed, owing to the greater care which must, in accordance with the By-laws, be exercised in the employment of quality A concrete, that higher strengths will be produced. If, however, in spite of such special precautions the specified strengths required for quality A concrete should not be attained, it is still permissible to use such concretes if the working stresses are modified accordingly. It is obvious, however, that in no case may the test strengths be less than those specified for ordinary concrete.

PART II. SCHEDULES

In the actual By-laws the three schedules describing the making of the concrete tests are attached to the end of Part II, after By-law 25. They have, however, been brought forward to this position so that reference from the by-laws relating to concrete, to which they apply, may be made easier.

Schedule I. Slump tests for Consistence of Concrete.—This test is for use in determining the consistence of concrete both in the laboratory and on the works. Samples tested are to be representative of the concrete being used, and the inside of the mould is to be thoroughly cleaned before the test is started.

The internal surfaces of the mould, which must be smooth, must be to the dimensions given in Fig. 17, and handles and foot pieces are to be provided. Fig. 17 illustrates a type of mould most frequently used in practice.

To fill the mould it is first placed on a flat non-absorbent surface and held there during filling by the operator's feet, placed on the foot pieces. The concrete is to be poured in, in four layers, each to be about 3 in. deep and each to be puddled twenty-five

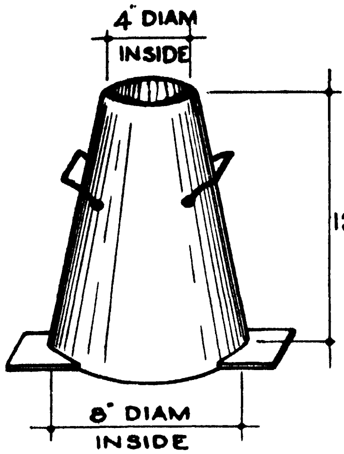


FIG. 17

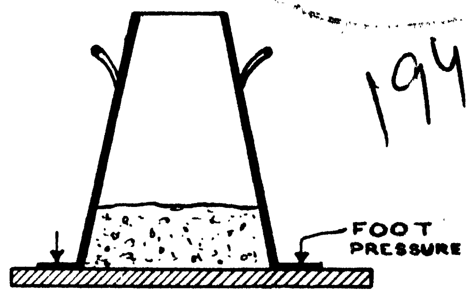


FIG. 18

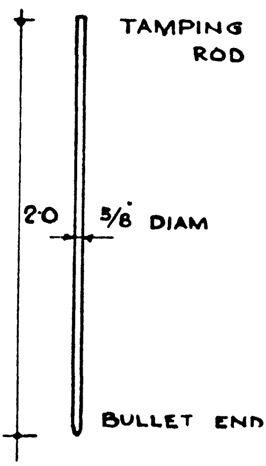


FIG. 19

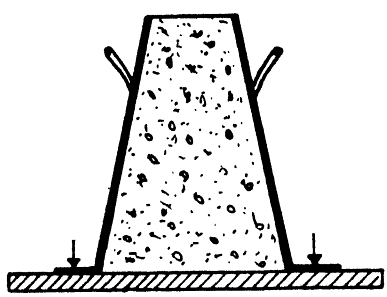


FIG. 20

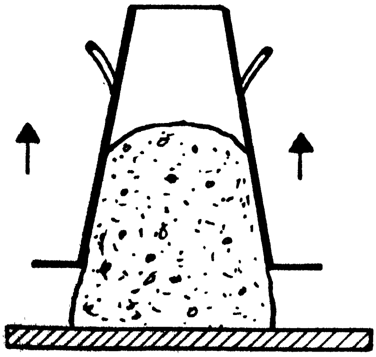


FIG. 21

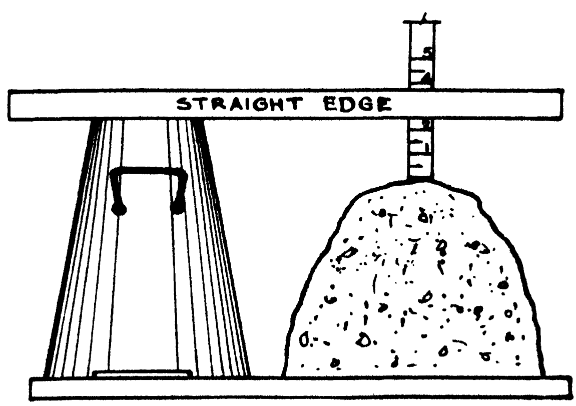


FIG. 22

times by the bullet end of the tamping rod of which the required dimensions are given in Fig, 19. The top is to be finished flat and level with the top of the mould, which must then immediately be lifted vertically off the concrete. The subsidence of the mound of concrete must then be measured—which measurement represents the " slump " test value.

Schedule II. Preliminary Concrete Cube Tests.—This examination comprises compression tests on concrete cubes made under accurate and controlled conditions in a laboratory.

" Materials and proportioning: the materials and the proportions used in making the preliminary tests shall be similar in all respects to those to be employed in the work. The water content shall be as nearly as practicable equal to that to be used in the work, but shall be not less than the sum of 30 per cent. by weight of the cement and 5 per cent. by weight of the aggregate unless specially authorised by the district surveyor. For porous aggregates, additional water shall be used to allow for the amount absorbed by the aggregates.

" Materials shall be brought to room temperature (58°–64° F.) before beginning the tests. The cement on arrival at the laboratory shall be mixed dry either by hand or in a suitable mixer in such a manner as to ensure as uniform a material as possible, care being taken to avoid intrusion of foreign matter. The cement shall then be stored in air-tight containers until required. Aggregates shall be in a dry condition when used in concrete tests.

" The quantities of cement, aggregate and water for each batch shall be determined by weight to an accuracy of 1 part in 1000.

" Mixing concrete : the concrete shall be mixed by hand or in a small batch mixer in such a manner as to avoid loss of water. The cement and fine aggregate shall first be mixed dry until the mixture is uniform in colour. The coarse aggregate shall then be added and mixed with the cement and sand. The water shall then be added and the whole mixed thoroughly for a period of not less than 2 minutes until the resulting concrete is uniform in appearance.

" Consistence : the consistence of each batch of concrete shall be measured, immediately after mixing, by the slump test made in accordance with the method of test for consistence of concrete given in Schedule I. Provided that care is taken to ensure that no water is lost, the material used for the slump test may be remixed with the remainder of the mix before making the test specimen.

" Size of test cubes : compression tests of concrete shall be made on 6-in. cubes. The mould shall be of metal with inner faces accurately machined in order that the opposite sides of the specimen shall be plane and parallel. Each mould shall be provided with a metal base, having a smooth machined surface. The interior surfaces of the mould and base shall be slightly oiled before concrete is placed in the mould.

" Compacting : concrete test cubes shall be moulded by placing the fresh concrete in the mould in three layers, each layer being rammed with a steel bar 15 in. long and having a ramming face of 1 in. square and a weight of 4 lb. For mixes of 1½ in. slump or less, 35 strokes of the bar shall be given for each layer ; for mixes of wetter consistence the number may be reduced to 25 strokes per layer.

" Curing : all test cubes shall be placed in moist air of at least 90 per cent. relative humidity and at a temperature of 58°–64° F. for 24 hours (± ½ hour), commencing immediately after moulding is completed. After 24 hours the test cubes shall be marked, removed from the moulds and placed in water at a temperature of 58°–64° F. until required for test.

" Method of testing : all compression tests on concrete cubes shall be made between smooth plane steel plates, without end packing, the rate of loading being kept approximately at 2000 lb. per sq. in. per minute. One compression plate of the machine shall be provided with a ball seating in the form of a portion of a sphere, the centre of which falls at the central point of the face of the plate.

"All test cubes shall be placed in the machine in such a manner that the load shall be applied to the sides of the cubes as cast.

"Distribution of specimens and standard of acceptance: for each age at which tests are required, three cubes shall be made and each of these shall be taken from a different batch of concrete.

"The acceptance limits are a difference of 15 per cent. of the average strength between the maximum and minimum recorded strengths of the three cubes. In cases where this is exceeded repeat tests shall be made, excepting where the minimum strength test result does not fall below the strength specified."

Schedule III. Works Concrete Cube Tests.—"The method described applies to compression tests of concrete sampled during the progress of the work.

"Size of test cubes and moulds: the test specimens shall be 6-in. cubes. The moulds shall be of metal, with inner faces accurately machined in order that opposite sides of the specimen shall be plane and parallel. Each mould shall be provided with a metal base plate, having a smooth machined surface. The interior surfaces of the mould and base shall be slightly oiled before concrete is placed in the mould.

"Sampling: wherever practicable concrete for the test cubes shall be taken immediately after it has been deposited in the work. Where this is impracticable samples shall be taken as the concrete is being delivered at the point of deposit, care being taken to obtain a representative sample. All the concrete for each sample shall be taken from one place. A sufficient number of samples, each large enough to make one test cube, shall be taken at different points so that the test cubes made from them will be representative of the concrete placed in that portion of the structure selected for tests. The location from which each sample is taken shall be noted clearly for future reference.

"In securing samples the concrete shall be taken from the mass by a shovel or similar implement and placed in a large clean pail or other receptacle, for transporting to the place of moulding.

"Care shall be taken to see that each test cube represents the total mixture of concrete from a given place. Different samples shall not be mixed together, but each sample shall make one cube. The receptacle containing the concrete shall be taken to the place where the cube is to be moulded as quickly as possible and the concrete shall be slightly remixed before placing in the mould.

"Consistence: the consistency of each sample of concrete shall be measured, immediately after remixing, by the slump test made in accordance with the method of test for consistence of concrete given in Schedule I.

"Providing that care is taken to ensure that no water is lost the material used for the slump tests may be remixed with the remainder of the mix before making the test cube.

"Compacting: concrete test cubes shall be moulded by placing the fresh concrete in the mould in three layers, each layer being rammed with a steel bar 15 in. long and having a ramming face of 1 in. square and a weight of 4 lb. For mixes of $1\frac{1}{2}$ in. slump or less, 35 strokes of the bar shall be given for each layer; for mixes of wetter consistence the number may be reduced to 25 strokes per layer.

"Curing: the test cubes shall be stored at the site of construction at a place free from vibration, under damp sacks for 24 hours ($\pm\frac{1}{2}$ hour), after which time they shall be removed from their moulds, marked, and buried in damp sand until the time for sending to the testing laboratory.

"They shall then be well packed in damp sand or other suitable damp material and sent to the testing laboratory, where they shall be similarly stored until the date of test. Test cubes shall be kept on the site for as long as practicable and for at least three-fourths of the period before test, except for tests at ages less than seven days.

"The temperature of the place of storage on the site shall not be allowed to fall below 40° F., nor shall the cubes themselves be artificially heated.

"Record of temperatures: a record of the maximum and minimum day and night temperature at the place of storage of the cubes shall be kept during the period the cubes remain on the site.

"Method of testing: all compression tests on concrete cubes shall be made between smooth plane steel plates without end packing, the rate of loading being kept approximately at 2000 lb. per sq. in. per minute. One compression plate of the machine shall be provided with a ball seating in the form of a portion of a sphere the centre of which falls at the central point of the face of the plate.

"All test cubes shall be placed in the machine in such a manner that the load shall be applied to the sides of the cubes as cast."

By-law 15. Structural Steel.—Structural steel is to be in accordance with the British Standard Specification No. 15—1936, except that, when certain conditions prescribed for a particular case by the Council are observed, structural steel complying with the British Standard Specification No. 548—1935 (high-tensile structural steel) or other structural steel or metal may be used in that particular case.

Applications for the use of any structural steel but that of British Standard Specification No. 15 quality are to be made in accordance with the Structural Steel Waiver Regulations.

Structural Steel Waiver Regulations.—The Council have agreed to consider a preliminary application to use a special structural steel, and if they are satisfied, in principle, with the material, an application setting out the details must be made. Alternatively, this detailed application may be made in the first instance; but in either case the detailed application is to be accompanied by full particulars of the character and quality of the proposed steel and by sufficient calculations and plans showing how it is to be used.

The steel should comply with a British Standard Specification for structural steel; but if no such specification exists for the particular steel, the Council have still the right to permit its use under the conditions which they might think fit. Sufficient information must be given to the Council on the matter of the composition, production, yield-point stress, ductility, ultimate tensile strength and other essential properties of the material. This information must refer to the actual steel as it will be produced in readiness for the work. Satisfactory arrangements must also be made, and indicated to the Council, by which the district surveyor can verify that the steel complies with the conditions of the Council's approval, by which the work will be supervised and by which it can be ensured that defective work will not be incorporated.

From all this information the Council will determine whether to permit the use of the material and whether any special conditions are to be imposed. Any approval which may be given will relate only to the particular building for which the application has been made. Thus a separate application must be made for every case in which waiver of the by-laws is desired.

Amongst other conditions, the Council will determine in each case the maximum permissible stresses and the limitations of deflections. Normally, consents will probably be based on the conditions that the permissible tensile or compressive stresses do not exceed one-half of the tensile yield-point stress, that the permissible shear stress on the gross sections of webs does not exceed 62·5 per cent. of the permissible tensile stress, that the permissible shear stress in the steel, except in webs, does not exceed 75 per cent. of the permissible tensile stress, and that the permissible bearing stress does not exceed 150 per cent. of the permissible compressive stress.

These Regulations are intended to serve only as a guide to the applicant, and the Council will consider each case on its merits.

Reinforcing Steel.—Steel for reinforced concrete is to be in accordance with the British Standard Specification No. 15—1936 except that, when certain conditions prescribed for a particular case by the Council are observed, steel complying with the British Standard Specification No. 165—1929 may be used for slabs only, or other steel or metal may be used generally for that particular case.

Applications for the use of any but steel of the British Standard Specification No. 15 quality are to be made in accordance with the Reinforcement Waiver Regulations.

Reinforcement Waiver Regulations.—The Council have agreed to consider a preliminary application to use a special reinforcing steel, and if they are satisfied in principle with the material an application setting out the details must be made. Alternatively, this detailed application may be made in the first instance; but in either case the detailed application is to be accompanied by full particulars of the character and quality of the proposed steel and by sufficient calculations and plans showing how it is to be used.

The steel should comply with a British Standard Specification; but if no such specification exists for the particular steel, the Council have still the right to permit its use under conditions which they might think fit.

Sufficient information must be given to the Council on the matters of yield-point stress, ductility, ultimate tensile strength, and other essential properties of the material. This information must refer to the actual reinforcing rod as it will be produced in readiness for the work. Satisfactory arrangements must also be made, and indicated to the Council, by which the district surveyor can verify that the reinforcement complies with the conditions of the Council's approval, by which the work will be supervised and by which it can be ensured that defective work will not be incorporated.

From all this information the Council will determine whether to permit the use of the material and whether any special conditions are to be imposed. Any approval which may be given will relate only for the particular building for which the application has been made. Thus, a separate application must be made for every case in which waiver of the By-laws is desired.

Amongst other conditions, the Council will determine in each case the maximum permissible stresses, the limitations for spacing of rods, the minimum concrete cover and the details for anchorage or restraint. Normally, consents will probably be based on the conditions that the permissible tensile stress does not exceed one-half of the yield-point stress and that the longitudinal compressive stress in a rod does not exceed the calculated compression in the surrounding concrete multiplied by the ascertained modular ratio, when the compressive resistance of the concrete is taken into account, nor 40 per cent. of the yield-point stress when the compressive resistance of the concrete is not taken into account. Both permissible tensile and permissible compressive stresses will be determined also by evidence that undesirable cracking of the concrete will not occur. These Regulations are intended to serve only as a guide to the applicant and the Council will consider each case on its merits.

By-law 16. Lime.—" Lime used for mortar or plaster shall contain not more than 7 per cent. by weight of carbon dioxide nor more than 3 per cent. by weight of matter insoluble in hydrochloric acid.

" All lime shall be properly hydrated before use."

By-law 17. Mortar.—" Cement mortar shall be composed of cement mixed with sand or other material approved as of like suitability by the district surveyor, in the proportions of one part of cement to not less than two, nor more than four, parts of the sand or other such material measured by volume.

"Cement-lime mortar shall be composed of Portland cement and hydrated lime mixed with sand or other material approved as of like suitability by the district surveyor. The proportions of cement and lime shall be as one volume of cement to not less than one nor more than five volumes of hydrated lime. The proportions of the cement-lime mixture to the sand (or other approved material) shall be as one volume of cement-lime mixture to not less than two, nor more than four volumes of sand (or other approved material).

"Lime-mortar shall be composed of putty from commercial hydrated lime or properly slaked sieved and matured lime mixed with sand or with other material approved as of like suitability by the district surveyor. The proportions of lime to sand (or other approved material) shall be as one volume of such slaked lime or putty to not less than two and not more than four volumes of sand or other material approved as of like suitability by the district surveyor."

By-law 18. Stone.—"Stone shall possess a resistance to crushing of at least 1500 lb. per sq. in. and shall be free from cracks, sand holes, and other defects which in the opinion of the district surveyor affect adversely its strength or permanence. Stone of a laminated formation which will be subjected to pressure shall be so bedded that the planes of cleavage will be at right angles to the direction of the pressure."

By-law 19. Bricks and Blocks.—"Bricks and blocks in walls or piers shall be composed of hard, well-burned clay, well-burned terra-cotta, stone, concrete, calcium silicate (in compliance with the British Standard Specification for Sand-lime (Calcium-Silicate) Bricks, numbered 187—1934) or other incombustible material of like hardness and durability or a combination thereof, and shall be such size, shape and surface as to permit of proper bonding and jointing.

"When such bricks or blocks are formed with cavities, hollows or perforations (except in the case of air bricks and ventilating blocks and of bricks or blocks in partition walls constructed in compliance with the requirements of proviso (*a*) to By-law 53)"* then :

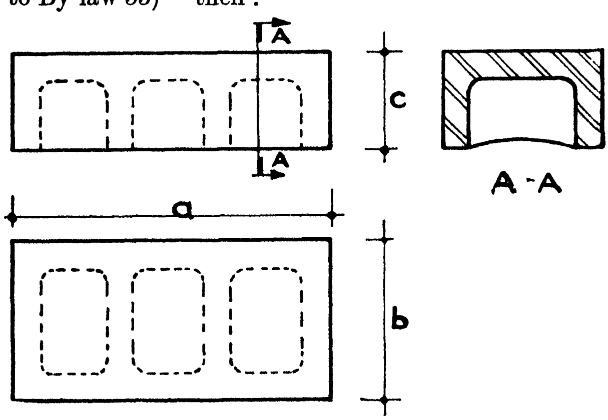


FIG. 23

Total solid material must not be less than $\frac{a \times b \times c}{2}$ so disposed that :

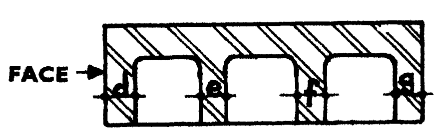


FIG. 24

$d + e + f + g$, etc., must not be less than $\frac{a}{3}$,

* Reproduced as clause 5 on p. 70.

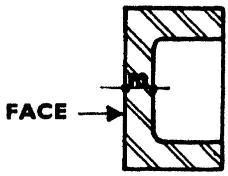


FIG. 25

m, etc., must not be less than $\frac{c}{3}$,

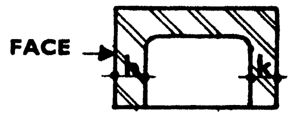


FIG. 26

$h + k$, etc., must not be less than $\frac{b}{3}$;

and so disposed that in a wall:

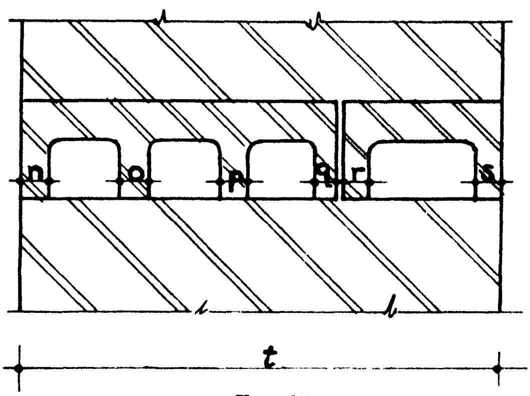


FIG. 27

$n + o + p + q + r + s$, etc. (excluding mortar or material forming one face of the wall), must not be less than $\frac{t}{4}$.

Bricks and blocks for load-bearing structures have been classified into seven groups, namely, "special," "first," "second," and so on up to "sixth." The designation of any brick or block may be found from Table 10, which gives the minimum stress which the brick or block must resist without crushing or cracking when it is saturated with water and loaded over the gross horizontal area in a manner similar to that in which it will be loaded in practice.

TABLE 10

DESIGNATION OF BRICKS OR BLOCKS

Designation	Load in lb. per sq. in. on Gross Horizontal Area
Special	Over 10,000
First	10,000
Second	7500
Third	5000
Fourth	4000
Fifth	3000
Sixth	1500

According to this ruling it will, in practice, be almost impossible, except by very remote chance, to have any bricks or blocks of the first grade. They will be either specials or seconds, or of lower grade, depending on whether their strengths are just over 10,000 lb. per sq. in. or under. It is difficult to state exactly into which class certain bricks will fall; but the brick strength values given in Table I of the Special Report No. 22 (Building Research) of the Department of Scientific and Industrial Research may be taken as a rough guide, although it should be remembered that the Building Research results are for bricks tested in a dry condition.

Bricks or blocks in non-load-bearing parts of a building (except in partition walls built in accordance with clause (*a*) of By-law 53) must, when saturated with water, have a crushing strength, without cracking, of at least 1000 lb. per sq. in. on the horizontal area, which horizontal area, in the case of hollow or perforated bricks or blocks, is to be taken as the least net or actual area of solid material measured on a plane at right angles to the face of the wall.

Bricks or blocks in a non-load-bearing partition wall built according to clause (*a*)* by By-law 53 must, when tested in the same manner as above, have a strength of at least 200 lb. per sq. in., this strength being calculated on the gross horizontal section.

By-law 20. Slates.—" Slates shall be sound and dense and shall be of such quality as to be capable of withstanding the acid test specified in the British Standard Specification numbered 680—1936."

By-law 21. Tiles (Clay and Concrete).—" Clay or marl plain roofing tiles shall comply with the British Standard Specification for Clay or Marl Plain Roofing Tiles, numbered 402—1930, and all other clay roofing tiles shall comply with the same British Standard Specification in regard to their quality and their resistance to frost and permeation by water.

" Concrete plain roofing tiles shall comply with British Standard Specification for Concrete Plain Roofing Tiles numbered 473—1932, and all concrete interlocking roofing tiles shall comply with the British Standard Specification for Concrete Interlocking Roofing Tiles numbered 550—1934."

By-law 22. Asbestos-Cement Sheeting. — " Asbestos-cement sheeting (including asbestos-cement sheets employed in the manner of roof slating) shall comply with the British Standard Specification (including Appendices A, B and C) for Asbestos-cement Slates and Unreinforced Flat Sheets and Corrugated Sheets No. 690—1936."

By-law 23. Steel Sheeting.—" Steel sheets used for the cover of any roof or any part of a roof shall be completely galvanised, or otherwise adequately protected from corrosion, and shall not be of a less thickness than that known as ' No. 24 Birmingham Wire Gauge.' "

By-law 24. Plastering.—" Lathing for plastering shall be of sound, well-seasoned wood free from sap, or of suitable metal lathing or of other material of like suitability.

" The filler for plastering shall consist of sand or other material of like suitability.

" The binding material for plastering shall consist of putty from commercial hydrated lime or of properly slaked, sieved and matured lime, or of cement, calcium sulphate plaster or of other material of like suitability, or of either cement or calcium sulphate plaster in suitable combination with commercial hydrated lime or properly slaked, sieved and matured lime.

" The materials for rendering and floating coats of plastering shall consist of filler and binding material and the proportion of filler to each volume of lime or cement shall be not less than two and not more than four volumes, and to each volume of calcium sulphate plaster not less than one and not more than three volumes. If the district surveyor so requires, there shall be mixed with

* Reproduced as clause 5 on p. 70.

every 3 cu. ft. of plastering 1 lb. of good, clean, well-beaten hair or other fibrous material of like suitability.

"The materials for setting coats of plastering shall consist of sound lime, putty and filler, or cement and filler, or a combination thereof, or of calcium sulphate plaster with or without the addition of sound lime, putty or filler, or of any combination of materials approved by the district surveyor.

"A subsequent coat of plastering shall not be applied until the previous coat has thoroughly dried.

"If fibrous slab or other slab or sheet plastering is employed, it shall be of sufficient thickness and shall be securely fixed."

By-law 25. Tests.—By notice the district surveyor may require tests to be made or other evidence to be submitted to him proving that any materials used or to be used conform to the By-laws. No materials but those which do conform to the By-laws may be employed.

CHAPTER 5

PART III. FOUNDATIONS BY-LAWS 26–38

Part III. By-laws 26–38. Sites, Foundations, and Excavations.—This part of the by-laws lays down the requirements for the treatment of the site, filling of voids or cavities, and for sizes and pressures on foundations to walls and piers. No detailed requirements are given for piled foundations, which must be constructed to the district surveyor's satisfaction. Fuller information on foundations, retaining walls and piling is given in Chapters 36 and 37, Volume 3.

Exactly what is a "foundation" is not defined in the By-laws; but in the Act a definition in relation to a wall is given and is reproduced on page 213. Because different stresses are permitted for materials in foundations from the stresses in the same materials used elsewhere in the structure, it is important that a distinction should be made and, therefore, it is suggested that all structural materials beneath the lowest floor and all external walls, etc., below, the ground level acting as retaining walls or supporting piers, columns or walls above, should be regarded as foundations. Those structural members of reinforced concrete which come within this class should be designed in accordance with the by-laws which relate specifically to reinforced concrete construction.

By-law 26. Foundations. Piling.—Foundations must be capable of carrying safely all the loads brought to them without the appropriate permissible stresses being exceeded. Stresses are given in By-laws 30, 34, 35 and 36. Piling must be to the satisfaction of the district surveyor.

This by-law entitles the district surveyor to use his discretion whether he will accept the empirical foundation sizes as for instance under By-law 33, or whether the foundation must be to exact calculation.

By-law 27. Site Treatment and Filling.—" Before the erection of any part of a building is commenced :—

(i) The site of such part shall be cleared of all material impregnated with fæcal or offensive animal or offensive vegetable matter (unless it has become or been rendered innocuous) and such other material as in the opinion of the district surveyor would, if not removed, affect adversely the building or some part thereof ; and

(ii) All excavations, voids, or cavities in the said site shall be filled or otherwise treated as the district surveyor may require for the purpose of ensuring the stability of the said building and of every part thereof."

Filling shall consist of material which has the district surveyor's approval as being suitable ; plain concrete may be used, provided that it is not inferior to concrete of designation VII in By-law 14. For permitted pressures on such concrete filling, see By-law 34.

It should not be inferred from this explanation that only concrete may be used for the filling of voids, etc. In many cases the spoil from the foundation excavations may be used with the approval of the district surveyor. Where, however, concrete is to be used, it must not be of weaker composition than 1 : 12.

By-law 28. Site Covering.—" The site of every part of a building shall be covered with concrete, as regards composition and quality not inferior to that designated V in By-law 14, at least 6 inches in thickness and smoothed on the upper surface.

" Provided that where such site is covered with reinforced concrete which complies with the requirements of these by-laws relating thereto the thickness of such reinforced concrete shall be at least 4 inches."

According to By-law 37 this concrete covering may be used as a floor of the building, provided that adequate protection shall be made in its construction against the action of moisture from the adjoining earth. This protection can be given in several ways, the choice of any particular method depending on the actual conditions of the site. For example, an asphalt layer below the slab, or the introduction of certain proprietary waterproofing compounds into the concrete may be used, and in many cases field draining of the site will give considerable additional protection.

A question which is not clearly settled by the by-laws is that of a floor covering the whole of the building site but not in contact with the ground (suspended slab), It is considered that, if such a floor be of any construction other than of solid concrete, a site covering under the floor in accordance with By-law 28 would be required. On the other hand, if the floor were of reinforced concrete at least 4 inches thick, the concrete not being inferior in quality to that of concrete III in By-law 14, it is considered that the floor would constitute adequate site covering, and that therefore a separate ground slab would not be necessary. Whether such a suspended solid slab should be waterproofed or not would depend on the dampness of the particular site and on the cross ventilation below the floor.

By-law 29. Exterior Filling.—" All excavations, voids, or cavities in the earth outside a building and less than 3 feet measured horizontally in any direction from the external face of any wall or pier of such building shall be filled or otherwise treated to the satisfaction of the district surveyor."

Filling shall consist of material which has the district surveyor's approval as being suitable ; plain concrete may be used, provided that it is not inferior to concrete of designation VII in By-law 14. For permitted pressures on such concrete filling, see By-law 34. See also By-law 27.

By-law 30. Foundation Bearing Pressures.—The dimensions of foundations for walls and piers will normally be determined by the requirements of By-law 33. However, if the district surveyor be satisfied that foundations of other dimensions would not reduce the stability of the building, or if for any reason he should require the foundations to be determined for any part of the building from the exact loads, he will specify a bearing pressure for the particular ground, and this pressure must not be exceeded.

As an indication of reasonable bearing pressures, a footnote attached to By-law 30 has been reproduced here as Table 11.

TABLE 11

NORMAL BEARING PRESSURES ON SUBSOIL

Nature of Subsoil	Load on Ground, tons per square ft.
Alluvial soil, made ground, very wet sand .	½
Soft clay, wet or loose sand	1
Ordinary fairly dry clay, fairly dry fine sand, sandy clay	2
Firm dry clay	3
Compact sand or gravel, London blue or similar hard, compact clay	4

By-law 31. External Earth Loads.—" Where the earth adjacent to any building will or may exert pressure upon, or otherwise cause the application of loading to, any part of such building, adequate provision shall be made in the design and construction of every part of the building to ensure that such pressure or loading will be supported and transmitted properly and safely and without exceeding the appropriate limitations of permissible stresses specified in these by-laws."

By-law 32. Protection of Metal in Foundations.—" Where metal is used in combination with the concrete required for compliance with By-law 33, or in combination with the corresponding lowermost portion of any other part of a building, proper protection shall be provided to prevent damage to such metal." See By-law 33.

By-law 33. Concrete Foundations.—Unless carried on a beam, the base or footing (if footings are used) of a wall or pier must rest on a concrete foundation; and unless an exact calculation is made and approved by the district surveyor, the projections beyond the sides and ends of the wall or pier shall be such that in Figs. 28, 29, and 30:—

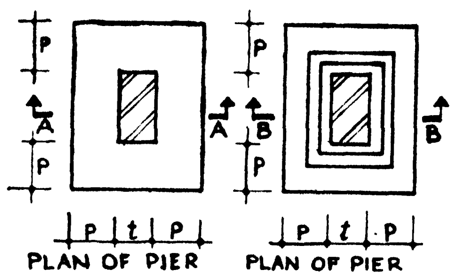


FIG. 28

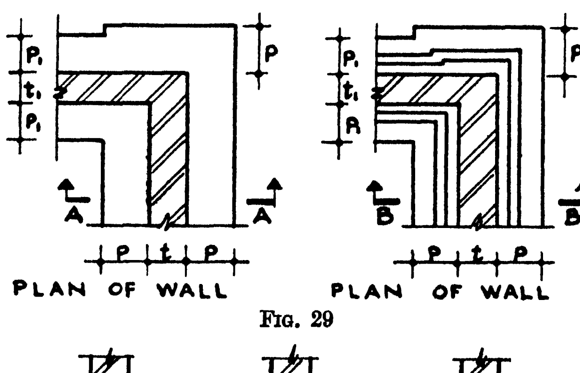


FIG. 29

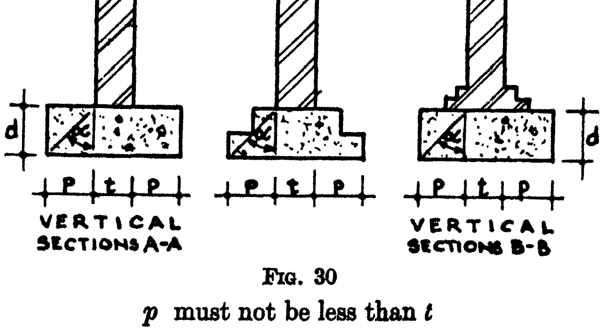


FIG. 30

p must not be less than t

p_1 " " " t_1

and p_2 " " " t_2, etc.

α " " greater than 45°.

When an external wall is to be built against another existing external or party wall as shown in Fig. 31, the district surveyor may agree to the omission of the required foundation projection "p_2x" on the side of the existing wall. Projections from existing foundations may, in such a case, be cut away as is customary.

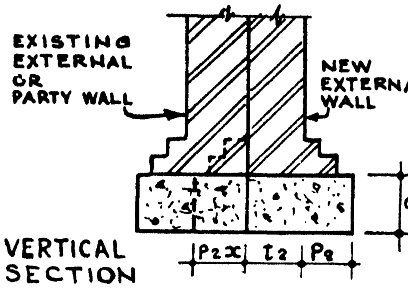


FIG. 31

Plain concrete of quality not inferior to that of concrete V (By-law 14) may be used for the wall or pier foundations, and in this case the angle of dispersion of the load α must not be greater than 45° (By-law 7). Therefore, in Figs. 28, 29, 30, and 31

$$\left.\begin{array}{l} d \text{ must not be less than } p \\ \text{and } d_2 \quad ,, \quad ,, \quad ,, \quad p_2, \text{ etc.} \end{array}\right\} \text{for plain concrete.}$$

The provision of footings is not required by the by-laws.

If reinforced concrete is used, the above requirements for the angle of dispersion and the depth do not apply. Then, however, concrete must conform to the requirements for "reinforced concrete," i.e. be not inferior in quality to concrete III (By-law 14), etc., and the reinforcement must have any additional protection as required by By-law 32. For steel reinforcement the authors suggest that a minimum of 3 inches of concrete should be sufficient for this purpose. As pointed out under By-law 30, the requirements for foundation sizes given here may be decreased or increased with the agreement of, or on the instructions of, the district surveyor, who must be satisfied that the foundations will afford sufficient strength and stability to the building. Such variations in foundation sizes must be to calculation, earth pressures being determined under By-law 30, and plain concrete stresses, if they are the governing factors, by By-law 35. By-laws 2, 26 and 31 entitle the district surveyor to ask for an exact calculation whenever he thinks it necessary.

By-law 34. Pressure on Plain Concrete Filling.—The loading on plain concrete filling must not exceed the intensity given for the appropriate concrete in Table 12. According to By-law 14, concretes VI and VII must not be used in any part of the construction of a building; but they may be used as filling materials under By-laws 27 and 29.

TABLE 12

PERMISSIBLE PRESSURES ON PLAIN CONCRETE FILLING

By-law Designation	Mix	Cubic ft. of Aggregate to 112 lb. of Cement	Maximum Permissible Pressure	
			Tons per foot	Lb. per sq. in.
IV	1 : 6	7½	20	311
	1 : 7	8¾	17·5	272
V	1 : 8	10	15	233
	1 : 9	11¼	12·5	195
VI	1 : 10	12½	10	156
	1 : 11	13¾	7·5	117
VII	1 : 12	15	5	78

By-law 35. Pressure on Plain Concrete Foundations.—The pressure coming on the top of a concrete foundation of a wall or pier must be calculated. If the foundation is of plain concrete it must be restrained to the satisfaction of the district surveyor at the top and bottom to prevent any horizontal movement, that is to say, in Fig. 32 at points A and C.

If the ratio $h : b$ be not greater than 2, or if the foundation be further restrained horizontally to the district surveyor's satisfaction between the top and bottom (say at point B in Fig. 32), the pressure on the concrete can be taken as the value given in column A of Table 13 for the appropriate mix. (Under By-law 33 no concrete inferior in quality to that of concrete V may be used for such a foundation.)

In cases in which there is no intermediate restraint the permissible pressures on the concrete are set out in Table 13 for values of $h : b$ up to 12.

By interpolation between the values given the exact permissible pressure can be obtained for any exact value of $h : b$.

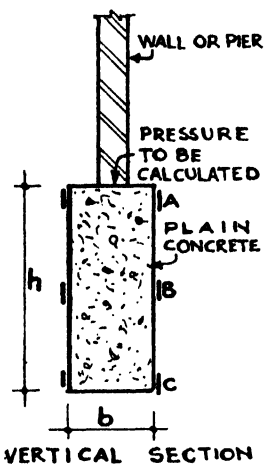


FIG. 32

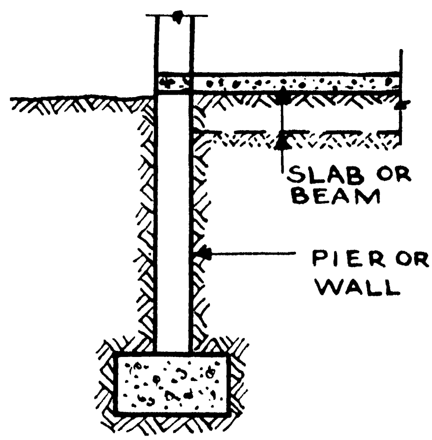


FIG. 33

For ratios between 11 and 12 the pressure may be taken by proportion from the table; but it will be observed that when the value 12 is reached the pressure is zero. Therefore, no such plain concrete foundation in which $h : b$ is 12 or over may be used, unless, however, adequate intermediate restraint is provided.

The restraint at the bottom of a foundation is usually afforded by the ground itself. The top restraint is generally given by the ground-floor slab; but if no such slab exists stiffening beams should be introduced. Where the base is very deep below the ground level, piers or walls might usefully be employed (see Fig. 33), which may be considered stiffened at their lower ends. For these piers or walls reductions in working stresses dependent on their slenderness ratios must be made in accordance with Table 13.

By-law 36. No Tension to be allowed in Plain Concrete.—" Plain concrete shall not be relied upon to resist tensile stresses otherwise than in accordance with the requirements of By-law 33."

By restricting the angle of dispersion of the load to 45° By-law 33 does not in fact allow any calculated tension to be taken by the concrete except the tensile component due to the shear. As this tensile component, however, is not normally considered, and as the intention of By-law 36 would appear to be to prohibit the taking of calculated tensile stresses by plain concrete, it is considered that the reference to By-law 33 should, except for academic purposes, be neglected.

TABLE 13

MAXIMUM PERMISSIBLE PRESSURE ON PLAIN CONCRETE FOUNDATIONS RESTRAINED HORIZONTALLY AT TOP AND BOTTOM ONLY

By-law Designation	Popular Reference	Cubic Feet of Aggregate to 112 lb. of Cement		Values of h/b. See Figure 32: A, Up to 2		Up to 3		Up to 4		Up to 5		Up to 6		Up to 7		Up to 8		Up to 9		Up to 10		Up to 11		12	
		Fine	Coarse	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. in.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.	Tons per sq. ft.	Lb. per sq. in.
I	1 : 1 : 2	1¼	2½	40	622	36·0	560	32·0	498	28·0	436	24·0	373	20·0	311	16·0	249	12·0	187	8·0	124	4·0	62	0	0
	1 : 1·1 : 2·2	1⅜	2¾	39	606	35·1	546	31·2	486	27·3	425	23·4	364	19·5	303	15·6	243	11·7	182	7·8	121	3·9	61		
	1 : 1·2 : 2·4	1½	3	38	591	34·2	532	30·4	473	26·6	414	22·8	355	19·0	295	15·2	236	11·4	178	7·6	118	3·8	59		
	1 : 1·3 : 2·6	1⅝	3¼	37	575	33·3	518	29·6	461	25·9	403	22·2	345	18·5	288	14·8	230	11·1	173	7·4	115	3·7	58		
	1 : 1·4 : 2·8	1¾	3½	36	560	32·4	504	28·8	448	25·2	392	21·6	336	18·0	280	14·4	224	10·8	168	7·2	112	3·6	56		
II	1 : 1·5 : 3·0	1⅞	3¾	35	545	31·5	490	28·0	436	24·5	382	21·0	327	17·5	272	14·0	218	10·5	164	7·0	109	3·5	55	0	0
	1 : 1·6 : 3·2	2	4	34	529	30·6	476	27·2	423	23·8	371	20·4	317	17·0	264	13·6	212	10·2	159	6·8	106	3·4	53		
	1 : 1·7 : 3·4	2⅛	4¼	33	514	29·7	462	26·4	411	23·1	360	19·8	308	16·5	256	13·2	206	9·9	154	6·6	103	3·3	51		
	1 : 1·8 : 3·6	2¼	4½	32	498	28·8	448	25·6	398	22·4	349	19·2	299	16·0	249	12·8	199	9·6	149	6·4	99	3·2	50		
	1 : 1·9 : 3·8	2⅜	4¾	31	482	27·9	434	24·8	386	21·7	338	18·6	289	15·5	241	12·4	193	9·3	145	6·2	96	3·1	48		
III	1 : 2·0 : 4·0	2½	5	30	466	27·0	420	24·0	373	21·0	327	18·0	280	15·0	233	12·0	187	9·0	140	6·0	93	3·0	47	0	0
IV	1 : 6	7½		20	311	18·0	280	16	249	14·0	218	12·0	187	10·0	156	8·0	124	6·0	93	4·0	62	2·0	31	0	0
	1 : 7	8¾		17·5	272	15·75	245	14	218	12·25	195	10·5	164	8·75	136	7·0	109	5·25	82	3·5	55	1·75	27		
V	1 : 8	10		15	233	13·5	210	12	187	10·5	164	9·0	140	7·5	117	6·0	93	4·5	70	3·0	47	1·5	23	0	0

By-law 37. Site Covering as Floor.—See By-law 28.

By-law 38.—" Throughout the period of preparation for the construction or conversion of a building or part of a building, the builder shall on the site take all precautions necessary for the purpose of securing the stability of the building and of every part thereof."

This requirement is particularly important. For instance, new retaining walls or existing adjoining walls may be stable only when a certain part or the whole of the new structure is completed. In these cases, temporary strutting or shoring is necessary, and it is the purpose of this By-law to ensure that such precautions are taken.

CHAPTER 6

PART IV. WALLS AND PIERS. BY-LAWS 39–62

Part IV. Walls and Piers.—Part IV, treating of walls and piers and containing By-laws 39 to 62, is divided into three sections. Section 1 contains the general requirements, such as the proportion of openings, materials of construction, and certain standard requirements for the minimum thicknesses of walls and piers. Section 2 gives the requirements for wall thicknesses when those thicknesses are not determined by actual calculation. Under Section 3, where permissible stresses and other governing conditions for the calculations are set out, wall thicknesses may be ascertained by calculation.

The requirements of Section 1 must be observed in all cases ; but either Section 2 or Section 3 can be adopted for detailed requirements. It appears to be the intention of the by-laws that the district surveyor should have the right to ask for the application of Section 3 where he thinks it necessary. (See By-laws 2 and 50.)

If the thickness of a wall, as given in Section 2 or as calculated under Section 3, should be different from that required by any of the general requirements of Section 1, then the wall must actually be constructed to the greater thickness.

PART IV. SECTION 1. GENERAL REQUIREMENTS FOR WALLS AND PIERS

By-law 39. Walls and Proportion of Openings.—Every building must be enclosed by walls. For the interpretation of the rest of this by-law, specifying the amounts and lengths of openings which may be made in a wall, piers are to be regarded as " wall " and any glass or glazing as " openings."

There are no restrictions on the openings below the soffit of the first-floor slab in any building.

Openings, represented in Figs. 34, 35, and 36 by the shaded areas, may be made in walls such that :

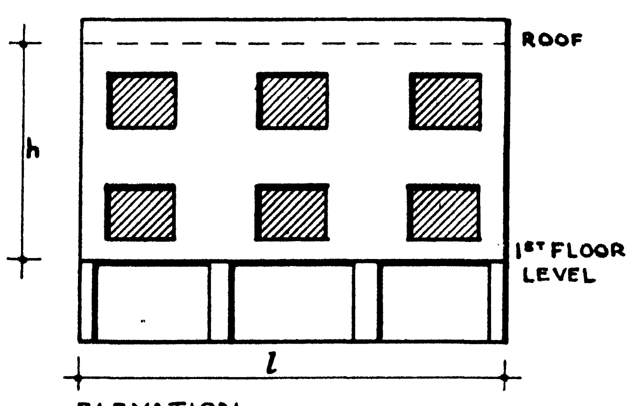


FIG. 34

the total shaded area must not be greater than $\frac{h \times l}{2}$, h being measured from the soffit of the first-floor slab, *and that on any storey height:*

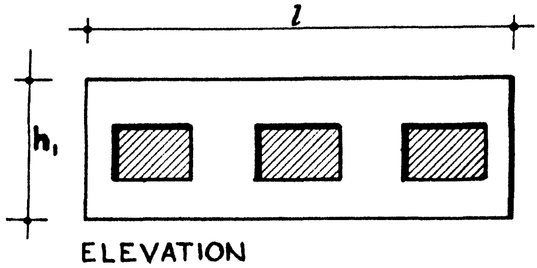


FIG. 35

the shaded areas must not be greater than $\frac{2}{3}(h_1 \times l)$,
and that also on any storey height:

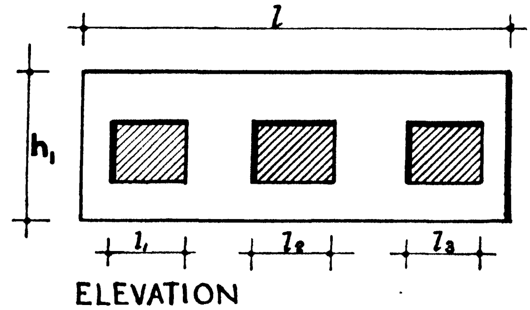


FIG. 36

$l_1 + l_2 + l_3$, etc., must not be greater than $\frac{3}{4}l$.

By-law 40. Materials for Walls.—" Every wall or pier of a building shall be constructed of bricks or blocks laid in horizontal courses properly bonded, bedded and jointed with mortar or of plain concrete or of reinforced concrete, or (except in the case of party walls) of such materials in combination with metal framework."

Although the by-law does not specifically mention it, there does not appear to be any reason why such materials should not be used also in combination with a reinforced concrete framework, as they may be with a metal framework.

" Where any walls of a building meet, or where such walls meet piers, they shall be properly bonded or otherwise securely and permanently bound together."

By-law 41. Restriction on Hollow Bricks or Blocks.—" No hollow bricks or hollow blocks shall be used in the construction of a wall or pier of a building (other than a non-load bearing partition wall) unless evidence has been produced to the satisfaction of the Council showing that such wall or pier will be equal as regards fire-resistance to that of a wall or pier constructed of solid bricks or solid blocks or of plain or reinforced concrete in accordance with the requirements of these by-laws."

Because the walls mentioned here are of different standards of fire resistance it can be inferred that the choice of this standard is left to the applicant.

By-law 42. Restriction on Timber in Walls.—" No timber or other combustible material (other than the ends of beams, joists, purlins and rafters, the horns of door frames and of window frames, fixing blocks and plugs and pole plates, bearing rafters and supporting no walling other than windpinning) shall be built into the required thickness of a wall or pier.

"When the end of a beam, joist, purlin or rafter or other timber is built into the required thickness of a party wall," then :

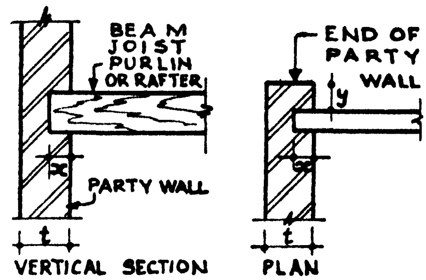


FIG. 37

x must not be greater than $\frac{t}{2}$

y ,, ,, less than 4 in.

when y = encasement, on all sides, of brickwork or other solid incombustible material.

By-law 43. Thickness of Walls.—Except for the two cases cited after this general requirement, no wall of any of the types given in Table 14 may be of less thickness in any part, exclusive of plastering, rendering, rough cast or other applied covering than the thickness given for the appropriate material of construction.

TABLE 14

	Minimum Thicknesses for Walls acting as :			
Construction	Buttressing	External	Panel in External	Party
Bricks or blocks . .	8½ in.	8½ in.	8½ in.	8½ in.*
Plain concrete . .	8½ in.	8½ in.	8½ in.	8½ in. *
Reinforced concrete .	Not specified	4 in.	4 in.	8 in.[1]

The thickness of buttressing walls of reinforced concrete is not specified but it seems reasonable that it should not be less than 4 in.

These thicknesses do not apply to

(i) a building
- (a) which is not a dwelling house, and at the same time
- (b) of which the walls are not more than 10 feet high, there being only one storey, and at the same time
- (c) which is not more than 30 ft. wide measured in the direction of the roof span.

Nor to

(ii) an erection
- (a) which is above the roof level of a building, and
- (b) which is for protecting a tank or motor or for a similar purpose, and

[1] A reinforced concrete party wall forming part of a warehouse building which is not of reinforced concrete construction must not be less than 13 in. thick, exclusive of plastering, etc.

* The 13 in. limit as specified in footnote 1 should logically apply also to these walls in warehouse buildings.

(c) which is not to be used for a work-room or habitation, and
(d) which is supported on the building to the district surveyor's satisfaction, and
(e) of which neither the length nor width exceeds 10 ft., and
(f) of which the height does not exceed 8 ft. from the roof of the building to the top of the walls.

In the first of these two cases, namely (i) above, the external walls may be of bricks or blocks bedded and jointed in cement mortar and bonded into piers at each end and intermediately where necessary so that the clear length of wall between any two piers shall be not more than 10 ft. The wall must not be less than 4 in. thick nor the piers less than the size required by calculation with a minimum size of $8\frac{1}{2}$ in. $\times$ $8\frac{1}{2}$ in. The walls may carry only the distributed load of the roof, which must be so designed that no side thrust is imposed.

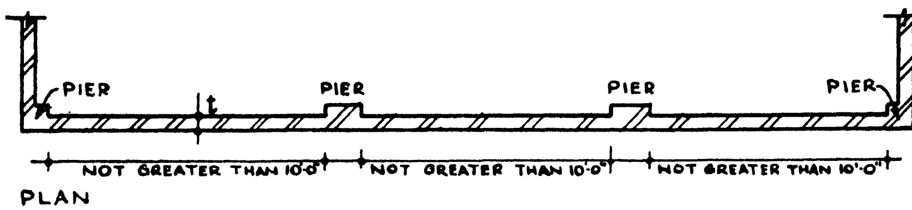


FIG. 38

t must not be less than 4 in.
any pier ,, ,, ,, $8\frac{1}{2}$ in. $\times$ $8\frac{1}{2}$ in.

For the second case above, namely (ii), the walls may conform to the same conditions as for the first case, except that, as the total length of any wall is restricted to 10 ft., intermediate piers are not required unless the calculations show them to be necessary for load bearing.

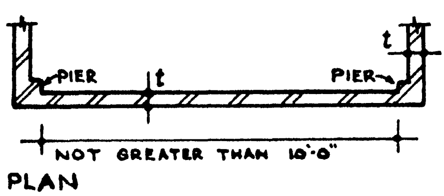


FIG. 39

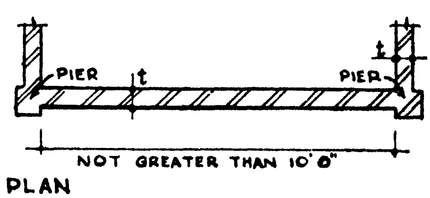


FIG. 40

t must not be less than 4 in.
any pier ,, ,, ,, $8\frac{1}{2}$ in. $\times$ $8\frac{1}{2}$ in.

Party Walls to any Building.—In any building every party wall over 35 ft. long and also over 40 ft. high must have a thickness of at least 13 in. of solid material in every part.

In any building every party wall less than 35 ft. long but over 50 ft. high must have a thickness of at least 13 in. of solid material in every part.

Party Walls in Basements of any Building.—In any building every party wall, higher than 25 ft. or longer than 30 ft., which divides a basement must have a thickness of at least 13 in. of solid material throughout the basement.

Party Walls to Buildings of the Warehouse Class.—Every party wall over 30 ft. high which encloses a building of the warehouse class must have a thickness of at least 13 in. of solid material in every part.

Party Walls in Basements of Buildings of the Warehouse Class.—Every party wall which divides a basement of a building of the warehouse class must have a thickness of at least 13 in. of solid material throughout the basement.

Length of Walls.—The length of a wall is to be taken as defined in By-law 52.

By-law 44. Thickness of Walls.

(1) *Any Wall.*—Every wall and pier in a building constructed of bricks, blocks or plain concrete shall have a thickness t at any level, which must not be less than one-sixtieth of the height of the wall or pier above that level

Thus in Fig. 41 :

$$t \text{ must not be less than } \frac{h}{60}$$

where t = thickness at level A and h = height to top of wall from level A.

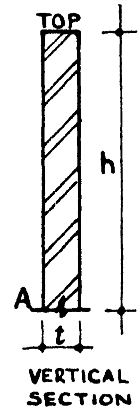


FIG. 41

Party Walls.—Every party wall and any pier combined with it constructed of bricks, blocks or plain concrete shall have a thickness t, of solid material at any level which must not be less than one-fortieth of the height of the wall above that level.

Thus in Fig. 42 :

$$t_1 \text{ must not be less than } \frac{h}{40}$$

where t_1 = thickness at level A and h = height to top of wall from level A.

If, under any other by-law, a different value for the thickness shall be given or calculated, that different value must be taken as the thickness to which the wall or pier shall actually be built, but only if it is greater than the value of t or t_1 as determined above.

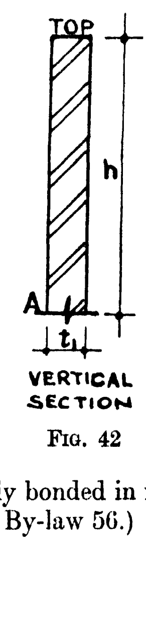


FIG. 42

(2) If a portion of a wall does not carry any of the loads coming on the whole wall, the rest of the wall being capable of carrying all these loads safely, and if that portion is adequately bonded in round all its sides, it may be regarded as a separate or panel wall. (See By-law 56.)

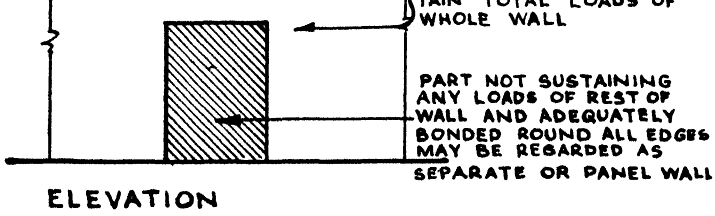
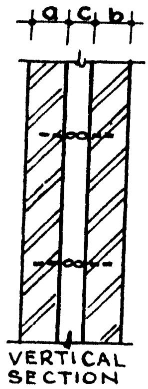


FIG. 43

By-law 45. Cavity Walls.—Cavity walls must consist of two leaves of properly bedded and jointed bricks or blocks.



a must not be less than 4 in.

b ,, ,, ,, ,, 4 in.

c ,, ,, ,, ,, 2 in.

c ,, ,, greater than 6 in.

FIG. 44

where a = thickness of one leaf; b = thickness of other leaf; c = width of the cavity.

The two leaves must be united by metal ties spaced according to the dimensions given in Table 15 for the appropriate cavity widths.

TABLE 15

SPACING OF TIES IN CAVITY WALLS

Width of Cavity in inches not exceeding	Minimum Number of Ties per sq. yd. of Wall	Maximum Horizontal Distance apart	Maximum Vertical Distance apart at Side of Openings
3 in.	2	3 ft. 0 in.	12 in.
4 in.	2·66	2 ft. 6 in.	9 in.
5 in.	3·33	2 ft. 0 in.	6 in.
6 in.	4	1 ft. 6 in.	6 in.

From the second and third columns of this table it will be observed that the normal vertical spacing will be 18 in. It is desirable that ties be staggered, i.e. arranged in diagonal rows, because this gives greater rigidity for the same number of ties.

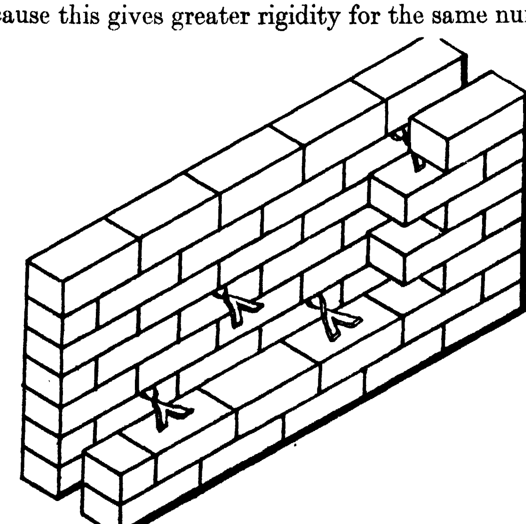


FIG. 45

Unless the district surveyor should agree otherwise, ties must consist of galvanised (or otherwise protected) iron, not less than $\frac{3}{4}$ in. × $\frac{3}{16}$ in. in cross-section and shaped to prevent the passage of moisture across the cavity.

Any timber projecting into or across the cavity must be protected from descending moisture by sheet lead or other permanent impervious flashing, so built into the wall that the passage of moisture across the cavity is prevented.

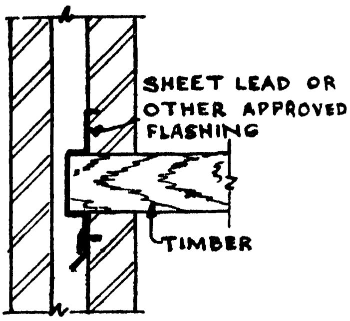


FIG. 46

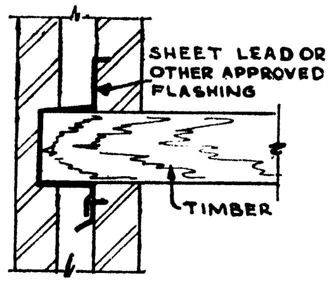


FIG. 47

The cavity must not contain any mortar droppings or debris.

By-law 46. Damp-proofing.—" Every building shall be so constructed as to ensure that it will not be affected adversely by moisture from adjoining earth."

When dealing with site concrete in By-law 28, certain suggestions were made for protection against the effects of moisture so far as the floor of the building was concerned. Consideration must also be given to the facts that moisture can penetrate horizontally through certain walls and that it can also rise vertically up a wall, by capillarity, from the ground.

The purpose of this by-law would, therefore, seem to be to guard against the horizontal penetration of moisture when a wall has earth on one side and habitable space, such as a basement, on the other, and against the rising of moisture up a wall to a level where it can have harmful effects, which level usually is that of the ground floor. In the first case the actual conditions of a particular instance must govern the choice of any protective measures. If there is just earth dampness present, and if the wall is of brick, an exterior or interior rendering containing a waterproofing compound would probably be sufficient, the designer having of course first ascertained the worst possible condition of dampness which is likely to obtain. If there is likely, at any time, to be a greater amount of water present, asphalt or other tanking would be necessary. A waterproofed concrete by itself would probably be sufficient if the wall were soundly constructed for all normal amounts of earth moisture. Where, however, such moisture would be under hydrostatic head, tanking, as before, would be the most satisfactory protection. In spite of the difficulties which are presented in making it there, the best position for asphalt or other tanking is on the outside of the wall or floor which is to be protected. Any hydrostatic pressure tends to press the tanking materials harder against the surface of the structure and deeper into the pores. This not only prevents bulging or bursting of the tanking, which frequently happens when it is on the inside, but at the same time as it prevents the passage of moisture, it protects the structural materials. Constructional difficulties can be overcome to a great extent, in the case of floors, by making the tanking before the structural floors are made, on a thin external concrete mat; and in the case of walls, by making the tanking on the inside of a $4\frac{1}{2}$-in. brick external wall. If the structural wall itself is of brick, even this thin outer wall is not necessary, although it is desirable if only to afford protection to the tanking from damage by earth which would come in contact with it.

Except in the case of concrete walls damp-proof courses are necessary to prevent the rising of moisture. Located below the ground-floor level, these may consist of slates laid flat in mortar, bituminous compounds, metal sheets such as copper or lead, or waterproofed mortar courses.

Sufficient damp-proofing can, in almost every case, be given to a concrete wall by the inclusion of a waterproofing compound in the concrete which is to be used up to the ground-floor level. This method does not involve making a definite joint plane across the wall.

By-law 47. Wall Copings.—This by-law specifies that every wall of which the top is exposed must have a suitable protecting coping, and also that the coping must be bedded and secured to the district surveyor's satisfaction. The by-law describes one type of coping which would be suitable, namely, that in which a top course of bricks on edge, laid in cement mortar, is carried on a creasing of two courses of slates or dense tiles, also laid in cement mortar and so arranged that they break joints. It is stated that any other suitable damp-proof course below the top course of bricks on edge could be used.

The simplest example of such a coping is shown in Fig. 48 for a 9-in. wall.

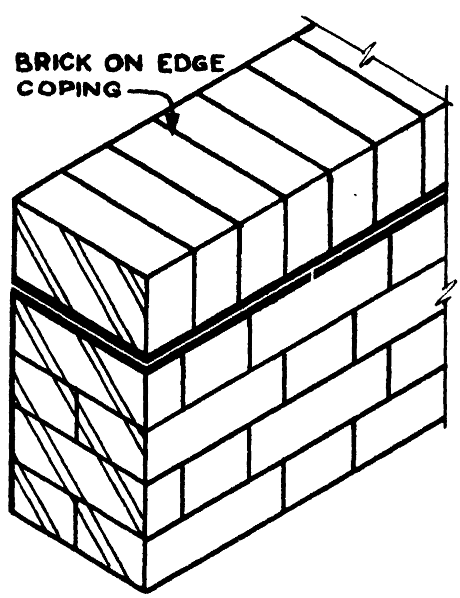


FIG. 48.

In addition to the By-law requirements for copings of this type, there are two points which should also be given attention. The first is that the top bricks should be of low porosity, and consequently of high resistance both to rain and, particularly, to frost. Engineering bricks satisfy this condition and also they usually afford a chance of providing certain architectural relief by change of colour. The second is that the lower course of slates or tiles should themselves break joint with the course of bricks below. There is often difficulty in satisfying this second condition, for the common slate dimensions do not fit regularly to brick dimensions. Nevertheless, both these requirements should be met whenever it is possible.

Coping bricks may be of any profile; for example, they may be standard square-cornered bricks or they may be moulded. The arrangement illustrated in Fig. 48, whilst it is in accordance with the By-law, has the disadvantage that it permits water falling on the top to run down the face of the wall. A modification of this type is shown in section in Fig. 49, where the tiles have been arranged to project an inch or two on either side, so as to oversail the face of the wall.

To a large extent water on the top is kept away from the wall faces. The coping in Fig. 50 produces similar effects.

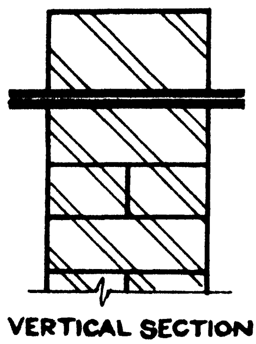


FIG. 49

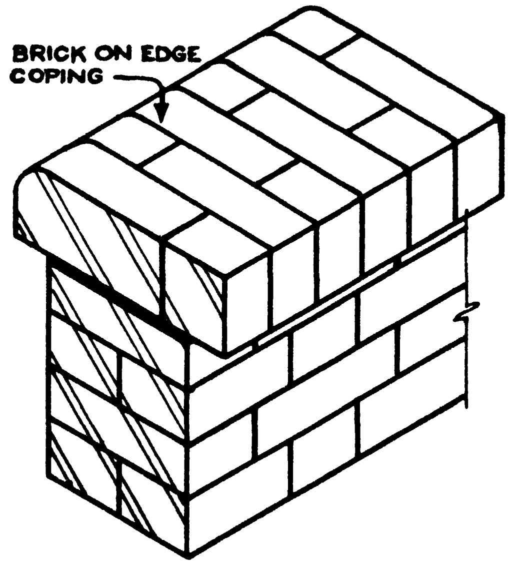


FIG. 50

Another type of coping which might be considered suitable is that shown in Fig. 51. It consists of a slab of stone, precast concrete, etc., or of burnt clay in the form of a large tile. These materials are very impervious to moisture, and the slabs can be produced with proper drip channels and to any reasonable profile. It should not be necessary to use such an elaborate damp-proof course as that described above as well as these impervious slabs; but any such course would afford additional protection. It is, however, good practice to providc a single layer of slates or tiles, or at least a slate or tile under every joint in the coping.

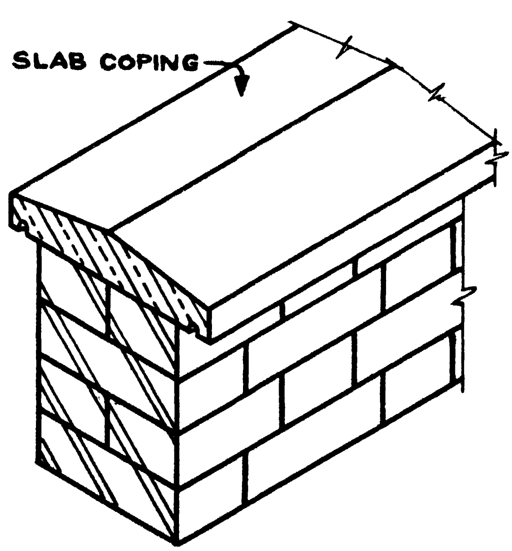


FIG. 51

Bituminous felts or lead sheet or similar materials which would afford at least the same resistance to moisture as the cement mortar slate or tile damp-proof course, could be regarded as suitable substitutes for that purpose.

In *Fig.* 52 one example is given of a coping to a cavity wall. The construction is the same as for previous examples; but the point to note is that the cavity must be bridged or closed below the damp-proof course. In the particular example, the

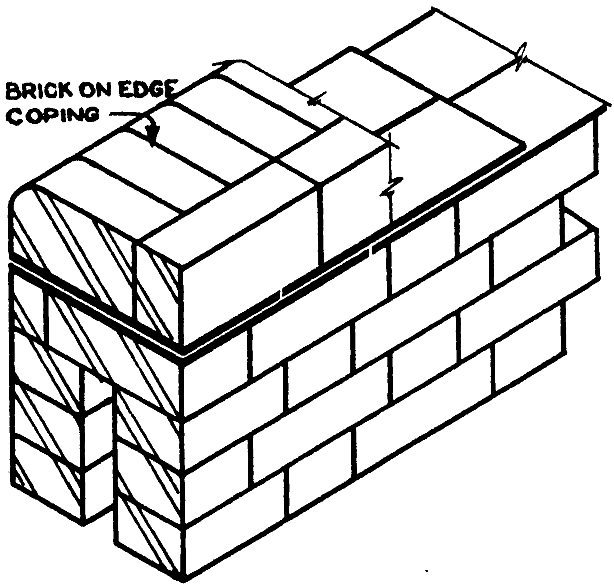


FIG. 52

cavity is narrow and it may be bridged in one brick course. Wider cavities require more bridging courses; for example, a 6-in. cavity may require one oversailing course on each side of the cavity, which reduces the gap, then the actual bridging course, and finally, on top, a third course to assist in distributing evenly any load or dead weight from above and to assist, by bond, in uniting the two leaves of the cavity.

By-law 48. Notice to District Surveyor regarding Existing Work.—" No earth, concrete, brickwork, stonework or any other material supporting or aiding in the support of any superstructure shall be disturbed within two clear days of the district surveyor having received notice in writing giving particulars of the nature of the work and the date of its commencement."

By-law 49. Notice to District Surveyor regarding Thickening of Walls.—" A wall shall not be thickened except after two days' clear notice being served on the district surveyor of the intention to thicken, and the thickening shall be executed to the satisfaction of the district surveyor, and such wall so thickened shall be of the required thickness."

By-law 50. Walls to be to Section II, Part IV, or to Calculations.—No wall or pier may be considered to conform to the By-laws (of which the general intention is summarised in By-law 2, namely, that every part of a building must be constructed to carry safely all the dead and superimposed loads without stressing any of the materials beyond the limits permitted by the By-laws) unless it is constructed either:

(*a*) in accordance with the special requirements, dimensions and other requirements of Section 2 of this Part (Part IV), by which the dimensions, etc., are determined more or less from empirical rules and tables. or

(*b*) in accordance with the requirements of Section 3 of this Part and with calculations which show that the permitted stresses given in Section 3 and in Parts V and VI of the By-laws are not exceeded.

Besides conforming to either of these alternative requirements a wall or pier must conform also to the general requirements set out in this section (Section I, Part IV).

Although according to the wording of this By-law, the requirements of By-law 2 are satisfied if the prescribed conditions required for Section 2 are satisfied it appears to be the intention of the By-laws to reserve the right to the district surveyor to insist upon calculations in accordance with Section 3. See particularly By-law 53.

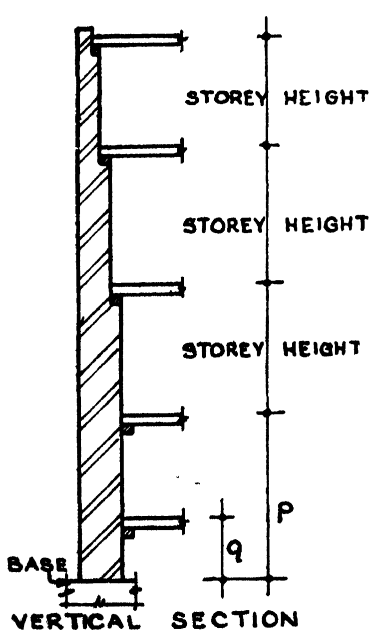
PART IV. SECTION 2

By-law 51. Conditions under which Section 2 (Wall Dimensions) may be Applied.—All of the following special conditions must be satisfied before the remaining by-laws of this section (Section 2) may be adopted for the determining of wall dimensions. If such conditions are not satisfied exact calculations are to be made and the by-laws under Section 3 are to be followed.

(i) *Materials.*—The wall must consist either of bricks or blocks laid horizontally and properly bonded, bedded and jointed with mortar, or of plain concrete which is not inferior in quality to concrete IV (By-law 14).

(ii) *Storey Heights.*—In By-laws 54 and 55 the thicknesses of walls are given for each storey height from the base to the top. For the purpose of applying these two by-laws, storey heights are to be taken according to the following rules: First, any storey height is to be the height between any two points where the wall is properly restrained laterally by the floors, or by the base and a floor, or by a floor and the roof. At such points the floors and roof must bear on the wall or be otherwise attached securely to it so as to afford sufficient lateral restraint to satisfy the district surveyor. Secondly, unless the lowest floor is more than 8 ft. above the base of the wall the bottom storey height (called the "First" storey height in By-laws 54 and 55) is to be taken as the height from the base to the second floor or lateral support above. Thus in Fig. 53 if q is less than 8 ft., p must be taken as the storey height; or in Fig. 54 if y is less than 8 ft., $(y + y_1)$ must be taken as the first storey height. In the latter case, Fig. 54, if the storey heights are being taken on the other side of the wall and if x is less than 8 ft., then $(x + x_1)$ constitutes the first storey height.

In Fig. 53:



if q is greater than 8 ft.

q = first storey height;

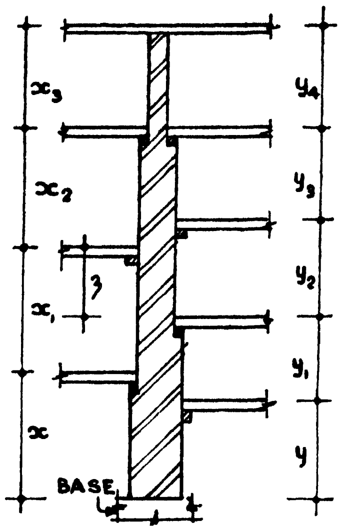
if q is less than 8 ft.

p = first storey height.

FIG. 53

If there are floors or other lateral supports on both sides of a wall, the storey heights are to be measured on one side only. In Fig. 54, subject to the 8-ft. limit for x or y, the storey heights would be either x, x_1, x_2, etc., or y, y_1, y_2, etc.

The storey height is not to be taken as the distance between floors on opposite sides of the wall. In Fig. 54 :



Storey heights $= x, x_1, x_2$, etc.

or $= y, y_1, y_2$, etc.

z must not be taken as storey height.

FIG. 54

(iii) *Only One Storey in Roof.*—The wall must not aid in supporting more than one storey in the roof. See Fig. 55.

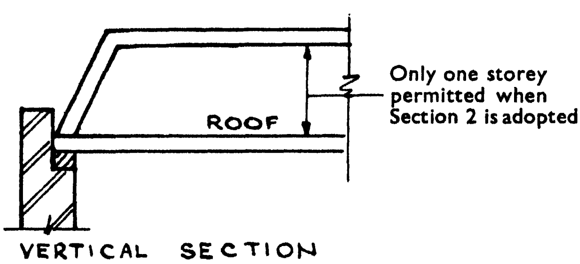


FIG. 55

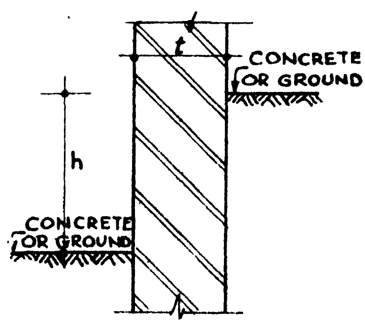


FIG. 56

h must not be greater than $3t$.

(iv) *Ground Levels.*—The level of the surface of the ground or of the concrete laid on the ground over the site on one side of the wall must not be more than three times the wall thickness above or below the corresponding level on the other side. See Fig. 56.

In many cases this clause prevents the use of this Section in the determining of the dimensions of a wall which encloses a basement, because the floor of the basement is frequently more than three times the wall thickness below the ground level.

(v) *Wall to carry only Distributed Loads.*—The wall must not carry any other loads than distributed loads. However, certain point or concentrated loads, if they are close enough, may be considered to be distributed loads. They are joists which bear directly on the wall and are not farther apart than 3 ft. 6 in., centre line to centre line, and joists of a timber floor properly carried by trimmer joists which bear on the wall, and which also are not at greater spacing than 3′ 6″.

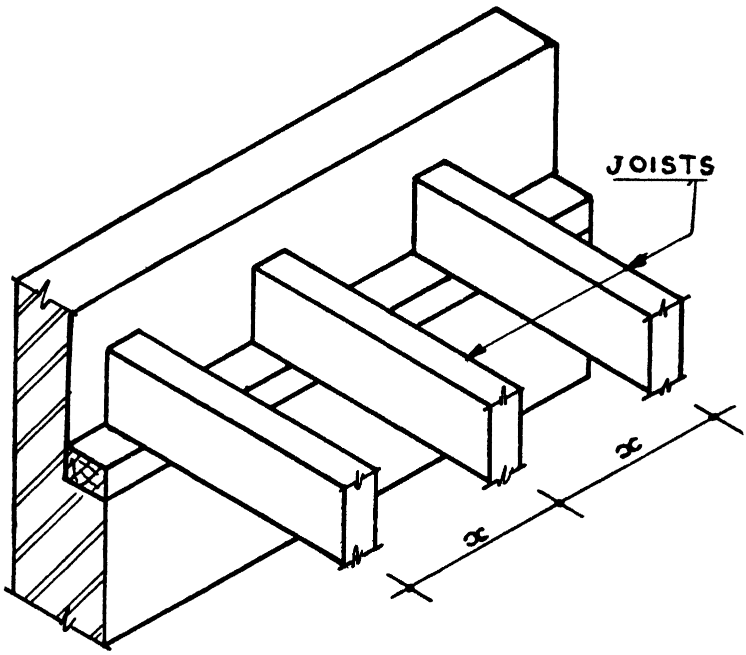


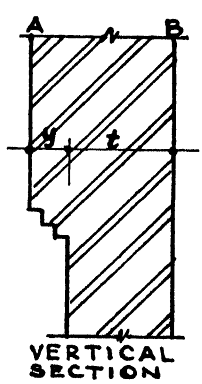
Fig. 57

Wall assumed to have a distributed load if x is not greater than 3 ft. 6 in.

(vi) *Overhanging Solid Walls.*—If any part of a solid wall overhangs any other part below, the overhanging portion must be regarded as an addition to the required thickness, and the opposite face of the wall must be carried straight up without any set-back from the face of the wall below.

The amount of overhang must not be greater than one-third of the required thickness of the wall, and it may be carried on corbels or in any other manner of which the district surveyor approves.

In Fig. 58 :



y must be additional to the required thickness t.

y must not be greater than $\frac{t}{3}$.

Wall face B must be carried up without a break.

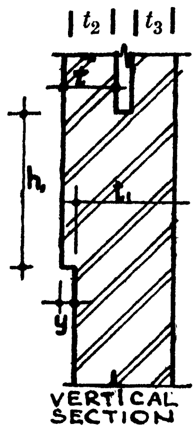
Fig. 58

A wall may still be constructed under Section 2 even though these conditions are not satisfied for part of the wall if that part be a separate (non-load bearing) wall (By-law 44 (2)) or if it be recessed in the manner described in clause (viii) which follows.

(vii) *Overhanging Cavity Walls.*—If any part of a cavity wall overhangs any other part below, the overhanging portion must be regarded as an addition to the required thickness, and the opposite face of the wall must be carried straight up without any set-back from the face of the wall below.

The amount of overhang must not be greater than one-third of the required thickness of the wall if there is solid material for the full thickness at the bottom, for a height of not less than the overall thickness of the cavity wall.

Thus in Fig. 59 :



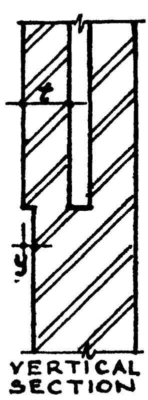
If h_1 is not less than t_1, y must not be greater than one-third of the required thickness of the cavity wall.

y must not be greater than $\frac{t_2+t_3}{3}$ when t_2+t_3 is the required thickness of the cavity wall.

FIG. 59

The amount of overhang must not be greater than one-third of the thickness of the over-hanging leaf if there is not solid material for the full thickness at the bottom for a height not less than the overall thickness of the cavity wall.

Thus in Figs. 59 and 60 :



If h_1 is less than t_1,

y must not be greater than $\frac{t}{3}$.

FIG. 60

The overhanging portion may be carried on corbels or in any other manner of which the district surveyor approves.

A wall may still be constructed under Section 2 even though these conditions are not satisfied for part of the wall if that part be a separate wall (By-law 44) or if it be recessed in the manner described in clause (viii), which follows.

(viii) *Proportion of Openings and Recesses.*[1]—In any storey height of

(*a*) any cross wall,
(*b*) any party wall, or
(*c*) any internal buttressing wall,

or in any storey height above the soffit of the first floor of

(*d*) any external wall :

the total area of openings and recesses cut into the required thickness of the wall must not be greater than one-half of the overall area of wall in that storey height.

The expression " recess " includes all chases and other reductions in the required thickness of the wall, and " opening " includes all glass or glazings.

In Fig. 61 the shaded areas represent openings or recesses.

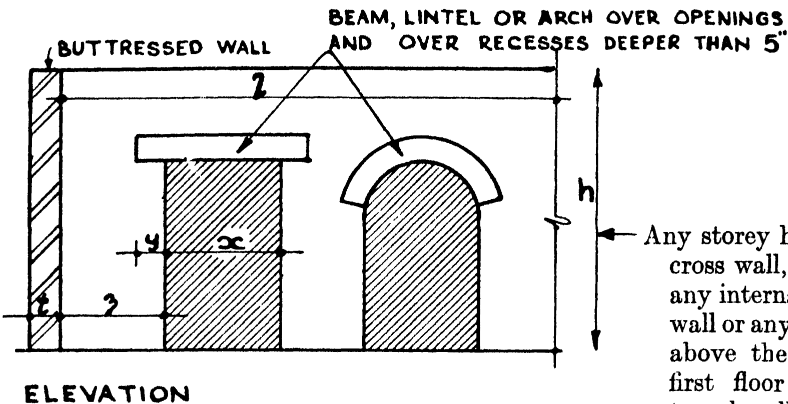


FIG. 61

The total shaded areas must not be greater than $\frac{h \times l}{2}$.

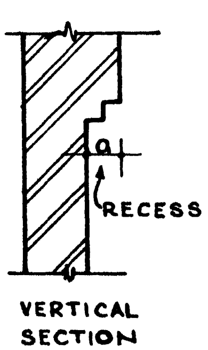
(ix) *Carrying of Superstructure over Openings and Recesses.*[1]—In any storey height of

(*a*) any cross wall,
(*b*) any party wall, or
(*c*) any internal buttressing wall,

or in any storey height above the soffit of the first floor of

(*d*) any external wall :

the superstructure above any openings or recesses must be carried on a beam, lintel or arch constructed of incombustible material and bearing adequately at each end



If *a* is less than 5 in. superstructure may be carried on corbels to the district surveyor's approval.

FIG. 62

[1] It seems to be by an oversight that internal load-bearing walls (load-bearing partition walls) are not mentioned under headings (viii) and (ix), but reasonably the same conditions should apply to openings and to the carrying of superstructure over such openings.

on a pier or wall for a distance which must not be less than one-sixth of the length of the opening or recess. In the case of a recess which is less than 5 in. deep, however, corbels of which the district surveyor must approve may be used to carry the superstructure. See Fig. 62.

The expression " recess " includes all chases and other reductions in the required thickness of the wall, and " openings " include all glass or glazings.

In Fig. 61 :

$$y \text{ must not be less than } \frac{x}{6}.$$

It must be assumed where there is a pier between two openings in the same wall, the superstructure being carried in each case by beams, lintels or arches over the openings, that the pier must not be less in length than one-sixth of the combined lengths of the two openings.

(x) *Width of Solid Material forming Buttressing Wall.*[1]—In any storey height of any buttressing wall which may be

(*a*) any cross wall, or
(*b*) any party wall, or
(*c*) any internal buttressing wall,

or in any storey height above the soffit of the first floor of

(*d*) any external wall :

the distance from the inside face of any buttressed wall to which it is bonded to the nearer face of an opening or recess must not be less than five times the thickness of the buttressed wall, unless the district surveyor should be satisfied that the stability of the building would not be prejudiced by a less distance. The expression " recess " includes all chases and other reductions in the required thickness of the wall and " opening " includes all glass or glazings.

In Fig. 61 :

$$z \text{ must not be less than } 5t$$

(unless the district surveyor agrees that z may be less).

In order that a wall may be considered to be a buttressing wall, for the purpose of this section (Section 2), it must conform to the requirements of the definition in By-law 1, namely, that its length must not be less than one-sixth of its height.

By-law 52. Length of Walls.—In By-laws 54 and 55 the thicknesses are given for walls of different heights and of different lengths. For the purpose of applying these two by-laws the length of the wall need not be taken as the overall length if it is braced properly at intermediate points by buttressing walls or piers. The buttressing walls and piers must be in accordance with the definitions in By-law 1 and with the requirement of clause (x) of By-law 51 above. The length of a wall to be used in finding the wall thickness from By-laws 54 and 55 may then be the clear distance between the buttressing members, provided that the latter continue for the full height up to the joists of the top floor, or up to the soffit of the top floor if there are no floor joists, or up to the top of the wall if the building is only of one

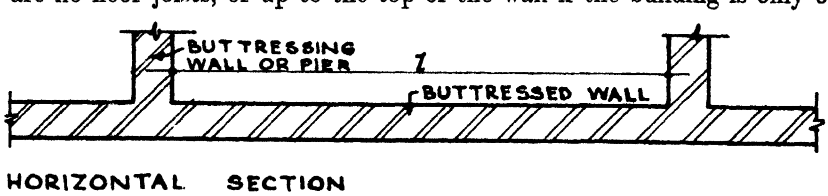
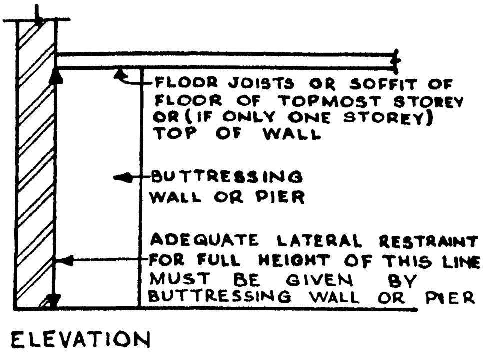


FIG. 63

[1] It seems to be by an oversight that internal load-bearing walls (load-bearing partition walls) are not mentioned under headings (viii) and (ix), but reasonably the same conditions should apply to openings and to the carrying of superstructure over such openings.

storey. Further, the buttressing walls or piers must be securely bonded to the buttressed wall to afford ample lateral restraint throughout this height.



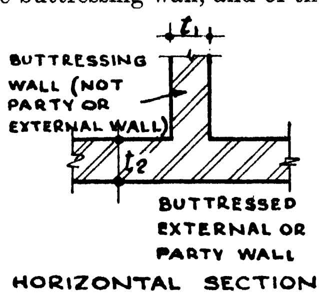
In Fig. 63:

l may be taken as the length of wall for the purposes of By-laws 54 and 55.

FIG. 64

By-law 53. Thickness of Walls.—*External and Party Walls.*—Every external wall and every party wall, with the exception of those walls given under note 6 below, must be of not less thickness than that required by By-law 54 or By-law 55. In addition, this by-law sets out under clause 7 below the cases in which required increase in thickness may be made in the form of piers. It specifies also the conditions for internal walls which are given in the following clauses 1 to 5.

1. *Buttressing Wall.*—Every wall which buttresses an external wall or party wall, but which itself is not such a wall, must have a thickness of not less than two-thirds of the thickness required for an external wall or party wall of the same height and length as the buttressing wall, and of the same class of building.

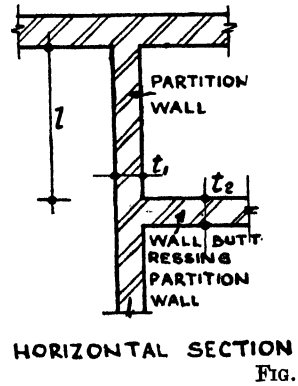


t_1 must not be less than $\frac{2}{3}t_2$, where t_2 is the required thickness, according to By-laws 44, 57 and 55.

FIG. 65

2. *Partition Wall.*—Every partition wall, except a non-load-bearing partition wall, must have a thickness of not less than one-half of the thickness required for an external or party wall of the same height but of twice the length and of the same class of building.

In Fig. 66:



t_1 must not be less than $\frac{tx}{2}$, where tx is the required thickness of an external or party wall, of the same height but of length equal to $2l$, and of the same class of building.

FIG. 66

Extreme care should be exercised in the application of this requirement, for the reason that in many cases the stresses become many times greater than those permitted by Section 3 for corresponding conditions.

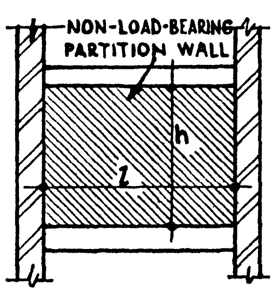
3. *Wall Buttressing Partition Wall.*—Every wall buttressing a partition wall other than a non-load-bearing partition wall must have a thickness of not less than one-half of the thickness required for an external or party wall of the same height but of twice the length and of the same class of building.

In Fig. 66, t_2 must not be less than half the thickness of an external or party wall of the same height but twice the length and of the same class of building.

4. *Internal Wall acting as External Wall.*—If any part of an internal wall should become an external wall, it and all the wall below it must have a thickness not less than that required for an external wall of the same height and length and of the same class of building.

5. *Non-load-bearing Partition Walls.*—A non-load-bearing partition wall which is securely restrained against lateral movement round all its edges and which is otherwise restrained if the district surveyor should consider it to be necessary, may have a thickness less than that given for partition walls above, but not less than one two-hundredth of the sum of its length and three times its height.

In Fig. 67 :



ELEVATION

if t = thickness of non-load-bearing partition wall :

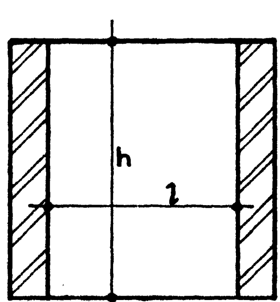
t must not be less than $\frac{l + 3h}{200}$,

FIG. 67

provided that the wall is adequately restrained on all edges and elsewhere if the district surveyor should require it.

Calcium sulphate plaster may be used for bedding and jointing bricks or blocks in such a non-load-bearing partition wall.

6. *External Cavity Walls.*—An external wall not greater than 25 ft. in height or 30 ft. in length, which is not a non-load-bearing panel or separate wall as allowed by By-law 44 and specified by By-law 56, but which is adequately restrained at each end, may be constructed as a cavity wall conforming to the requirements of By-law 45. That is to say, a load-bearing external wall, within the above limits of length and height and properly restrained at the ends, may be of cavity construction.



ELEVATION

FIG. 68

For the purpose of arriving at the thickness of such a wall, from By-laws 54 and 55, the sum of the thicknesses of the two leaves must not be less than the thickness required for a solid wall of the same length and height.

Thus, in Fig. 44, $a + b$ must not be less than the thickness required by By-law 54 or By-law 55, namely $8\frac{1}{2}$ in. for domestic buildings and 13 in. for warehouse buildings, if (in Fig. 68) h is not greater than 25 ft. and l

is not greater than 30 ft. and if the wall is properly restrained for the full height at each end.

7. *Special Increases in Wall Thicknesses may be confined to Piers.*—In By-laws 54 and 55 the thicknesses of walls over 60 ft. high have been increased from the thicknesses given for lengths not exceeding 45 ft. by $4\frac{1}{2}$ in. when the lengths do exceed 45 ft. Also, in By-law 54 if the thickness given for any storey height is less than one-sixteenth of that storey height, or in By-law 55 if the thickness given for any storey height is less than one-fourteenth of that storey height, the wall thicknesses and the thicknesses of any walls below must be increased to the latter values of one-sixteenth or one-fourteenth of the storey heights respectively.

However, the increase in thickness in each of these cases may be confined to piers which are distributed along the length of the wall and which have collective widths not less than a quarter of the length of the wall.

Thus in Fig. 69 if the piers are distributed along the length of the wall:

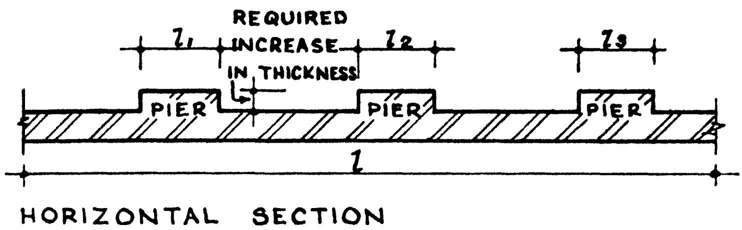


FIG. 69

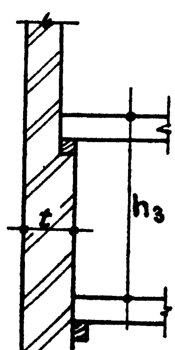
$$l_1 + l_2 + l_3, \text{ etc., must not be less than } \frac{l}{4}.$$

By-law 54. Thickness of Walls in Buildings other than Public or Warehouse.—Every external wall and every party wall of a building, other than of the warehouse or public building class, must have a thickness not less than that set out in Table 16 for the appropriate length and height.

This by-law applies to walls of which the thicknesses are determined under this section (Section 2) and not to walls of which the thicknesses are determined by calculation in accordance with Section 3.

In every case the thickness of the wall in any storey height and of any wall below must be not less than one-sixteenth of that storey height, and if this thickness is greater than that given in the table (Table 16), the latter may be made up to the required (storey height) thickness by piers constructed in the manner specified in By-law 53 (7).

In Fig. 70:



t = thickness of external or party wall in building other than of public or warehouse class.

h_3 = storey height.

t must not be less than value in Table 16, and

t must not be less than $\frac{h_3}{16}$.

FIG. 70

If the latter value is greater than the former the difference may be made up by piers in accordance with By-law 53.

TABLE 16

	BY LAW 54				THICKNESSES (WHEN NOT DETERMINED BY CALCULATIONS) OF EXTERNAL AND PARTY WALLS IN BUILDINGS OTHER THAN OF PUBLIC OR WAREHOUSE TYPE									
	HEIGHT OF WALL IN FEET		LENGTH OF WALL IN FEET		THICKNESS IN INCHES PER STOREY HEIGHT									
	EXCEEDING	NOT EXCEEDING	EXCEEDING	NOT EXCEEDING	1ST	2ND	3RD	4TH	5TH	6TH	7TH & 8TH	② ALL INTERMEDIATE STOREY HEIGHTS	NEXT UNDER TOP	TOP
a	—	12'	ANY LENGTH		8½"	–	–	–	–	–	–			
b	12'	25'	–	30'	8½"	–	–	–	–	–	–	8½"		8½"
b	12'	25'	30'	–	13"		–	–	–	–	–	8½"		8½"
c	25'	30'	–	20'	8½"	–	–	–	–	–	–	8½"		8½"
c	25'	30'	20'	30'	13"	–	–	–	–	–	–	8½"		8½"
c	25'	30'	30'	–	13"	13"	–	–	–	–	–	8½"		8½"
d	30'	40'	–	35'	13"	–	–	–	–	–	–	13"		8½"
d	30'	40'	35'	–	17½"	–	–	–	–	–	–	13"		8½"
e	40'	50'	–	35'	17½"	–	–	–	–	–	–	13"		8½"
e	40'	50'	35'	45'	17½"	17½"	–	–	–	–	–	13"		13"
e	40'	50'	45'	–	21½"	17½"	–	–	–	–	–	13"		13"
f	50'	60'	–	45'	17½"	17½"	–	–	–	–	–	13"		13"
f	50'	60'	45'	–	21½"	17½"	17½"	–	–	–	–	13"		13"
g	60'	70'	–	45'	21½"	17½"	17½"	–	–	–	–	13"		13"
g	60'	70'	45' ①	–	26"	22"	22"	–	–	–	–	17½"	13"	13"
h	70	80'	–	45'	21½"	17½"	17½"	17½"	–	–	–	13"		13"
h	70'	80'	45 ①	–	26"	22"	22"	22"	–	–	–	17½"	13"	13"
i	80'	90'	–	45'	26"	21½"	17½"	17½"	17½"	–	–	13"		13"
i	80'	90'	45 ①	–	30½"	26"	22"	22"	22"	–	–	17½"	13"	13"
j	90'	100'	–	45'	26"	21½"	21½"	17½"	17½"	17½"	–	13"		13"
j	90'	100'	45 ①	–	30½"	26"	26	22	22	22"	–	17½"	13"	13"
k	100'	120'	–	45	30"	26"	26"	21½"	21½	17½"	17½"	13"		13"
k	100'	120'	45 ①	–	34½"	30½"	30½"	26"	26"	22"	22	17½"	13"	13"

① THE EXTRA 4½" THICKNESS OF WALL REQUIRED BECAUSE THE LENGTH EXCEEDS 45 FEET MAY BE CONFINED TO PIERS PROPERLY DISTRIBUTED, OF WHICH THE COLLECTIVE WIDTHS AMOUNT TO AT LEAST A QUARTER OF THE TOTAL LENGTH OF THE WALL AS REQUIRED BY BYLAW 53 C.

② THIS IS TO BE THE WALL THICKNESS ON ALL STOREY HEIGHTS BETWEEN THE STOREY HEIGHTS FOR WHICH THE THICKNESSES ARE GIVEN TO THE LEFT AND TO THE RIGHT OF COLUMN ②

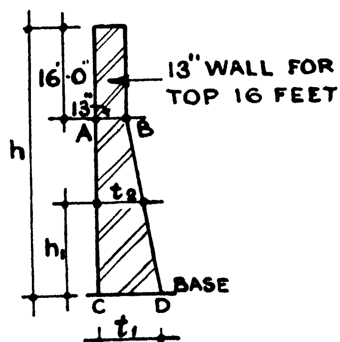
The extra $4\frac{1}{2}$-in. thickness required on the thickness of a wall over 60 ft. high, but not more than 45 ft. long, because a wall of the same height is over 45 ft. long, may also be added locally in the form of piers, the piers being in accordance with By-law 53.

By-law 55.—Thickness of Walls in Buildings of Warehouse Class.—In buildings of the warehouse class, every external wall and every party wall of any length and of a height not greater than 60 ft., and every external wall and every party wall of any height and of a length not greater than 45 ft., must have a thickness for the top 16 ft. of not less than 13 in., and a thickness at the base not less than the appropriate value of t_1 given in Table 17, and a thickness at any height h_1 above the base of not less than t_2 such that

$$t_2 = 13 + \frac{(t_1 - 13)(h - 16 - h_1)}{(h - 16)} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

t_1 and t_2 being in inches and h and h_1 being in feet.

In Fig. 71 :



thickness t_2 at any height h_1 to be not less than the horizontal distance between the lines AC and BD at the height h_1.

Values of t_1 are given in Table 17.

Fig. 71 is purely diagrammatic.

FIG. 71

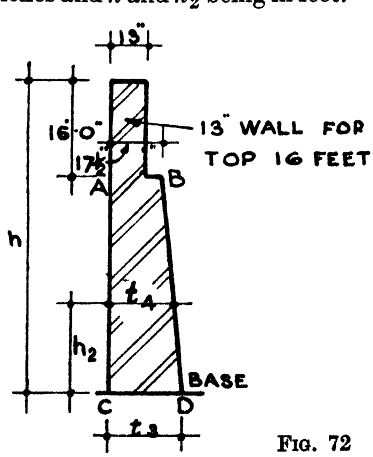
If the total height is not greater than 30 ft. the requirement for a minimum thickness of 13 in. for the top 16 ft. does not apply.

In buildings of the warehouse class every external wall and every party wall of greater height than 60 ft. and of greater length than 45 ft. must have a thickness for the top 16 ft. of not less than 13 in., and a thickness at the base not less than the appropriate value of t_3 given in Table 17, and a thickness at any height h_2 above the base of not less than t_4 such that

$$t_4 = 17\tfrac{1}{2} + \frac{(t_3 - 17\frac{1}{2})(h - 16 - h_2)}{(h - 16)} \quad . \quad . \quad . \quad . \quad (2)$$

t_3 and t_4 being in inches and h and h_2 being in feet.

In Fig. 72 :



thickness t_4 at any height h_2 to be not less than the horizontal distance between lines AC and BD at the height h_2.

Fig. 72 is purely diagrammatic.

FIG. 72

TABLE 17

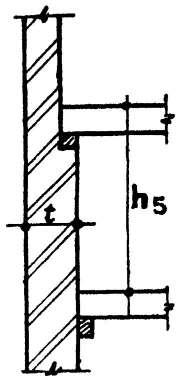
THICKNESSES (WHEN NOT DETERMINED BY CALCULATION) OF EXTERNAL AND PARTY WALLS IN BUILDINGS OF WAREHOUSE CLASS

	Height of Wall		Length of Wall		t_1, in.	t_3, in.
	Exceeding, ft.	Not exceeding, ft.	Exceeding ft.	Not exceeding, ft.		
a	—	25	Any	length	13	—
b	25	30	—	45	13	—
b	25	30	45	—	17½	—
c	30	40	—	35	13	—
c	30	40	35	45	17½	—
c	30	40	45	—	21½	—
d	40	50	—	30	17½	—
d	40	50	30	45	21½	—
d	40	50	45	—	26	—
e	50	60	—	45	21½	—
e	50	60	45	—	26	—
f	60	70	—	45	21½	—
f	60	70	45 [1]	—	—	26
g	70	80	—	45	21½	—
g	70	80	45 [1]	—	—	26
h	80	90	—	45	26	—
h	80	90	45 [1]	—	—	30½
i	90	100	—	45	26	—
i	90	100	45 [1]	—	—	30½
j	100	120	—	45	31	—
j	100	120	45 [1]	—	—	35½

It will be seen that the thicknesses of these walls, up to a level 16 ft. from the top are 4½ in. greater than the thicknesses for walls of corresponding heights but of lengths not greater than 45 ft. This extra thickness may be made up in the form of piers distributed along the length of the wall and constructed in accordance with By-law 53 (7). In every case the thickness of the wall in any storey height and of any wall below must not be less than one-fourteenth of that storey height, and if this thickness is greater than that given in Table 17 the latter may be made up to the required storey height thickness by piers constructed in the manner specified in By-law 53 (7).

[1] The extra 4½ in. thickness of wall required because the length exceeds 45 ft. may be confined to piers properly distributed, of which the collective widths amount to at least a quarter of the total length of the wall, as required by By-law 53.

In Fig. 73 :



t = thickness of external or party wall in building of warehouse class.
h_5 = storey height.
t must not be less than t_2 or t_4, depending on whether the wall is over 60 ft. high and 45 ft. long, and
t must not be less than $\frac{h_5}{14}$.
(For t_2 and t_4 see p. 73)

FIG. 73

If the latter value of t is greater than the former the difference may be made up by piers in accordance with By-law 53.

By-law 56. Height and Thickness of Separate or Panel Walls.—The height of a part or panel of a wall which is considered under By-law 44 (2) to be a separate wall must not be greater than 25 ft., and the thickness, exclusive of all finishings, must not be less than one-eighteenth of the length or of the height, whichever is the less.

In Fig. 74 :

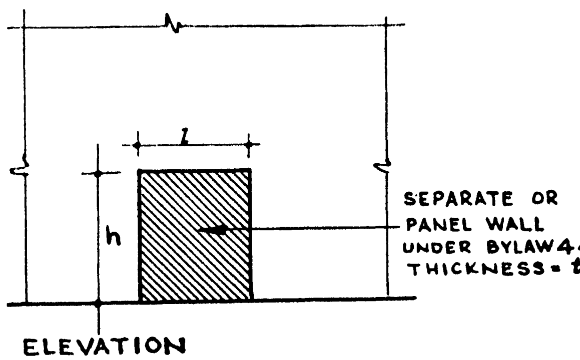


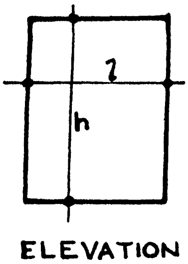
FIG. 74

h must not be greater than 25 ft.

$$\left.\begin{array}{l} t \text{ must not be less than } \frac{h}{18} \\ \qquad\text{or} \\ t \text{ must not be less than } \frac{l}{18} \end{array}\right\} \text{whichever is the less.}$$

Separate Walls of Cavity Construction.—Such a separate wall may be of cavity construction in accordance with By-law 45, and in this case the superficial area must not exceed 200 sq. ft. and either the height or the length must not exceed 13 ft.; and the thickness (taken as the sum of the thicknesses of the two leaves) must not be less than one-eighteenth of the length or of the height, whichever is the less.

In Fig. 75:



$h \times l$ must not be greater than 200 sq. ft.
either h or l must not be greater than 13 ft.

$$\left.\begin{array}{c} t \text{ must not be less than } \frac{h}{18} \\ \text{or} \\ t \text{ must not be less than } \frac{l}{18} \end{array}\right\} \text{whichever is the less.}$$

FIG. 75

By-law 57. Underpinning.—Where it is necessary to underpin any portion of an existing structure, the underpinning must be to the full thickness of the existing work or to any greater thickness which would be required for a wall or pier under this section (Section 2).

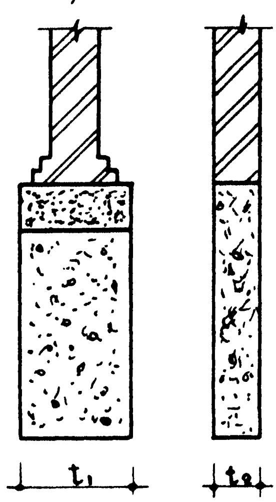


FIG. 76

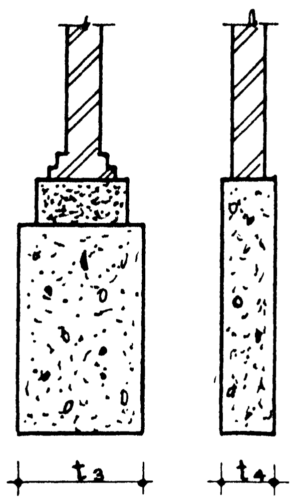


FIG. 77

Thus in Fig. 76:

the thickness of underpinning t_1 and t_2 must not be less than the thickness of the work to be carried, nor,

less than t_3 and t_4 (in Fig. 77), where t_3 and t_4 are the thicknesses required under this section (Section 2), and where they are greater than t_1 and t_2 respectively.

All underpinning must be to the satisfaction of the district surveyor. For fuller information see Chapter 36, Volume 3.

PART VI. SECTION 3

Permissible Stresses for the Calculation of Wall and Pier Thicknesses.—The thickness of a wall or pier may be ascertained by calculation instead of by the empirical rules given in Section 2. This section (Section 3) sets out the limiting stresses and other requirements governing wall thicknesses when they are determined by calculation. If the thickness of any wall calculated under this section be different from that required by any by-law of Section 1 of this part (Part IV), then the wall must actually be constructed to the greater thickness.

By-law 58. Piers and Walls on Adjacent Storey Heights.—If a pier in any storey height is supported on part of a wall in the storey below, then the load must be calculated as being carried on a pier in the lower storey height. In the second storey below, if there is also a wall, the pier load may be regarded, however, as distributed over such a length of that wall which would come within the 45° angle of dispersion of load. If the width of the pier is w and if the storey height is h_2, then the load may be distributed over a length of wall not greater than $(w + 2h_2)$.

In Fig. 78 the load of the pier in storey height h_3 must be carried on the wall h_2 by a portion calculated as a pier. The load of the pier in h_3 may, however, be con-

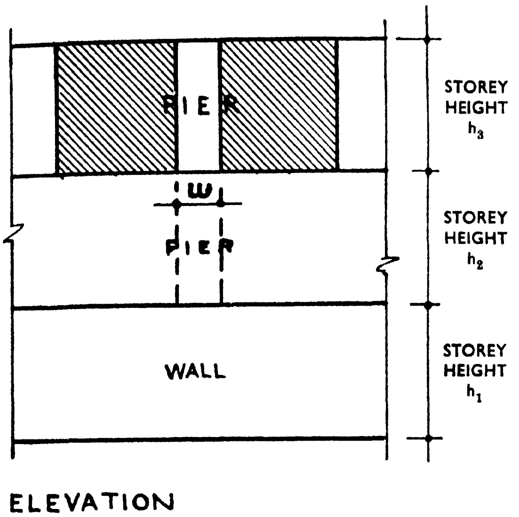


FIG. 78

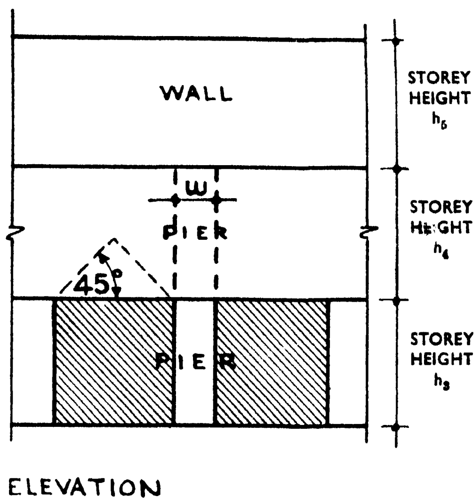


FIG. 79

sidered as distributed on the wall in h_1 over a length $(w + 2h_2)$.

In the same way if a pier in any storey height supports part of a wall in the storey height above, then the load on that part of the wall must be calculated as being carried on a pier in the upper storey height. In the second storey above, if there is also a wall, the load may, however, be regarded as distributed over the length of that wall which would be subjected to the load as dispersed at an angle of 45°.

In Fig. 79 the load may be calculated as distributed on the wall in storey height h_5, but on a pier in h_4 if there is a pier in storey height h_3.

Piers and Walls in Combination on Same Storey Height.

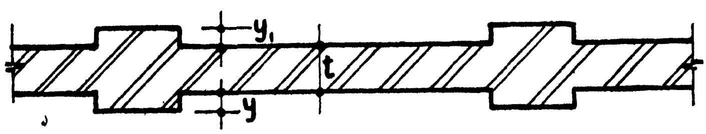
Although not specifically mentioned in the by-law, a wall carried on a pier as in the second case above will have to transmit its load to that pier by means of a beam. If the preceding rules are observed, namely that the pier must be considered to continue through the next storey height above, then the beam could be calculated to carry only the portion of wall load within a 45° triangle over the openings, as shown in Fig. 79. It would, of course, also have to carry any floor or other loads which it might get in any other manner; but these special loads are independent of the consideration of the amount of wall load which it would have to resist. If, however, a beam is provided which is capable of carrying the whole of the wall load from above and not just that portion within the 45° triangle, then it is not reasonable to consider that the pier continues through the next storey height.

By-law 59 distinguishes between piers and walls for the determination of the slenderness ratio. It is important, therefore, to make this distinction when walls and piers are in combination in the same storey height.

Although the actual wording of this portion of the by-law relating to piers and walls in structural combination, on the same storey height, is rather ambiguous, it would appear that the intention is that which is set out in the following paragraphs.

If a pier which is in structural combination with a wall projects from one face of the wall to an extent not greater than one-quarter of the wall thickness, then the combined pier and wall must be regarded, as far as this section (Section 3) is concerned, as being just a wall and the thickness must be taken as that of the wall for purposes of determining the slenderness ratio ; but the full area may be used for load bearing.

Similarly, if such a pier in combination with a wall projects on each side of the wall in such manner that the sum of the two projections from the wall faces is *not greater* than one-third of the wall thickness, then the combined pier and wall must be regarded, as far as this section (Section 3) is concerned, as being just a wall, and the thickness must be taken as that of the wall, for purposes of determining the slenderness ratio ; but the full area may be taken for load bearing.



HORIZONTAL SECTION

FIG. 80

In Fig. 80 if the pier projects only on one side of the wall such that y_1 (or y if the projection is only on the opposite side of the wall) is not greater than $\frac{t}{4}$, then the combined construction is to be regarded as a wall.

Also, if the pier projects on both sides of the wall such that $(y_1 + y)$ is not greater than $\frac{t}{3}$, then the combined construction is to be regarded as a wall.

If a pier which is in structural combination with a wall projects from one face of the wall to an extent *greater* than one-quarter of the wall thickness and if this projection forms part of the required thickness, then the combined pier and wall must be regarded, as far as this section (Section 3) is concerned, as being a pier, and the thickness, for the purpose of the slenderness ratio, may be taken as being the distance from the face of the projection to the opposite face of the wall.

Similarly, if such a pier in combination with a wall projects on each side of the wall in such manner that the sum of the two projections from the wall faces is *greater* than one-third of the wall thickness and if the projections form part of the required thickness, then the combined pier and wall must be regarded, as far as this section (Section 3) is concerned, as being a pier, and the thickness for the purpose of the slenderness ratio may be taken as being the distance from the face of the projection on one side of the face of the projection on the opposite face of the wall.

In Fig. 80 if one pier projects only on one side of the wall such that y_1 (or y if the projection is only on the opposite side of the wall) is greater than $\frac{t}{4}$, then the combined construction is to be regarded as a pier.

Also, if the pier projects on both sides of the wall such that $(y_1 + y)$ is greater than $\frac{t}{3}$, then the combined construction is to be regarded as a pier.

By-law 59. Storey Heights for calculating Slenderness Ratios.—In By-law 60, within certain upper and lower limits, the permissible stresses on walls and piers are dependent on the slenderness ratios of those walls and piers.

For the purpose of computing the slenderness ratio in any storey height of a wall or pier constructed of bricks, blocks or plain concrete, the effective height must be divided by the horizontal dimension of the wall or pier measured in the direction of the lateral supports which determine the storey height. For walls and piers in combination the horizontal dimension which must be taken in any case is defined in By-law 58.

The following are the "effective heights" which must be used in calculating the slenderness ratio for a specified case :

A. Effective height of wall in a storey height, h

(i) without lateral support at the top : $1{\cdot}5h$;
(ii) with " " " " : $0{\cdot}75h$.

B. Effective height of a pier in a storey height, h

(i) without lateral support at the top : $2{\cdot}0h$;
(ii) with " " " " : $1{\cdot}0h$.

By-law 60. Permissible Pressures on Brick, Block or Plain Concrete Walls and Piers. *Brick or Block Walls and Piers.*—In a wall or pier in any storey height, constructed of bricks or blocks, the total compressive stress due to vertical, lateral and any other loading must not exceed the appropriate value given in Table 18.

TABLE 18

PERMITTED STRESSES IN BRICK OR BLOCK WALLS OR PIERS. TONS PER SQ. FT.

Designation of Bricks or Blocks (By-law 19)	Proportion of Mixture of Mortar by Volume			Slenderness Ratio						
	Cement	Lime	Sand	6 or less	7	8	9	10	11	12
Special	1	—	2	K	·9K	·8K	·7K	·6K	·5K	·4K
1st	1	—	2½	30	27·0	24·0	21·0	18·0	15·0	12·0
2nd	1	—	2½	23	20·7	18·4	16·1	13·8	11·5	9·2
3rd	1	—	3	16	14·4	12·8	11·2	9·6	8·0	6·4
4th	1	—	3	13·5	12·15	10·8	9·45	8·1	6·75	5·4
5th	1	—	4	11	9·9	8·8	7·7	6·6	5·5	4·4
5th	1	1	6	10	9·0	8·0	7·0	6·0	5·0	4·0
6th	1	—	4	8	7·2	6·4	5·6	4·8	4·0	3·2
6th	1	1	6	7	6·3	5·6	4·9	4·2	3·5	2·8
6th	1	2	9	6	5·4	4·8	4·2	3·6	3·0	2·4
6th	1	3	12	5·5	4·95	4·4	3·85	3·3	2·75	2·2
6th	1	4	15	5	4·5	4·0	3·5	3·0	2·5	2·0
6th	1	5	18	4·5	4·05	3·6	3·15	2·7	2·25	1·8
6th	—	1	3	4	3·6	3·2	2·8	2·4	2·0	1·6

The value K (the permissible stress on a special brick or block wall or pier of slenderness ratio 6 or less than 6) is to be determined by adding 10 to one-five-hundredth part of the number of pounds per square inch of the resistance of the brick or block to crushing. Thus, if the resistance to crushing (without cracking or breaking) is 13,000 lb. per sq. in. for a special brick,

$$K = \left(\frac{13,000}{500}\right) + 10 = 26 + 10 = 36 \text{ tons per sq. ft.}$$

In no case, however, must the value of K be taken higher than 40 tons per sq. ft.

It will be seen that working stresses have been given both for different qualities of bricks or blocks (as defined in By-law 19) and for different mortar compositions. Therefore, in any particular case the permissible stress on the structural unit is to be taken as that specified for the brick or block or for the mortar, whichever of these materials is the weaker. For example, if a wall, having a slenderness ratio of 8, is built of third-class bricks in 1 : 4 (cement : sand) mortar, the permissible stress would be only 8·8 tons per sq. ft. as governed by the mortar, whereas if the wall were built in 1 : 3 mortar the stress could be 12·8 tons per sq. ft.

Plain Concrete Walls and Piers.—In a wall or pier in any storey height constructed of plain concrete the total compressive stress due to vertical, lateral and any other loading must not exceed the appropriate value given in Table 19.

TABLE 19

PERMITTED STRESSES IN PLAIN CONCRETE WALLS OR PIERS. TONS PER SQ. FT.

Designation of Concrete as regards to Strength (By-law 14)	Cubic feet of Aggregate per 112 lb. of Cement		Slenderness Ratio						
	Fine Aggregate	Coarse Aggregate	6 or less	7	8	9	10	11	12
I	$1\frac{1}{4}$	$2\frac{1}{2}$	40	36	32	28	24	20	16
II	$1\frac{7}{8}$	$3\frac{3}{4}$	35	31·5	28	24·5	21	17·5	14
III	$2\frac{1}{2}$	5	30	27	24	21	18	15	12
IV	$7\frac{1}{2}$		20	18	16	14	12	10	8
V	10		15	13·5	12	10·5	9	7·5	6

The By-law gives also the stresses which would be permitted on walls or piers consisting of plain concrete of designations VI and VII; but because By-law 14 definitely prohibits the use of such concretes in the construction of a building or any part of it, they have been omitted from Table 19, which refers to walls and piers in any storey height of a building.

Brick, Block or Plain Concrete Walls or Piers.—In any brick, block or plain concrete wall or pier, or in any storey height of such, the total compressive stress due to vertical lateral and any other loading is dependent on the slenderness ratio, and the permissible values are set out in Tables 18 and 19 for slenderness ratios from 6 up to 12. Values of the stresses for intermediate ratios may be found by interpolation. The slenderness ratio is the effective height (as defined in By-law 59) divided by the thickness (see By-law 58).

In no load-bearing wall or pier constructed of bricks, blocks or plain concrete, or in any storey height of such, may the slenderness ratio exceed 12; but, however, in a cavity wall or partition wall built in accordance with By-law 45, with the special requirements of Section 2 and with By-law 53, the slenderness ratio may be greater than 12. This requirement agrees with By-law 54 in which for buildings other than of the warehouse class it is specified that the thickness should not be less than

$\frac{1}{16}$ of any storey height. It must be remembered that according to By-law 59 the slenderness ratio is to be taken as $0{\cdot}75\frac{h}{t}$. Thus, if $\frac{h}{t} = 16$ the slenderness ratio becomes 12.

It should be noted that the permitted stresses in this section are higher for certain slenderness ratios than for foundation piers; but no explanation can be offered for this discrepancy.

Increased Stress for Local Loading.—The permissible pressures given in Tables 18 and 19 for walls and piers of bricks, blocks or plain concrete may be increased by an amount up to 20 per cent. if the load is purely local (for example, under a beam or column), and if that concentrated load is distributed immediately to the surrounding material not subjected to such loading.

Different Materials in Combination.—If materials of different designations are used in the same storey height for a wall or pier, the permissible stress must be taken as that given in the tables for the weaker material.

Combining of New and Old Work.—The district surveyor must approve working pressures when existing walls are thickened or when new and old work are combined in any other manner.

Materials not Designated.—If the correct designations of the materials have not been determined (By-law 14 and By-law 19) from which permitted stresses could be obtained from Tables 18 and 19, the district surveyor must approve the working stress which is adopted.

By-law 61. Restrictions on Shear and Tensile Forces in Walls or Piers.—Only in arches, lintels, corbels, footings and similar constructions may brick, block or plain concrete masoning be considered to resist any calculated shearing or tensile forces, and in such cases the permitted stresses may be taken at not more than one-tenth of the permitted compressive stress for a wall or pier having a slenderness ratio less than 6. In no other case, except that of wind loads, for which the conditions are given below, may any allowance in the design be made for the shear and tensile resistance of a brick, block or plain concrete wall or pier.

Resistance of Walls to Wind Pressures.—The above limitations on shear and tensile forces do not apply to wind loads on a brick, block or plain concrete wall which is restrained laterally by buttressing walls, piers or other constructions, of which the district surveyor approves, if the length of wall between such supports is not more than forty-five times the wall thickness nor more than 45 ft., whichever is the less. Thus, if these conditions are satisfied, the wall may be considered to transmit horizontally to the lateral supports 25 per cent. of the wind load on the wall surface, and the wall may, therefore, be calculated to resist forces due to only 75 per cent. of the wind pressures which are specified in By-law 6.

Walls can generally be relied upon to transfer this 75 per cent. to the base and floors. The bending moments calculated in this way together with the maximum and minimum vertical loads define the stresses which (*a*) must remain within the permitted limit, and (*b*) must be compressive throughout. It may be assumed that a waiver allowing $33\frac{1}{3}$ per cent. increased stress for this condition would be granted.

By-law 62. Underpinning.—The working stresses prescribed in the by-laws of this section (Section 3) must not be exceeded in any brick, block or plain concrete work used for purposes of underpinning, and all such work must be to the satisfaction of the district surveyor.

Although this by-law does not require that underpinning should be to the full thickness of the work which is to be carried, it is considered that this condition should be satisfied and that the calculated thickness should be used only if it is greater than that of the existing work, and not if it is less.

Refer also to Chapter 36 for fuller information on foundations and underpinning.

CHAPTER 7

PART V. STRUCTURAL STEELWORK. BY-LAWS 63–91

The Use of Structural Steel.—This Part relates to the use of common structural steel in buildings, and as the requirements for waivers to permit the use of other steels have been discussed already, under By-law 15 in Part III (Materials), it is not proposed to go again into that matter here. However, the regulations for applications to use welding will be discussed at the end of this Part. (Pages 112–121).

In Chapters 26 to 32 are set out calculations and designs of structural steelwork in a detailed way, and only the actual requirements of the By-laws have been given here.

By-law 63. Structural Steel to comply with the By-laws.—" Structural steel shall comply with these By-laws ; and such steel shall not be relied upon to support, collect or transmit loading otherwise than as provided in these By-laws.

" Structural steel shall, as regards the shape and dimensions of cross-section, comply with the British Standard Specification for Channels and Beams numbered 4—1932, with the British Standard Specification for Angles and Tees numbered 4A—1934 or with the British Standard Specification for Bulb Angles and Bulb Plates numbered 6—1924, as the case may be. Provided that the district surveyor shall be satisfied as regards the shape and dimensions of a cross-section for which there is no appropriate British Standard Specification."

The steel itself is specified in By-law 15 as in accordance with British Standard Specification No. 15—1936, or British Standard Specification 548 under the conditions imposed by the Council, or as any other approved steel used under the Council's conditions.

By-law 64. Supports for Steelwork.—" All construction which will support or transmit loading supported, collected or transmitted by steel, shall comply with these by-laws."

That is to say, all walls or piers, or any other constructions on which the loads carried by steelwork are imposed, and the steelwork itself, must be in accordance with the particular by-laws for such walls or piers or the other constructions, or for structural steelwork.

By-law 65. Foundations for Steelwork.—" Loading supported, collected or transmitted by structural steel shall be distributed upon the earth by concrete which shall :

" (*a*) comply with the requirements of By-law 32 in the same manner as is required for concrete which is to support walls or piers ;

" (*b*) if plain, be of composition and quality not inferior to that designated V in By-law 14. The angle of dispersion through such plain concrete shall be taken as not less than 45° with the horizontal ; and such plain concrete shall not be relied upon to resist tensile stresses otherwise than in accordance with this by-law ; and "(c) if reinforced, comply with the requirements of these by-laws relating to the use of reinforced concrete.

The pressure upon such concrete shall be calculated, and such concrete shall, if plain, comply in all respects with the requirements of By-law 35 in the same manner as is required for plain concrete which is to support walls or piers."

By-law 66. Protection of Steelwork.—Protection must be provided for steel against any damage which might reduce the stability of the building or any part of it. In most cases a covering of concrete or other incombustible material is required by By-law 68, and whereas periodic painting will provide adequate protection in the majority of other cases, further precautions may be necessary in particular instances.

By-law 67. Scope of Structural Steel By-laws.—Except for By-law 91, the remaining by-laws of this part, namely By-laws 68 to 90, apply directly to only two types of buildings.

In the first type all loads and stresses must be transmitted through each storey height down to the foundations entirely by a framework of structural steel.

In the second type all loads must be transmitted through each storey height down to the foundations partly by a framework of structural steel and partly by a party wall or by party walls. In this type, therefore, an interior frame of steel may be used as well as any steel in external walls; but where, on any side, there is a party wall which is in accordance with the By-laws, that wall may be used to carry some of the loads.

Steelwork for buildings of either of these two types may be designed and constructed in conformity with the structural steel by-laws; but where structural steel is used in a building in any other manner, for instance to bring loads to internal or external walls, it must, according to By-law 91, be designed and constructed to satisfy the district surveyor and have a standard of stability and fire protection at least equal to that required for buildings of the first two types. Thus By-laws 68 to 90 apply indirectly to structural steel used in ways other than those to which By-laws 68 to 90 refer specifically.

By-law 68. Casing for Structural Steel.—The Council are preparing requirements for the cover on steelwork which may be used as alternatives to those given in this by-law. It is probable that these alternative requirements (which take into account the actual fire-resistance of materials as determined under test) will come into operation in 1939. It is impossible yet to give these requirements; but they will be included in Volume 2 as an Appendix.

Except in a single-storey building, not higher than 25 ft., in which there is no structural steel in any party or external wall, every structural steel beam and column must be protected against the effects of fire by a casing of incombustible materials. The casing is to consist of brickwork, terra-cotta, concrete, stone, tiles, or other equally incombustible material or of a suitable combination of these. Portland cement mortar must be used for the bedding and jointing of all materials which are in the form of blocks and all joints must be completely filled. Casing must be hard against the steel without any intervening voids, and it must have the thickness given for the appropriate case in the following paragraphs:

(a) *Columns in Party and External Walls.*—On every steel column wholly or partly in

(i) an external wall, or
(ii) a recess in a party wall,

not less than 4 in. at any part.

In Figs. 81 and 82:

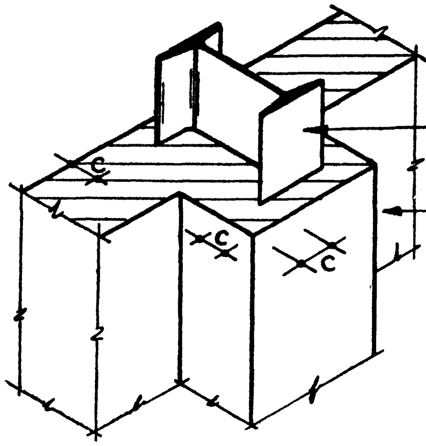


FIG. 81

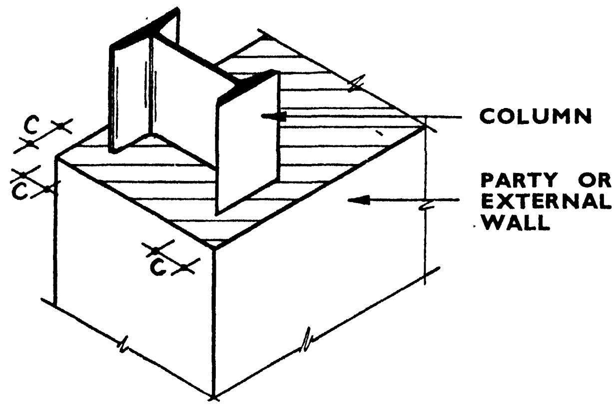


FIG. 82

C must not be less than 4 in.

(*b*) *Other Columns.*—On every column (except one in an external or party wall and except one elsewhere in a building less than 25 ft. high of only one storey) not less than 2 in., except on projecting cleats and rivet heads, etc., where the thickness must be not less than 1 inch.

In Fig. 83:

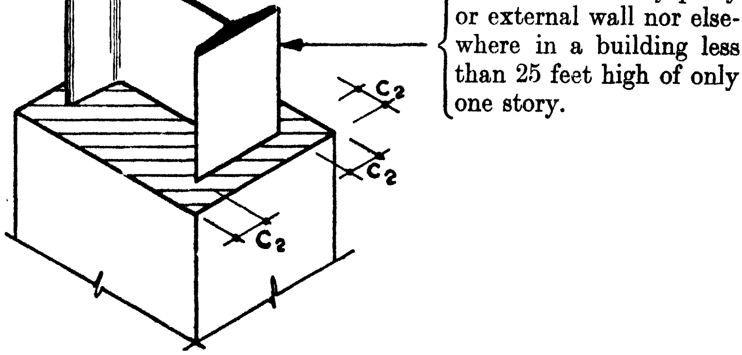


FIG. 83

C_2 must not be less than 2 in.

(c) *Beams in Party and External Walls.*—On the top flange and the web of every beam wholly or partly in

(i) an external wall, or
(ii) a recess in a party wall,

not less than 4 in. except on plates and angles connected to the flange, where the casing must be not less than 2 in. See Figs. 84 and 85.

(d) On the bottom flange, at the edges of all flanges, and on all plates and angles connected to the flanges, of every beam wholly or partly in

(i) an external wall, or
(ii) a recess in a party wall,

not less than 2 in. See Figs. 84, 85 and 86.

In Figs. 84, 85 and 86 :

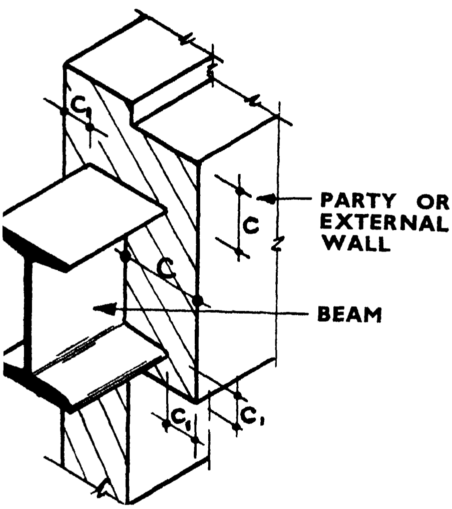


FIG. 84

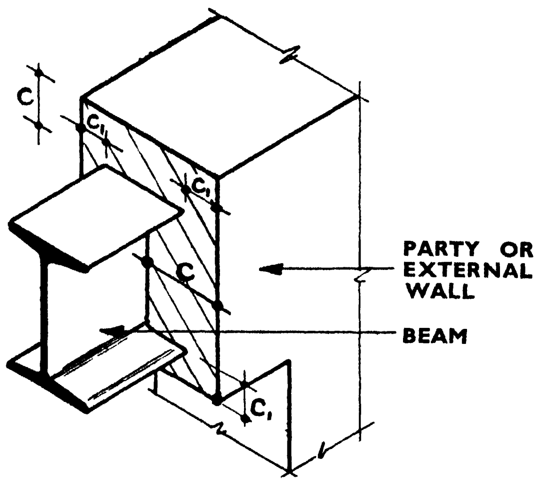


FIG. 85

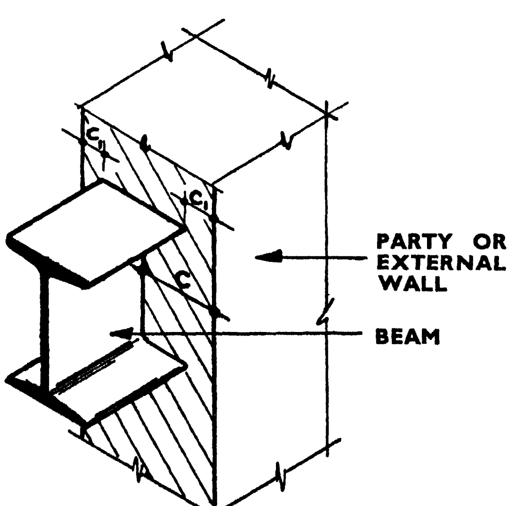


FIG. 86

C must not be less than 4 in.
C_1 „ „ „ „ 2 in.

(*e*) *Other Beams.*—On the bottom flange and the web and at the edges of the flanges of every beam (except one in an external or party wall and except one elsewhere in a building less than 25 ft. high of only one storey), not less than 2 in., except on projecting cleats and rivet heads, etc., where the casing must be not less than 1 in. See Fig. 87.

(*f*) On the top flange and on projecting cleats and rivet heads, etc., of every beam (except one in an external or party wall and except one elsewhere in a building less than 25 ft. high of only one storey) not less than 1 in. See Fig. 87.

In Fig. 87 :

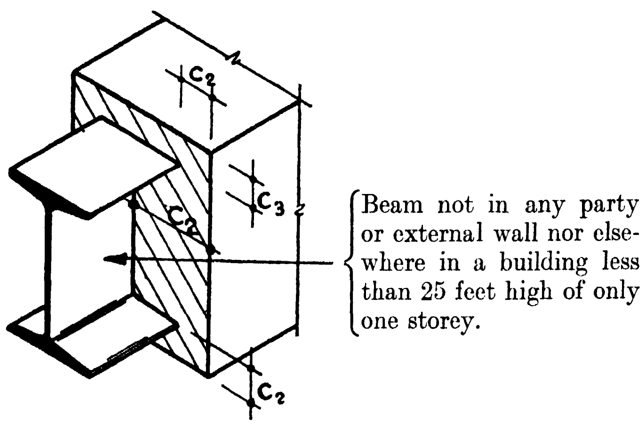


FIG. 87

C_2 must not be less than 2 in.
C_3 ,, ,, ,, ,, 1 in.

There is no restriction now on re-entrant angles in the casing, which can therefore be made, when necessary, to follow the lines of the section encased. This is not to be recommended nor has it any economic advantages, but in certain special cases it may be necessary.

By-law 69. Column Bases.—The foot of every steel column, which is not of solid round section or which is not for a one-storey building, must, after being riveted up, be machined over the whole area of the foot and have attached to it, in close contact, a base plate which must be of either of two types.

Stiffened Base Plate.—The first type may consist of a plate to which the column load is distributed by sufficient gusset plates, angles, cleats, stiffeners, etc., and by direct bearing of the machined column end. The base plate must be capable, within the limits of the By-law stresses, of distributing this load further as may be necessary. An example to illustrate just the principle of this type of base is given in Fig. 88.

Slab or Bloom Base Plate.—The second type is known as a slab or bloom base, and it may consist of a relatively thick steel plate held in close contact with the machined end of the column from which it receives the load by direct bearing. In Fig. 89, which illustrates only the principle of this type of base, the small angle cleats which would be fixed to the web to secure the base plate have been omitted.

The working stress may be taken as 9 tons per sq. in. in such a base, and for uniform loading the thickness of the plate must not be less than

$$\sqrt{\frac{W(B-b)}{12D}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

or

$$\sqrt{\frac{W(D-d)}{12B}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

whichever is the greater. The symbols represent the dimensions shown in Fig. 89.

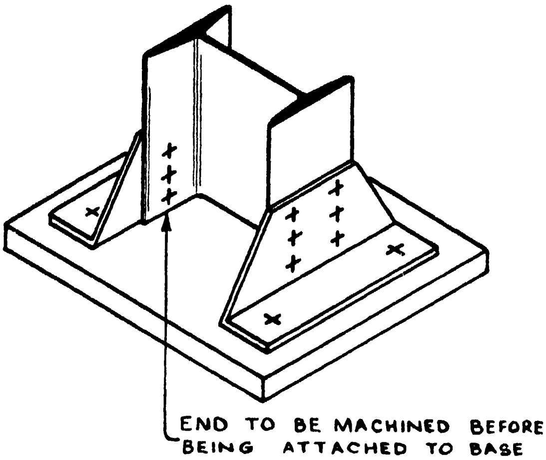


FIG. 88

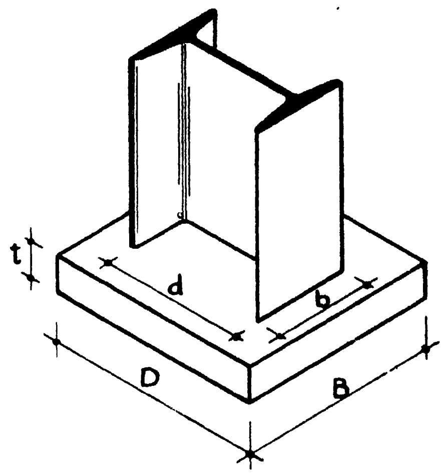


FIG. 89

In no case where there may be an unequal distribution of load under a base or where the base may not be rectangular may the stresses exceed those specified in the By-laws as the maximum working stresses.

For single-storey buildings the whole load may be transmitted by angles and gussets to the base plate, and in this case the end need not be machined.

By-law 70. Column Ends.—Except in the case of a solid round slab section, each end of every length of a column must be machined over the whole area after any cleats or gussets, etc., have been attached. Where joints must be made in a column the machined ends of adjacent lengths must be butted closely and if seating plates are used they must be in close contact with the column ends. The ends of columns must also be machined where caps are to be attached, and such caps must be attached in the same manner as for bases. See By-law 69.

In buildings of only one storey in which the column bases have sufficient gussets and rivets to transmit the whole load safely to the foundations, the column end or base need not be machined as required for other cases foregoing or by By-law 69.

By-law 71. Bearing Stresses and Widths of Packing Pieces between Column Lengths. —" The bearing stress in any steel packing or beam interposed between the ends of a superimposed column and a column beneath shall not exceed the permissible stress in the superimposed column, and the width across such interposed steel shall not be less than the corresponding width of the superimposed column."

Thus in Fig. 90, in which a column splice having an intermediate packing piece between the machined ends of the column lengths is shown, the bearing stress on the plate must not be greater than the permissible stress in the upper column length and its width must not be less than the width of the upper column.

Where, in accordance with the by-law, a beam of an R.S.J. or similar section is introduced between the column ends, it is advisable, whether calculations require it or not, to stiffen the web of the beam by angles or tees fitted to the flanges and so arranged that they can transmit a great part of the load coming from above directly to the column below.

By-law 72. Column Splices.—Joints or splices in steel columns must be made as near as possible to floor levels.

If the resultant stress due to all direct loads and to all bending moments is a compressive stress on every part of the section, the two portions may be connected by flange splice plates which may serve no other purpose than that of holding the pieces correctly in position; but such splice plates must, however, have a length, above and below the joint, of not less than the width of the wider flange nor less than 12 in., whichever is the greater.

If the resultant stress due to all direct loads and to all bending moments is a tensile stress at any part of the section, the two portions must be connected by splice plates which, with all their rivets and bolts, must be capable of resisting the whole of the tensile and compressive forces that are due to the bending moments. Although this by-law does not actually require it, the lengths of splice plates above and below the joint should not be less than the minimum lengths specified in the preceding paragraph. Compressive stresses due to direct loads may be transmitted by bearing through the machined ends.

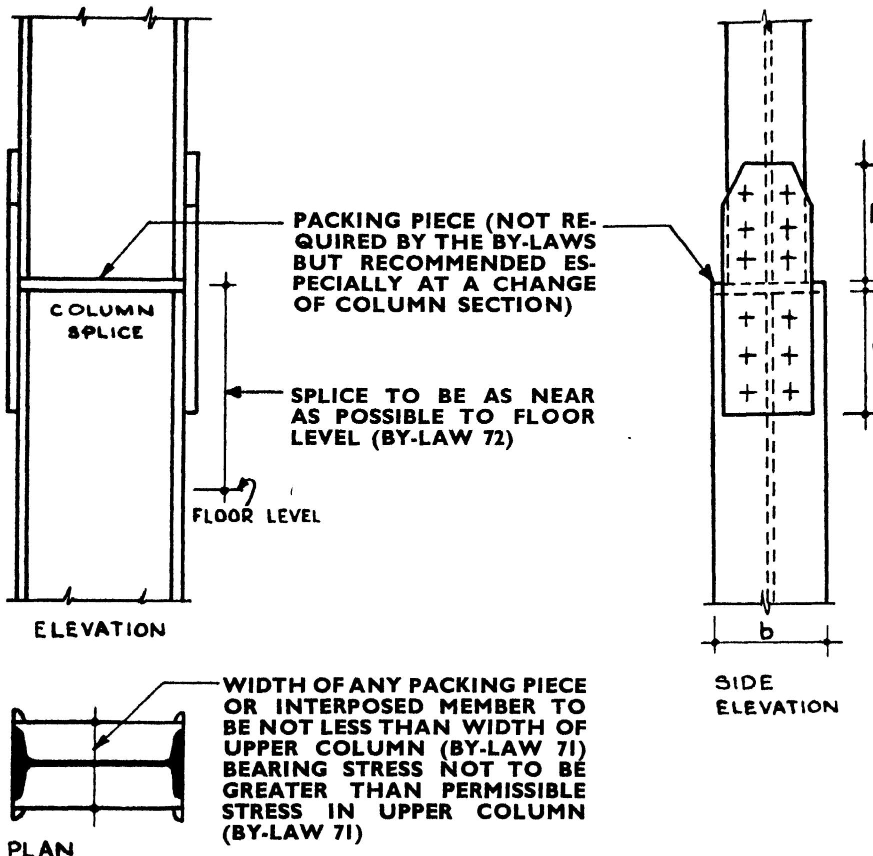


FIG. 90

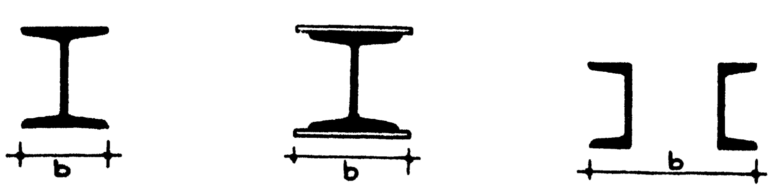


FIG. 91

L_1 must not be less than b
L_1 ,, ,, ,, ,, 12 in. } whichever is the greater;

L_2 must not be less than b
L_2 ,, ,, ,, ,, 12 in. } whichever is the greater;

and neither L_1 nor L_2 must be less than the length required by calculation to transmit the whole of the bending moment forces where there is any tensile stress on the section.

By-law 73. Caps and Bases to Solid Round Columns.—Caps and bases must be attached, by screwing or shrinking, on to machined shouldered ends of every solid round structural steel column. The whole area of the bearing faces of all such caps or bases must then be properly machined.

The shorter side of a rectangular base or cap, or the side of a square base or cap, or the diameter of a round base or cap must in no case be of less length than $1{\cdot}5d + 4{\cdot}5$ in., where d is the diameter in inches of the shouldered portion of the column.

The working stress in such a base or cap may be taken as 9 tons per sq. in.

The thickness of a rectangular base or cap with uniform loading (that is to say, with an axial column load) must not be less than

$$\sqrt{\frac{WD}{12(B-d)}} \text{ in.} \quad \ldots\ldots \quad (5)$$

where the symbols refer to the dimensions indicated in Fig. 92. The thickness of a similar square base or cap in which B = D must not be less than $\sqrt{\frac{WD}{12(D-d)}}$ in., where D is the length of a side of the square.

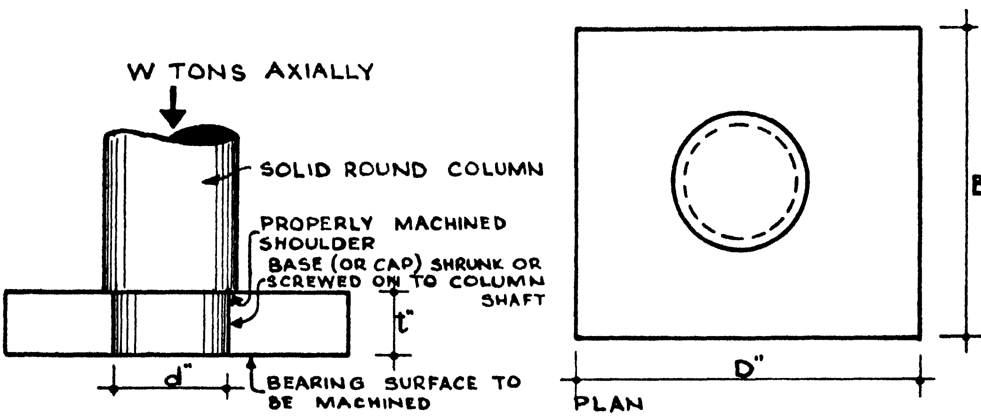


FIG. 92

For a rectangular cap or base:

$$t \text{ must not be less than } \sqrt{\frac{WD}{12(B-d)}}$$

$$\text{and B} \quad ,, \quad ,, \quad ,, \quad ,, \quad 1{\cdot}5d + 4{\cdot}5 \text{ in.}$$

For a square cap or base of side " D ":

$$t \text{ must not be less than } \sqrt{\frac{WD}{12(D-d)}}$$

$$\text{and D} \quad ,, \quad ,, \quad ,, \quad ,, \quad 1{\cdot}5d + 4{\cdot}5 \text{ in.}$$

By-law 74. Fabrication of Steelwork: Rivets; Turned Bolts; Welding.—" As much of the work of fabrication of all steelwork as is reasonably practicable shall be completed in the works where the steelwork is fabricated, and either rivets or turned bolts of driving fit shall be used for all such work, provided that black bolts may be used for the end cleat connections of secondary floor beams.

"Provided that, in any particular case, welding may be used if such welding is carried out in accordance with the conditions prescribed by the Council in that case."

An explanation of the Welding Regulations governing applications for any particular case is given at the end of this part (Part V).

By-law 75. Erection of Steelwork: Rivets; Turned Bolts; Black Bolts; Welding.—Only those connections which it is impracticable to make in the workshops may be made on the site. Rivets and turned bolts may be used in all cases without any additional seating stools, packing pieces, brackets, etc., which would give dead bearings. Black bolts may also be used without such dead-bearing seatings, but only for the ends of secondary floor beams and of roof trusses. It may be assumed that roof beams also would not require dead-bearing seatings when black bolts are used. In all other cases in which connections are made with black bolts, suitable dead bearings must be provided capable of resisting the whole of the loads.

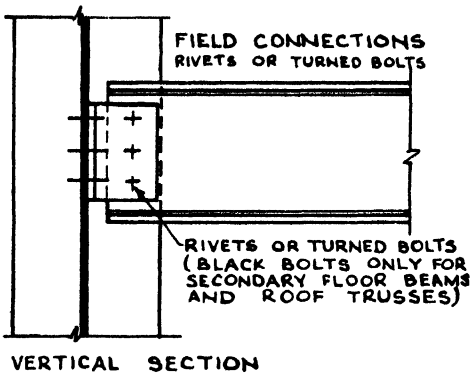


FIG. 93

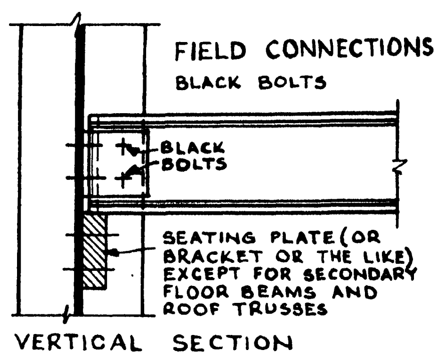


FIG. 94

In any particular case welding may be used for the erection of the steelwork if it is carried out in accordance with the conditions prescribed by the Council for that particular case.

An explanation of the Welding Regulations governing applications for any particular case is given at the end of this part (Part V).

By-law 76. Bolts.—" Bolts shall be provided with washers under the nuts of such thickness that the thread is clear of the hole and the shanks shall project at

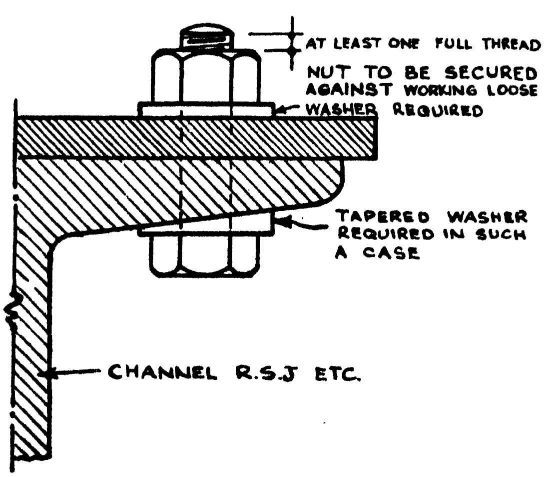


FIG. 95

least one full thread beyond the nuts and shall be so secured as to avoid the risk of their becoming loose. Washers shall be tapered where necessary to give the heads and nuts of bolts a true bearing. Bolts and nuts shall be in accordance with the British Standard Specifications Nos. 28—1932 (for black bolts) or 190—1924 (for turned bolts), except as regards the length of the threaded portion."

Where the steelwork is to be encased with incombustible materials in accordance with By-law 68, the casing, being in close contact with the steel, may normally be relied upon to prevent the working loose of nuts. In other cases the nuts may be secured by such means as the burring over of the top of the stem of the bolt, or by a special lock nut, in which case the length of the screwed portion of the bolt must be greater.

By-law 77. Spacing of Rivets and Bolts.—In this and in subsequent by-laws the diameter of a rivet is to be taken as the diameter of the hole in which it will be driven.

No rivet or bolt hole must, at any point, be nearer to the edge of any section than the diameter of the rivet or bolt. Rivets must in no case be nearer, centre to centre, than three times the diameter of the rivets. That is to say, the width of solid material between rivets must not be less than twice the rivet diameter. If the rivets are of different diameters the average diameter should be taken for the above purpose. Although the by-law does not actually specify it, the requirements of this paragraph should apply equally to bolts.

Except for tacking rivets (i.e. rivets which do not transfer any calculated stresses) and except for two rows of staggered rivets in one flange of a single angle, the straight-line pitch of rivets in any member must not exceed

sixteen times the thickness of the thinnest outside section,
nor 8 in. for tension members,
nor 6 in. for compression members.

In Figs. 96 and 97 :

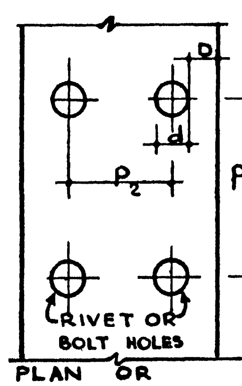


FIG. 96

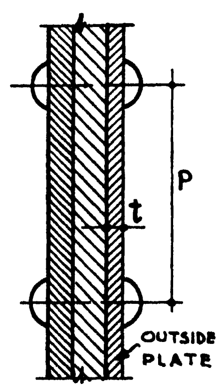


FIG. 97

D must not be less than d
P_2 ,, ,, ,, ,, $3d$
P ,, ,, ,, ,, $3d$

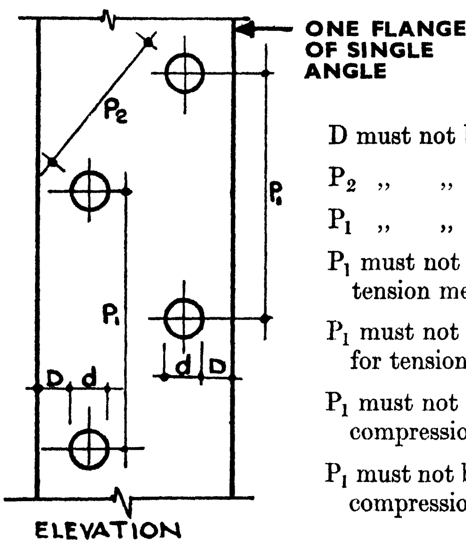
P must not be more than $16t$ for tension members
P must not be more than 8 in. for tension members } whichever is less.

P must not be more than $16t$ for compression members
P must not be more than 6 in. for compression members } whichever is less.

When there are two rows of staggered rivets in one flange of a single angle the straight line pitch must not exceed

twenty-four times the thickness of the thinnest outside section,
nor 12 in. for tension members,
nor 9 in. for compression members.

In Fig. 98 :



D must not be less than d

P_2 ,, ,, ,, ,, $3d$

P_1 ,, ,, ,, ,, $3d$

P_1 must not be more than $24t$ for tension members
P_1 must not be more than 12 in. for tension members
} whichever is less.

P_1 must not be more than $24t$ for compression members
P_1 must not be more than 9 in. for compression members
} whichever is less.

FIG. 98

The maximum pitch of tacking rivets is given in By-law 79.

By-law 78. Unsupported Flange Width.—When there are more flange plates than one on any section, the distance from the edge of the plates to the nearest line of rivets, by which the plates are ultimately attached to the web, must never be greater than twelve times the thickness of the thinner outside plate. Tacking rivets must always be introduced when this distance exceeds nine times the thickness of the thinner outside plate. When there is only one flange plate the distance must not be more than nine times the thickness.

The minimum pitch of tacking rivets is given in By-law 79.

FIG. 99

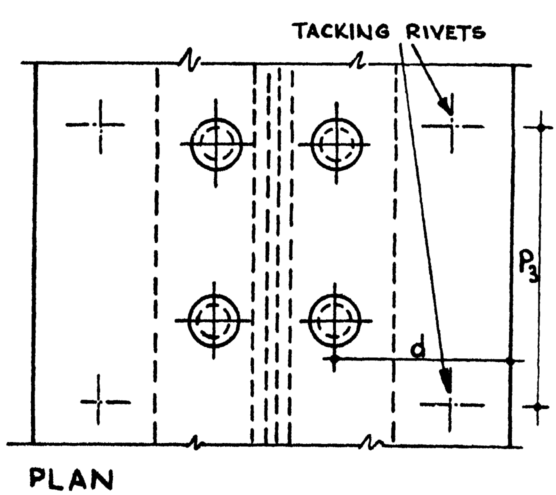
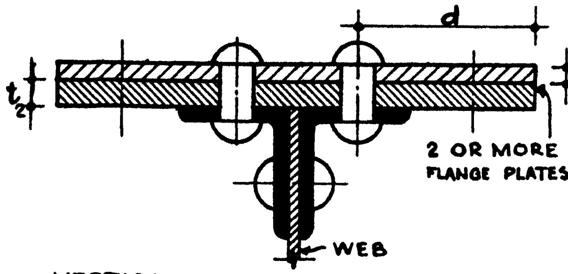


FIG. 99 (*contd.*)

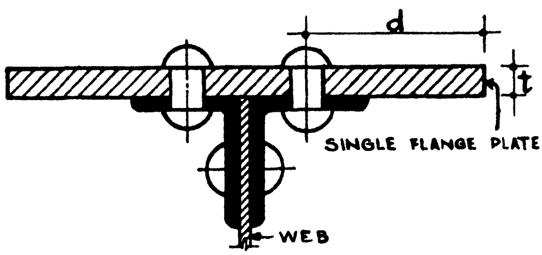


VERTICAL SECTION

P_3 must not be greater than $24t_1$
P_3 ,, ,, ,, ,, $24t_2$ } whichever is the least (By-law 79).
P_3 ,, ,, ,, ,, 12 in.

d must not be greater than $9t_1$ when tacking rivets are not used (By-law 78).
d must not be greater than $12t_1$ when tacking rivets are used (By-law 78).

FIG. 100



VERTICAL SECTION

d must not be greater than $9t$ (By-law 78)

This by-law applies directly only to flanges; but logically its requirements should apply also to any section composed of two or more thicknesses of material.

By-law 79. Spacing of Tacking Rivets.—The pitch of tacking rivets, i.e. rivets which connect flange plates together but which transmit no calculated stress, must not be greater than 24 times the thickness of the thinner outside plate nor greater than 12 in., whichever is the less. Such rivets are subject also to the minima spacings specified in By-law 77 but exempt from the maxima spacings given in that by-law, which maxima are specified here (By-law 79). See Fig. 99.

By-law 80. Thickness of Plates of Steel Columns.—Every flange and web plate in a column must be not less than $\frac{3}{8}$ in. in thickness. This by-law does not apply to the common rolled sections such as joists, etc., but only to plates from which a section may be fabricated.

By-law 81. Permissible Working Stresses.—The working stresses for structural steel, used in different ways, are specified in this by-law, and unless modifications of these stresses can be made, under the conditions of By-laws 69, 73, 82, 83, 85, 87 and 90, the stresses in the steel, as actually calculated from the loads, must not exceed the values given.

The cases to which these stresses do not apply or for which they may be modified are:

(1) Plates forming bloom bases or caps (to columns), for which By-laws 69 and 73 permit working stresses of 9 tons per sq. in.

(2) Grillage beams in accordance with By-law 82, which permits a 50 per cent. increase on the working stresses given in this by-law (By-law 81).

(3) Filler floor beams in accordance with By-law 83, which permits a working stress of 9 tons per sq. in.

(4) Columns or other struts in which the tendency to buckle calls for a reduction of stress depending on the slenderness ratio. See By-law 85.

(5) Columns with combined direct and bending stresses in which the working stress is dependent on the proportions of each of these individual stresses. See By-law 87.

(6) All steelwork, except that given for cases (2) and (3) above, when subjected to increased loads due to wind only. In this case in accordance with By-law 90 the working stresses may be increased by 33⅓ per cent. except for the two instances mentioned.

Otherwise the calculated stress in any structural steel unit must not be greater than the permissible stress for that unit given in Table 20.

Cases are frequently met in which it is not the total area that is to be taken for the calculated stress but this total area modified by certain reductions. The by-laws refer to such modified areas as "net sections," whereas the total areas are called "gross sections." To avoid any misunderstandings which might arise, due to the rather loose wording, the exact meaning is indicated in each case.

TABLE 20

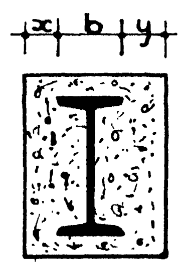
(i) PERMITTED STRESSES IN STRUCTURAL STEEL

The references are to the notes at the end of the table (pp. 96 and 97)

		Structural Unit (see Note 2 below)	Permissible stress tons per sq. in.
a	Tension	On the net section of any axially loaded tension member (see Note 3).	8
b	Tension flanges	In the extreme fibres of the net section of the tension flange of any beam (see Note 4).	8
c	Compression flanges (encased)	In the extreme fibres of the gross section of the compression flange of a beam embedded in a concrete floor or otherwise secured laterally (no deduction for holes).	8
d	Compression flanges (uncased)	In the extreme fibres of the gross section of the compression flange of an uncased beam if the laterally unsupported length is not greater than twenty times the width of the compression flange (no deduction for holes).	8
e	Compression flanges (uncased)	In the extreme fibres of the gross section of the compression flange of an uncased beam in which the laterally unsupported length L is greater than twenty times, but not greater than fifty times the width *b* of the compression flange (no deduction for holes). Stresses are given in Table 21 for values of $\frac{L}{b}$ from twenty to fifty, which latter must never be exceeded in any beam.	$11{\cdot}0-0{\cdot}15\frac{L}{b}$

TABLE 20—*continued.*

(i) PERMITTED STRESSES IN STRUCTURAL STEEL—*continued.*

		Structural Unit (see Note 2 below)	Permissible stress tons per sq. in.
f	Compression flanges, encased but inadequately supported laterally	In the extreme fibres of the gross section of the compression flange of a beam other than of type *c* encased in concrete in accordance with By-law 68 in which the laterally unsupported length L is greater than twenty times, but not greater than fifty times the width b_1, which is the width b of the compression flange plus the smaller width (not exceeding 4 in.) of concrete casing beyond either edge of the compression flange (no deduction for holes). In Fig. 101: $b_1=b+x$ or $b+y$, whichever is the less. x must not be taken as being more than 4 in. y must not be taken as being more than 4 in. Stresses are given in Table 21 for values of $\frac{L}{b}$ from 20 to 50, which latter must never be exceeded in any beam.  FIG. 101	$11{\cdot}0 - 0{\cdot}15\frac{L}{b_1}$
g	Shear in webs	On the gross section of webs (see Note 5) (see Note 1 below regarding stiffening of the webs of beams).	5
h	Bearing on plates, etc.	On packings, seatings, etc. (see Note 1 below regarding stiffening of webs of beams).	12

(ii) PERMISSIBLE STRESSES IN RIVETS AND BOLTS.

i	Shop Rivets: Tension	On the net section of a shop driven rivet for an axial tensile load (see Note 6).	5
	Single Shear	On the section for a single shear load (see Note 7).	6
	Bearing	On the area given by the diameter of a shop-driven rivet and the thickness of the thinnest section on which the rivet will bear (see Note 10).	12
j	Field Rivets: Tension	On the net section of a field rivet for an axial tensile load. (The parts to be joined must be in close contact before the rivet is driven and the rivet must be driven hot, snap-headed and of the proportions required by the British Standard Specification No. 275—1927) (see Note 6).	4
	Single shear	On the section of a field rivet (see Note 7).	5
	Bearing	On the area given by the diameter of the field rivet and the thickness of the thinnest section on which the rivet will bear (see Note 10).	10

TABLE 20—*continued.*

(ii) PERMISSIBLE STRESSES IN RIVETS AND BOLTS—*continued.*

		Rivets and bolts	Permissible stress tons per sq. in.
k	Turned and fitted bolts: Tension	On the net section of a turned bolt of a driving fit for an axial tensile load. (All such bolts in tension must be not less than ⅝ in. in diameter and of the proportions required by the British Standard Specification No. 190—1924 except as regards the length of the threaded portion. All the parts to be connected must be in close contact before the bolts are tightened) (see Note 8).	5
	Single shear	On the net section of a turned bolt of driving fit for a single shear load (see Note 9).	6
	Bearing	On the area given by the diameter of a turned bolt of driving fit and the thickness of the thinnest section on which the bolt will bear (see Note 11).	12
l	Black bolts: Tension	On the net section of a black bolt for an axial tensile load. (All such bolts in tension must not be less than ⅝ in. in diameter and of the proportions required by the British Standard Specification No. 28—1932 except as regards the length of the threaded portion. All the parts to be connected must be in close contact before the bolts are tightened (see Note 8).	5
	Single shear	On the section of black bolt (see Note 9).	4
	Bearing	On the area given by the diameter of a black bolt and the thickness of the thinnest section on which the bolt will bear (see Note 11).	8
m	All rivets and bolts in double shear	On the net section of a shop-driven rivet, of a field rivet of a turned bolt of driving fit or of a black bolt for a double shear load.	Twice the single shear value given for the appropriate case, in items *i*, *j*, *k*, or *l* above.

Note 1: The webs of beams must be stiff enough or be provided with stiffeners to prevent buckling. Formulæ and notes on web buckling are given in Chapter 28, Volume 3.

Note 2: The permissible stresses for columns are given under By-laws 85 and 87.

Note 3: The areas of all rivet or bolt holes which occur in any one section at right angles to the direction of the tensile load are to be deducted from the gross section of the member. Where rivets or bolts are staggered, as for example in Fig. 98, only those holes which occur in the same cross section at right angles to the load are to be deducted provided that the distance measured in the direction of the load between the centres of holes in adjacent sections is not less than 1·5 times the diameter of the larger holes. If this condition is not satisfied it is considered that the combined areas of the holes in the adjacent sections should be deducted.

Note 4: It is not usual, so far as tensile stresses are concerned, to deduct the areas of holes in the portion of the web lying on the tensile side of the neutral axis of a beam, because such holes have very little influence on the resistance of the section against bending. For staggered rivets or bolts see Note 3. When it is necessary to make a reduction for holes in the tensile flange of a beam, a similar reduction is usually assumed for the compressive flange. When the flanges of the beam are symmetrical this method simplifies the calculations because then the symmetry remains. There is, however, no necessity to make this assumption and if advantage is to be taken of the full effective section a calculation of the properties of the section can be

made taking the gross compressive flange area into account. Examples are given in Chapter 28, Volume 3.

Note 5: In this case the word "gross" does not mean that holes need not be deducted. In fact, all holes occurring in a section under shear must be deducted and the stress must be computed on the remaining area. The intention in this case is that the web may be regarded as continuing through the flanges to their outer faces. This, of course, applies only to joists or channels, etc., in which the web forms part of the same rolled section. It should not be applied to a plated flange for which case an exact calculation should be made. In other words, the effective web area for the purpose of shear resistance is to be taken as the area given by the thickness multiplied by the overall depth of the web (as defined above) minus the areas of any holes occurring in the cross section of the web.

The method of calculating the shear stress by dividing the shear load by this area is only an approximation and it gives results which are about 10 per cent. too low. It might be assumed, therefore, when the stress is calculated by more exact methods (Chapter 28, Volume 3) or when the shear being investigated is not in the web of a beam, that a stress of 5·5 tons per sq. in. would be permitted instead of 5 tons per sq. in.

Note 6: In By-law 77 it is pointed out that the diameter of a rivet is to be taken as the diameter of the hole into which it is to be driven. This can be taken as the dimension determining the gross areas for the purpose of shear and bearing stresses. This by-law (By-law 81), however, specifies that the tensile stress is to be calculated on the net area of a rivet and it must be assumed, therefore, as the shank of the rivet near the head is restrained from expanding and filling the hole, that "net area" in this case means the diameter of the rivet before it is driven and not the diameter of the hole.

The authors support the principle of avoiding the use of rivets to resist direct tensile loads. Due to the contraction of a rivet on cooling there are generally unknown stresses set up in it even before the application of any external loading. The fact that rivet heads sometimes fly off indicates that these stresses are very high. If it is at all possible the arrangement of the detail of a joint should be such that no direct tensile loads are transmitted through rivets. Either bolts should be used or the joint arranged in a different manner. See Table 20 (*a*).

Note 7: Under By-law 77 the diameter of the rivet is to be taken as the diameter of the hole in which it is driven. See Table 20 (*a*).

Note 8: The net section in this case should be taken as that of the cross section of the bolt measured at the root of the threads. See Table 20 (*a*).

Note 9: The net section in this case should be taken as that of the actual cross section of the unscrewed portion of the stem. See Table 20 (*a*).

Note 10: The bearing area in this case is the rectangular area given by the diameter of the hole multiplied by the thickness of the thinnest section on which the rivet will bear. See Table 20 (*a*).

Note 11: The bearing area in this case is the rectangular area given by the actual diameter of the unscrewed portion of the stem multiplied by the thickness of the thinnest section on which the bolt will bear. See Table 20 (*a*).

In the Council's publication "Construction of Buildings in London," tables of the properties of British Standard rolled steel sections are given and in Table 20 (*a*) which follows, the loads in various bolts and rivets based on the stresses in Table 20 are set out.

TABLE 20 (*a*)

TENSILE, SHEAR, AND BEARING LOADS OF BOLTS AND RIVETS

a=shop rivets and turned bolts. *b*=field rivets. *c*=black bolts.
d=all bolts. *e*=shop rivets. *f*=field rivets.

Diameter of bolt or rivet	Type	Load in Tons							
		Single Shear	Tension	Bearing on Plates of Thicknesses of :—					
				$\frac{3}{8}$ in.	$\frac{1}{2}$ in.	$\frac{5}{8}$ in.	$\frac{3}{4}$ in.	$\frac{7}{8}$ in.	1 in.
$\frac{1}{2}$ in.	*a*	1·178	—	2·23	3·0	3·75	4·50	5·25	6·0
	b	0·981	—	1·87	2·5	3·12	3·75	4·37	5·0
	c	0·785	—	1·50	2·0	2·50	3·00	3·50	4·0
	d	—	0·607						
	e	—	0·883						
	f	—	0·707						

TABLE 20 (*a*)—*continued*

TENSILE, SHEAR, AND BEARING LOADS OF BOLTS AND RIVETS—*continued*

a=shop rivets and turned bolts. *b*=field rivets. *c*=black bolts.
d=all bolts. *e*=shop rivets. *f*=field rivets.

Diameter of bolt or rivet	Type	Load in Tons							
		Single Shear	Tension	Bearing on Plates of Thicknesses of:—					
				3/8 in.	1/2 in.	5/8 in.	3/4 in.	7/8 in.	1 in.
5/8 in.	*a*	1·840	—	2·82	3·75	4·68	5·62	6·56	7·50
	b	1·533	—	2·34	3·12	3·90	4·68	5·46	6·25
	c	1·227	—	1·87	2·50	3·12	3·75	4·37	5·00
	d	—	1·016						
	e	—	1·380						
	f	—	1·104						
3/4 in.	*a*	2·650	—	3·37	4·50	5·62	6·75	7·87	9·00
	b	2·208	—	2·82	3·75	4·68	5·62	6·56	7·50
	c	1·767	—	2·25	3·00	3·75	4·50	5·25	6·00
	d	—	1·519						
	e	—	1·987						
	f	—	1·590						
7/8 in.	*a*	3·607	—	3·94	5·25	6·56	7·87	9·18	10·50
	b	3·005	—	3·28	4·37	5·46	6·55	7·65	8·75
	c	2·404	—	2·63	3·50	4·37	5·25	6·12	7·00
	d	—	2·108						
	e	—	2·706						
	f	—	2·165						
1 in.	*a*	4·712	—	4·50	6·00	7·50	9·00	10·50	12·00
	b	3·927	—	3·75	5·00	6·25	7·50	8·75	10·00
	c	3·141	—	3·00	4·00	5·00	6·00	7·00	8·00
	d	—	2·770						
	e	—	3·534						
	f	—	2·827						

TABLE 21

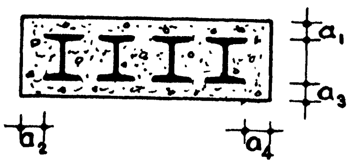
$\frac{L}{b}$ Case *e*, Table 20. $\frac{L}{b_1}$ Case *f*, Table 20.	Permissible Stress. Tons per sq. in.	$\frac{L}{b}$ Case *e*, Table 20. $\frac{L}{b_1}$ Case *f*, Table 20.	Permissible Stress. Tons per sq. in.
20	8·00	35	5·75
21	7·85	36	5·60
22	7·70	37	5·45
23	7·55	38	5·30
24	7·40	39	5·15
25	7·25	40	5·00
26	7·10	41	4·85
27	6·95	42	4·70
28	6·80	43	4·55
29	6·65	44	4·40
30	6·50	45	4·25
31	6·35	46	4·10
32	6·20	47	3·95
33	6·05	48	3·80
34	5·90	49	3·65
35	5·75	50	3·50

Table 21 gives the permissible extreme fibre stress in the top flanges of uncased and encased beams for values of $\frac{L}{b}$ and $\frac{L}{b_1}$ in accordance with cases *e* and *f* in Table 20.

The value of $\frac{L}{b}$ or of $\frac{L}{b_1}$ must never be greater than 50 for any beam except for beams of case *c* in Table 20, that is to say except for beams encased in a concrete floor or otherwise secured laterally.

By-law 82. Working Stresses in Grillage Beams.—For structural steel grillage beams the working stresses given in By-law 81 may be increased by 50 per cent. if the beams are entirely encased in concrete in such a way that:

In Fig. 102:



a_1 must not be less than 3 in.
a_2 ,, ,, ,, ,, 3 in.
a_3 ,, ,, ,, ,, 3 in.
a_4 ,, ,, ,, ,, 3 in.

FIG. 102

(*a*) the concrete is not inferior in quality or composition to concrete IV in By-law 14 (i.e. 1 : 6 plain concrete of crushing strength not less than 1480 lb. per sq. in. at an age of 28 days).

(*b*) All voids are filled.

(*c*) A solid block is formed of constant horizontal section throughout the full depth.

And

(*d*) The cover to the steel beams is nowhere less than 3 in.

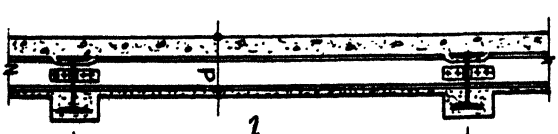
By-law 83. Filler Floor Joists.—"The strength of filler floor beams of structural steel entirely encased in a concrete floor slab may be estimated on the basis of the combined moment of inertia of the steel and surrounding concrete calculated as in reinforced concrete, neglecting the strength of concrete in tension and taking the limit of flexural stress in the steel at 9 tons per sq. in."

Filler joists, encased in concrete, represent actually a type of reinforced concrete, and they can be used economically only if they are designed to work in co-operation with the concrete. This design is investigated in Chapter 28, in which tables have been derived and set out giving the carrying capacities as functions of the joist sizes and spacings.

There is no reference to the thickness of cover which is required, but from the reinforced concrete by-laws it can be inferred that the inch above and below the joists would be accepted. Tables in Chapter 28 show that the top cover can economically be increased.*

By-law 84. Span of Beams. *Filler Floor Beams.*—The span of any filler floor beam encased in concrete must not be greater than thirty-two times the depth from the top of the concrete to the soffit of the bottom flange. See Fig. 103. Conversely the depth must not be less than one-thirty-second part of the span.

In Fig. 103 for filler floor beams:



l must not be greater than $32d$

d must not be less than $\frac{l}{32}$

FIG. 103

* For filler joists in conjunction with hollow blocks, see p. 233.

Any Other Beam.—Only if the calculated deflection of any beam (other than a filler floor beam entirely encased in concrete) is less than one three-hundred-and-twenty-fifth part of the span may that span exceed twenty-four times the depth of the beam.

In Fig. 104 for any other beam :

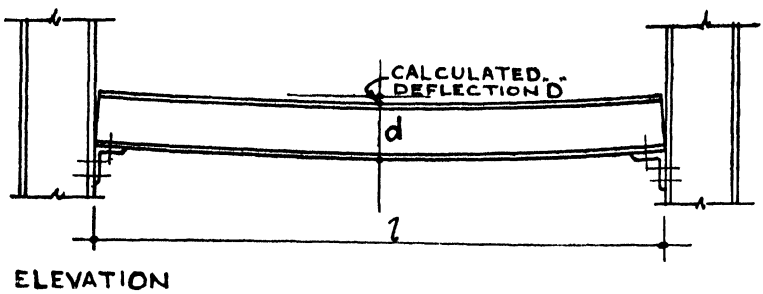


FIG. 104

$$\left.\begin{array}{l} l \text{ must not be greater than } 24d \\ \text{or } d \text{ must not be less than } \dfrac{l}{24} \end{array}\right\} \text{unless D is less than } \frac{l}{325}$$

By-law 85. Permissible Stresses in Columns and Struts for Axial Loads.—The permissible stress in a column or strut is to be determined by the highest slenderness ratio of that column or strut. Briefly, the slenderness ratio is the ratio of the effective length of the column or strut to its radius of gyration and the manner in which the effective length is to be found is explained under By-law 86, where it is indicated also which radius of gyration is to be adopted.

The slenderness ratio must not be greater than 150 for any column or strut which is part of the main structure of a building, nor more than 200 for any subsidiary compression member. Thus any steel column or strut, whether it is in a complete structural steel framework or not, which carries directly any dead or live loads of the building must not have a slenderness ratio greater than 150 ; but any subsidiary column or strut which is given compression loads indirectly, as for instance a strut in a lattice girder or roof truss, may have a slenderness ratio up to but not exceeding 200.

In any column or strut the calculated compression stress must not exceed that given in Table 22 for the appropriate slenderness ratio.

For main members the value must be to the left of or above the dotted line.

For subsidiary members the value may lie anywhere within the table. Intermediate values for any case may be attained by interpolation.

These values must not be exceeded except when

(*a*) there is in addition to the direct load a bending moment in which case the permissible stress will be determined by By-law 87 ; or

(*b*) when the steelwork is subjected to increased loads due to wind only. In this case in accordance with By-law 90 the working stresses may be increased by $33\frac{1}{3}$ per cent.

As no guidance is given in the by-laws as to how to compute the slenderness ratio of a latticed or battened section, explanations and calculations are given in Chapter 29 where reference is made also to the sizes of the battens and the rigidities of their connections.

TABLE 22

PERMISSIBLE STRESSES F_1 IN COLUMNS AND STRUTS FOR AXIAL LOADS

Ratio (l/r) of Effective Length to Least Radius of Gyration	Working Stress in tons per sq. in. of Gross Section $=F_1$	Ratio (l/r) of Effective Length to Least Radius of Gyration	Working Stress in tons, per sq. in. of Gross Section $=F_1$	Ratio (l/r) of Effective Length to Least Radius of Gyration	Working Stress in tons per sq. in. of Gross Section $=F_1$
20	7·20	82	4·78	144	2·18
22	7·14	84	4·66	146	2·12
24	7·08	86	4·54	148	2·06
26	7·02	88	4·42	150	2·00
28	6·96	90	4·30	152	1·96
30	6·90	92	4·20	154	1·92
32	6·84	94	4·10	156	1·88
34	6·78	96	4·00	158	1·84
36	6·72	98	3·90	160	1·80
38	6·66	100	3·80	162	1·76
40	6·60	102	3·70	164	1·72
42	6·54	104	3·60	166	1·68
44	6·48	106	3·50	168	1·64
46	6·42	108	3·40	170	1·60
48	6·36	110	3·30	172	1·58
50	6·30	112	3·22	174	1·56
52	6·22	114	3·14	176	1·54
54	6·14	116	3·06	178	1·52
56	6·06	118	2·98	180	1·50
58	5·98	120	2·90	182	1·46
60	5·90	122	2·84	184	1·42
62	5·80	124	2·78	186	1·38
64	5·70	126	2·72	188	1·34
66	5·60	128	2·66	190	1·30
68	5·50	130	2·60	192	1·28
70	5·40	132	2·54	194	1·26
72	5·30	134	2·48	196	1·24
74	5·20	136	2·42	198	1·22
76	5·10	138	2·36	200	1·20
78	5·00	140	2·30	—	—
80	4·90	142	2·24	—	—

l/r must not be greater than 150 for main structural members.

l/r ,, ,, ,, 200 for subsidiary members.

By-law 86. Effective Lengths of Columns and Struts.—Six conditions for end fixing and continuity of columns are given in this by-law, and the proportion of the actual column length which must be taken for each of these conditions and which must be regarded as the effective length for the purposes of calculating the slenderness ratio are given. The by-law points out that the values given are only for the few typical cases and that they are based on the general principles on which the effective length must be ascertained to the district surveyor's satisfaction for the actual end conditions in any particular case.

There is no further indication in the by-laws as to how such intermediate effective lengths are to be determined; but in Chapter 15 the authors have given certain rules by which the amounts of positional and rotational restraints may be computed. Tables 52 to 54 have also been compiled, setting down the effective lengths of columns in terms of the actual lengths for these different proportions of end restraints. The values in those tables are based on the assumption that the actual connections between beams and columns are themselves rigid. However, in the present stage of knowledge on the behaviour of such joints in steelwork, there can be no truly scientific approach to the problem. At present for the purposes of the design of the column it is the custom to regard the joints as hinged and to consider bending moments that are due only to the eccentricities of the reactions of the beams on the column stools or cleats. In actual fact there is, in even the simplest type of connection, a certain amount of rigidity, which, no matter how small it may be, is increased by the steelwork casing that is required by the by-laws. In order to come to some result, however, the authors suggest, when it is clear that a joint is not rigid and that therefore the method recommended in Chapter 15 cannot be applied, that the following rules should be observed in determining the " rotational " or " directional " restraint (see Fig. 104 (*a*)) :—

If d_1 represents the distance between the top and bottom bolts or rivets of a beam-to-column connection on one side of the column, and if d_2 represents the corresponding distance of the opposite side, and if d is the width of the column between the ends of the beam, then " proper " restraint may be assumed only if $d_1 + d_2$ is not less than $4d$.

If there is a beam on only one side then proper restraint may be assumed if d_1 (or d_2) is not less than $4d$.

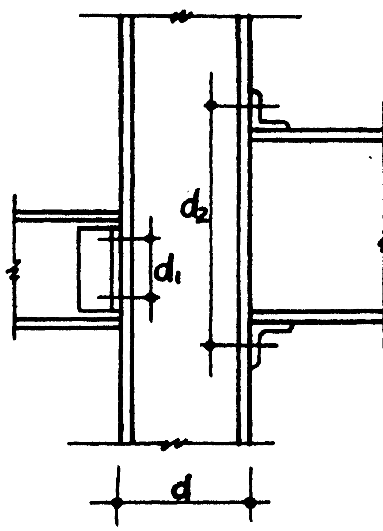


FIG. 104 (*a*)

A measure of the " efficiency of the restraint " for the purpose of Tables 23 and 24 can be taken as $\frac{d_1 + d_2}{4d}$ (with a maximum value of unity) for beams on two opposite sides of a column. Or $\frac{d_1 \text{ (or } d_2)}{4d}$ (with a maximum value of unity) for a beam on only one side of a column.

As far as the positional restraint is concerned, it can be assumed that the simplest connection between a beam (which itself is restrained in some manner) and a column is sufficient to impose full positional restraint.

By-law 86 actually specifies the effective lengths, given in Tables 23 and 24 (the illustrations, which indicate purely diagrammatically the end conditions intended for each case and the index letters and references to other parts of this volume have been introduced by the authors).

Table 23

COLUMNS OF ONE STOREY

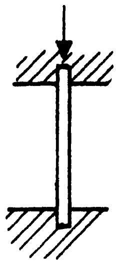
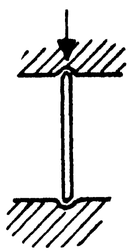
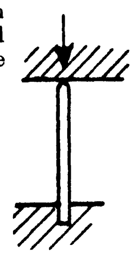
	Type of Column	Effective Column Length
A	Properly restrained at both ends in position and direction  Fig. 105	0·75 of the actual column length
B	Properly restrained at both ends in position but not in direction  Fig. 106	Actual column length
C	Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end  Fig. 107	A value intermediate between the actual column length and twice that length, depending upon the efficiency of the imperfect restraint

Table 24

COLUMNS CONTINUING THROUGH TWO OR MORE STOREYS

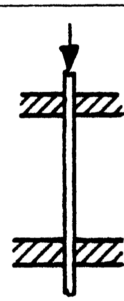
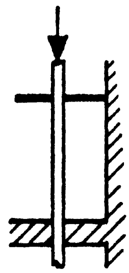
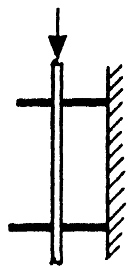
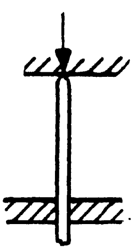
	Type of Column	Effective Column Length
D	Properly restrained at both ends in position and direction  Fig. 108	0·75 of the distance from floor level to floor level

TABLE 24—*continued*

COLUMNS CONTINUING THROUGH TWO OR MORE STOREYS—*continued*

	Type of Column	Effective Column Length
E	Properly restrained at both ends in position and imperfectly restrained in direction at one or both ends  FIG. 109  FIG. 110	A value intermediate between 0·75 and 1·00 of the distance from floor level to floor (or roof) level, depending upon the efficiency of the directional restraint
F	Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end  FIG. 111	A value intermediate between the distance from floor level to floor (or roof) level and twice that distance, depending upon the efficiency of the imperfect restraint.

The correct effective length having been ascertained in any case, it is usually necessary only to take the least radius of gyration of the section to obtain the slenderness ratio from which the permissible column stress may be found under By-law 85. Cases frequently occur, however, in which a column is braced laterally either for its full height in one direction or at certain points in that direction. Thus in Fig. 112, if l is the effective length of a column unsupported in one direction and if in the other direction there is an intermediate lateral support which is so attached that the effective lengths of the column above and below are l_1 and l_2 respectively, then the value of whichever of $\frac{l}{r_x}$, $\frac{l_1}{r_y}$, $\frac{l_2}{r_y}$ is the greatest must be taken as the slenderness ratio by which the permissible column stress will be determined. In these three expressions for the slenderness ratios the symbols r_x and r_y refer to the radii of gyration of the column section about its xx and yy axes respectively.

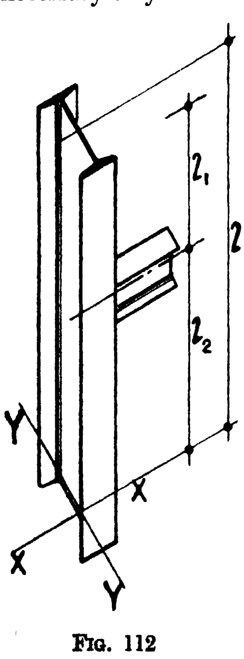


FIG. 112

By-law 87. Combined Direct and Bending Stresses in a Column.—Where a column is subjected to an eccentric load the maximum bending moment stress and the direct compression stress must be added together and the result must not be greater than F_2 in the formula

$$F_2 = fc + 7{\cdot}5\left(1 - \frac{fc}{F_1}\right)\left(1 - 0{\cdot}002\frac{l}{r}\right) \text{ tons per sq. in.} \quad (6)$$

in which fc = the direct stress ; or the vertical load divided by the cross-section of the column,

and F_1 = the permissible column stress for an axial load as determined from By-law 85 for the appropriate slenderness ratio $\frac{l}{r}$.

If the load is eccentric to the two main axes of the column the maximum bending moment stress must be found by adding the bending moment stresses for the individual eccentricities about each axis. It will be seen from the foregoing remarks that the increased permissible stress F_2 is specifically allowed by this by-law only for columns with eccentric loads. But a column may, in fact, have a central vertical load yet still be subjected to bending moment stresses, as for instance a central column of a rigid frame which is under a lateral wind load. To such cases it is considered that this by-law would also apply.

The formula for F_2 is rather cumbersome to apply, and in order to save the designer's time the graph reproduced as Fig. 113 has been devised. Values of the permitted combined stress may be found by interpolation for intermediate values of fc and $\frac{l}{r}$; but to illustrate the use of the graph a simple case will be chosen in which $fc = 4{\cdot}0$ and $\frac{l}{r} = 85$.

From the formula (6) : $F_2 = 4{\cdot}0 + 7{\cdot}5\left(1 - \frac{4{\cdot}0}{F_1}\right)\left(1 - 0{\cdot}002 \times 85\right)$

F_1 from Table 22 = $4{\cdot}6$.

$\therefore\ F_2 = 4{\cdot}0 + 7{\cdot}5 \times 0{\cdot}13 \times 0{\cdot}83 = 4{\cdot}0 + 0{\cdot}81 = 4{\cdot}81$.

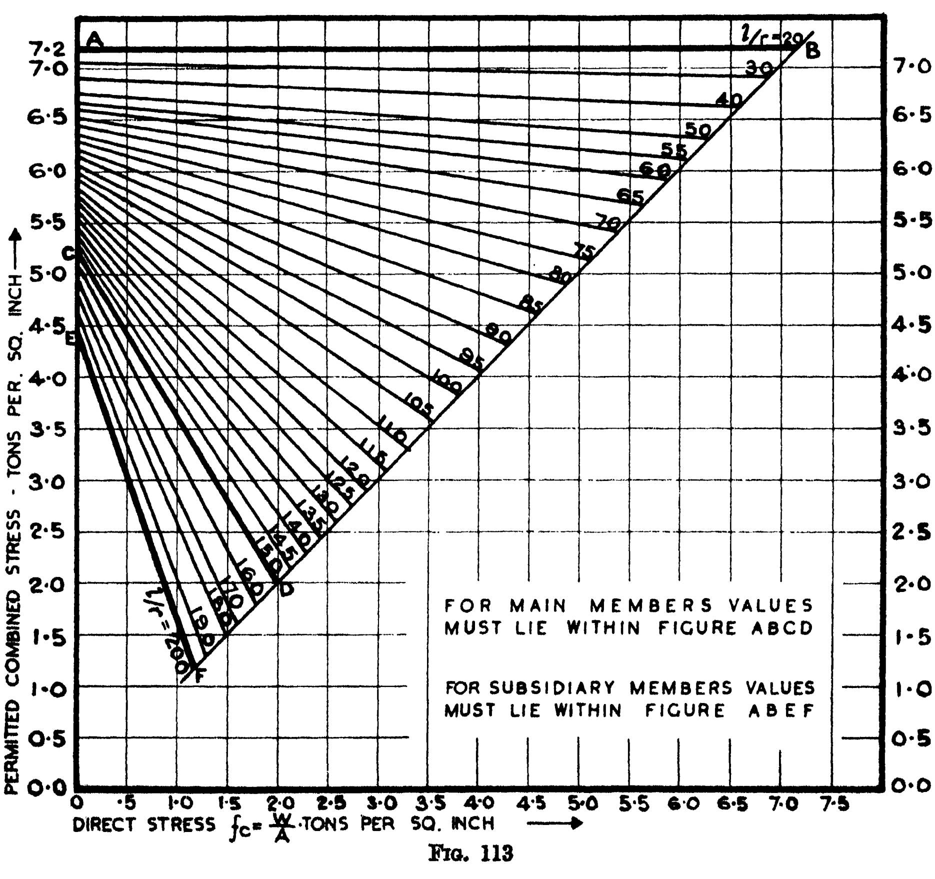


FIG. 113

Alternatively by using the graph :—

In Fig. 113 follow up the vertical line through $fc = 4{\cdot}0$, until it intersects the $\frac{l}{r} = 85$ sloping line. Then follow the level of this point of intersection to the left or right and read off in either margin the permissible combined stress 4·8. Points below the line EF would give values of $\frac{l}{r}$ greater than those permitted by By-law 85, and points below the line BF would give values of F_1 greater than those permitted.

By-laws 88 and 89. Bending Moments in Columns due to Eccentric Loads and Dissipation of such Bending Moments.—If a beam is connected to a column continuing up past the connection to another lateral support at a higher level, and if the connection is such that an eccentric load is applied to the column, then the bending moment due to this eccentricity may be regarded as divided between the lengths of columns above and below the beam in proportion to the stiffness of these lengths.

Thus in Fig. 114 :

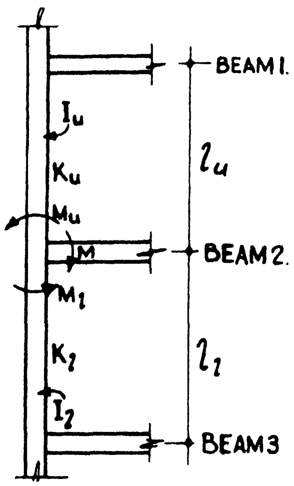


FIG. 114

if l_u = the length of column above the beam,
I_u = the moment of inertia of the upper length,
l_l = the length of column below the beam,
and I_l = the moment of inertia of this lower length,

the stiffness of the upper length is :

$$K_u = \frac{I_u}{l_u} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

and the stiffness of the lower length is :

$$K_l = \frac{I_l}{l_l} \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

If the bending moment due to the eccentric load is M and the bending moment in the upper length of column is M_u and the bending moment in the lower length of column is M_l then :

$$M = M_u + M_l$$

$$M_u = \left(\frac{K_u}{K_u + K_l}\right) \times M \quad . \quad . \quad . \quad . \quad (9)$$

and :

$$M_l = \left(\frac{K_l}{K_u + K_l}\right) \times M \quad . \quad . \quad . \quad . \quad (10)$$

Attention must be given to the distribution of these bending moments and o their shear forces throughout the column lengths.

Under By-law 89 if a continuous column is adequately restrained in position at the floor levels above and below that at which an eccentric load is applied, the bending moments in the column due to this eccentricity may be considered to be entirely dissipated at the adjacent floor levels.

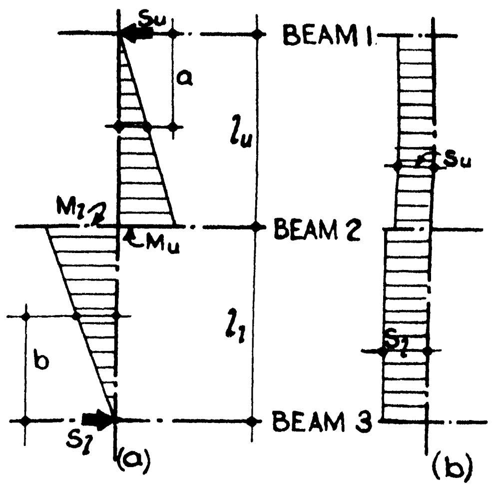


FIG. 115

The simplest case will first be investigated. It is that in which there is no such eccentric load from the beam or other support at the top of the upper length of column nor from the beam at the bottom of the lower length. In this case, as shown in Fig. 115, the bending moment M_u in the upper length would be resisted by a horizontal force S_u at the top of the upper length acting at an arm of length l_u, and the magnitude of the horizontal force S_u would, therefore, be such that :

$$M_u = S_u \times l_u \quad \text{or} \quad S_u = \frac{M_u}{l_u} \quad . \quad . \quad . \quad . \quad (11)$$

Similarly the bending moment M_l in the lower length of column would be resisted by a horizontal force S_l at the bottom acting at an arm of length l_l, and the magnitude of the horizontal force S_l would, therefore, be such that :

$$M_l = S_l \times l_l \quad \text{or} \quad S_l = \frac{M_l}{l_l} \quad . \quad . \quad . \quad . \quad (12)$$

The bending moments at any points in the column, distant a and b from S_u and S_l respectively, would be $(S_u \times a)$ and $(S_l \times b)$, and the bending moment diagrams would, therefore, be triangular, as shown in Fig. 115 (a).

These two horizontal forces S_u and S_l would have to be provided by the rest of the structure acting through the horizontal supports above and below the beam, and it would be necessary for the designer to check that there was such a provision. Usually, however, it will be found that the forces are relatively small and that the normal connection would be capable of providing them without any special modifications.

Because they are the only horizontal forces acting on the column as a result of the eccentric load, S_u and S_l will represent the shear forces in the column lengths l_u and l_l above and below the beam respectively, and the distribution of shear would be as shown in Fig. 115 (b).

In the general case in which there would be similar eccentric loads at the beam levels above and below the connection under consideration, the amounts of the eccentric load bending moments taken by the lengths of column above and below each beam must first be calculated from the column stiffnesses in the manner already described. Then the point in each column length must be established at which there is no bending moment. This may be done graphically as illustrated in Fig. 116 (*a*),

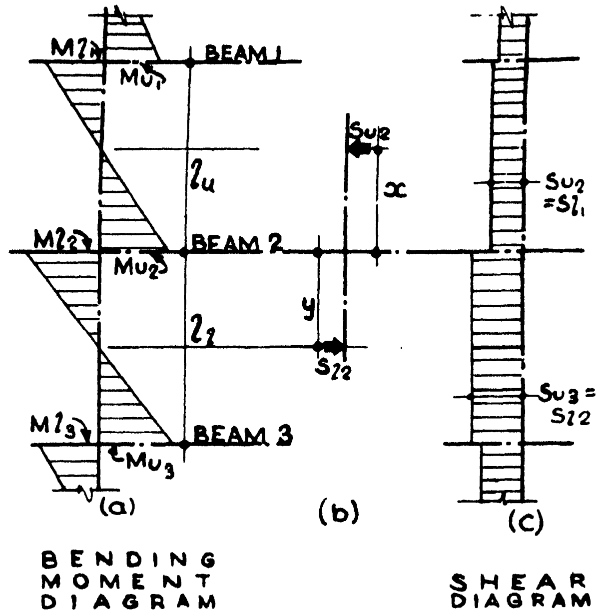


FIG. 116

in which the ends of the lines representing the magnitudes of M_{l_1} and M_{u_2} and of M_{l_2} and M_{u_3} are joined by straight lines which intersect the column centre line at points at distances x and y above and below the level of Beam 2 respectively. Alternatively x and y, Fig. 116 (*b*), may be found algebraically from the formulæ :

$$x = \left(\frac{M_{u_2}}{M_{u_2} + M_{l_1}}\right) \times l_u \quad . \quad . \quad . \quad . \quad . \quad (13)$$

and

$$y = \left(\frac{M_{l_2}}{M_{l_2} + M_{u_3}}\right) \times l_2 \quad . \quad . \quad . \quad . \quad . \quad (14)$$

At these points there must be set up in the column horizontal forces such as S_{u_2} and S_{l_2}, which, acting at the arms x and y, resist the bending moments M_{u_2} and M_{u_2} respectively such that :

$$M_{u_2} = (S_{u_2} \times x) \quad \text{or} \quad S_{u_2} = \frac{M_{u_2}}{x} \quad . \quad . \quad . \quad . \quad (15)$$

and

$$M_{l_2} = (S_{l_2} \times y) \quad \text{or} \quad S_{l_2} = \frac{M_{l_2}}{y} \quad . \quad . \quad . \quad . \quad (16)$$

The corresponding forces for the adjacent portions of column lengths must be equal to these values but of opposite sign, because, in order that a condition of equilibrium should obtain, the algebraic sum of the horizontal forces at these points must be zero. Therefore $S_{l_1} = S_{u_2}$; $S_{l_2} = S_{u_3}$, etc.

The forces S_{l_1}, S_{u_2}, S_{l_2}, etc., represent the shears in the respective portions of column, and the shear diagram for such a condition is of the form given in Fig. 116 (*c*).

These shear forces may also be calculated directly from formulæ which follow naturally from the foregoing investigations, namely :

$$S_{l_1} = S_{u_2} = \frac{M_{l_1} + M_{u_2}}{l_u} \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

$$S_{l_1} = S_{u_2} = \frac{M_{l_2} + M_{u_3}}{l_l}, \text{ etc.} \quad . \quad . \quad . \quad . \quad . \quad (18)$$

A fuller explanation of all such problems as that covered by this by-law is given in Chapter 15, where the effects of rigid connections and continuity in framed structures are investigated.

By-law 90. Increase in Working Stresses for Wind Loads.—Where the calculated stresses due to vertical and all other loads except wind loads in any beam or column or their connections are increased by the wind loads on the building, the working stresses given in By-laws 81 and 85 may be increased by an amount up to but not greater than 33⅓ per cent. This increase is not permitted for stresses caused by any other influence than that of the wind, nor is it permitted on the working stresses given for grillage beams and filler floor beams used in accordance with By-laws 82 and 83 respectively.

Table 25 gives values of permitted stresses in axially loaded columns, increased by 33⅓ per cent. for wind effects on the values given in By-law 85.

TABLE 25

DIRECT COLUMN STRESSES PERMITTED FOR INCREASED LOADS DUE TO WIND EFFECTS ONLY

Ratio of Effective Column Length to Least Radius of Gyration = l/r	Working Loads in tons per sq. in. of Cross Section = F_1	Ratio of Effective Column Length to Least Radius of Gyration = l/r	Working Loads in tons per sq. in. of Cross Section = F_1
20	9·60	86	6·05
22	9·52	88	5·89
24	9·44	90	5·73
26	9·36	92	5·60
28	9·28	94	5·47
30	9·20	96	5·34
32	9·12	98	5·20
34	9·04	100	5·07
36	8·96	102	4·94
38	8·88	104	4·80
40	8·80	106	4·66
42	8·72	108	4·53
44	8·64	110	4·40
46	8·56	112	4·29
48	8·48	114	4·18
50	8·40	116	4·08
52	8·29	118	3·97
54	8·18	120	3·87
56	8·07	122	3·79
58	7·97	124	3·71
60	7·86	126	3·63
62	7·73	128	3·55
64	7·60	130	3·47
66	7·47	132	3·39
68	7·34	134	3·31
70	7·20	136	3·23
72	7·07	138	3·15
74	6·93	140	3·07
76	6·80	142	2·99
78	6·67	144	2·91
80	6·53	146	2·83
82	6·37	148	2·75
84	6·21	150	2·67

It should be noted that although this by-law does not specifically allow the 33⅓ per cent. increase to be made on the value F_2, for combined direct and bending moment stresses in columns in accordance with By-law 87, it is the intention of the authorities to allow increases in stresses for combined direct and bending moment stresses also.

The authors suggest the use of the formula :

$$F_2 = fc + 10\left(1 - \frac{fc}{F_1}\right)\left(1 - 0{\cdot}002\frac{l}{r}\right) \text{ tons per sq. in.} \qquad (19)$$

This is based on exactly the same principles. F_1 has the values given in Table 25.

The values for combined bending moment and direct column stresses, as determined by the formula above, may be found from Fig. 117. Follow up the vertical line through the particular calculated value of fc until it intersects the appropriate $\frac{l}{r}$ line. Then follow the level of this point of intersection to the left or right and read off in either margin the permissible combined stress. Figure 117 may be used also for stresses increased by waiver.

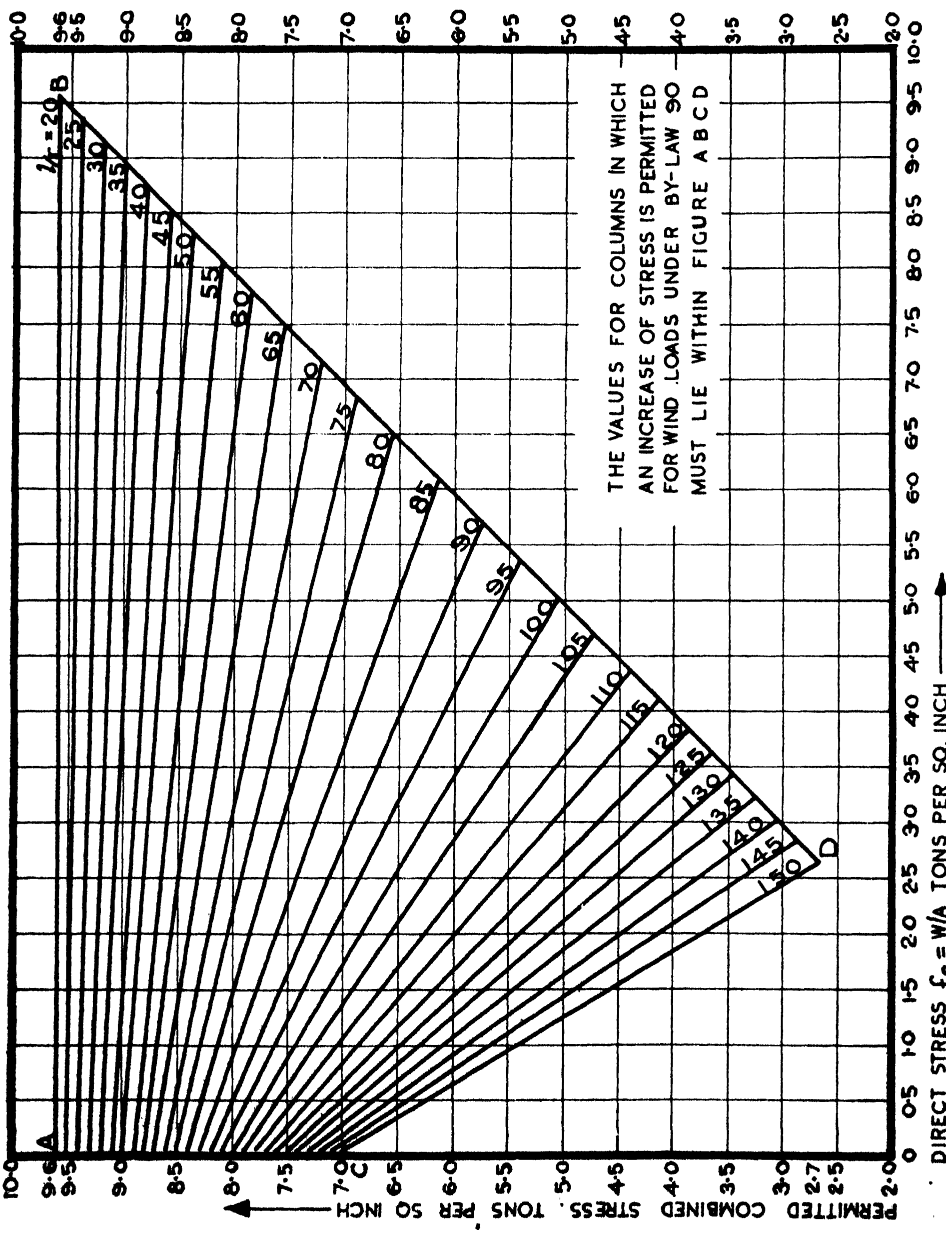


Fig. 117

By-law 91. Any Other Structural Steel Members in a Building.—By-laws 68 to 90 inclusive, which have been discussed in the preceding pages, apply specifically to the use of steel in the construction of a building in which all the loads and stresses are transmitted down through each storey height to the foundations either entirely by a skeleton framework of steel or partly by a skeleton framework and partly by a party wall or party walls.

All other steelwork which serves any structural purpose in a building must be to the satisfaction of the district surveyor, and it must have at least the stability and the amount of fire protection which is required for steelwork governed directly by By-laws 68 to 90 inclusive.

In those cases in which particular hardship would be imposed by the application of this by-law, as for instance by the demand for a much higher standard than that obtaining in the rest of the existing structure, waivers are frequently granted by the Council.

CHAPTER 8

WELDING REGULATIONS

APPLICATION may be made under the Regulations (made by the Council on 7 December, 1937) for the use of metallic arc welding in the fabrication and erection of structural steelwork by waiver of By-laws 63, 64, 69, 70, 72, 74, 75, 78, 81, 85, 86, 87, 90 and 91.

In the following notes where the word " welding " is used it refers only to metallic arc welding.*

Any waiver of a by-law or any approval for the use of welding which may be given by the Council is to apply only to the particular structure for which the application is made. A separate application must therefore be made for each case.

By-law 63 may be waived so as to permit the use of special welded members which should not be governed by the ordinary structural steel By-laws.

By-laws 69 and 70 may be waived so as to permit the attaching of column bases by welding with or without the use of stiffening webs and the use of welding in the treatment of column ends.

By-law 72 may be waived so as to permit column splices to be made by welding.

According to a formal interpretation of the Welding Regulations, By-law 72, relating to the attaching of caps and bases to solid round columns, is not to be waived, although there is little doubt that the Council would give consent to such caps being attached not by screwing or shrinking, as required, but by welding applied in the same manner as welding done under waiver of By-law 69.

By-laws 74 and 75 may be waived so as to permit the use of welding instead of rivets and bolts in the fabrication and erection of structural steelwork.

Although according to a formal interpretation of the Welding Regulations, By-law 81 may be waived, the working stresses for the structural steel itself should remain the same for welded work as those set out in this by-law for ordinary structural steel.

By-laws 85 and 86 also may be waived ; but no indication is given anywhere as to what would be the permissible stresses in welded columns and struts, nor as to what could be taken as the effective length of a welded column for the different conditions of end fixing. It would appear that the requirements regarding these matters for ordinary steelwork should also be used for welded constructions.

Where beams are rigidly fixed to columns by welding the effective lengths can be taken as indicated in Chapter 15.

The same argument applies to the waiver of By-law 87. The permitted column stresses for the combined effects of bending and direct stresses of an eccentric load should be independent of the method of construction.

By-laws 90 and 91 may be waived for the reason that they both refer to preceding By-laws for which waivers may be granted.

GENERAL CONDITIONS COVERING APPLICATIONS

It is necessary for adequate particulars, calculations and drawings to be submitted with the application ; but the Council will consider preliminary applications for approval in principle provided that the subsequent designs and methods meet with the detailed consents of the Council.

Nomenclature used on all plans and calculations is to conform to the British Standard Specification 499—1933.

* Regulations for Oxy-Acetylene Welding are being prepared by the Council and they will be included in Chapter 26 of Volume II.

The detailed calculations and plans must specify exact lengths, sizes and types of welds and welding which are proposed to be used. Information on the strength and ductility and other mechanical properties of electrodes and weld metal are required, as is also evidence that the work will be carried out by skilled workers, that the work will be supervised and that defective work will not be included. The district surveyor concerned must be satisfied that the welding used, or to be used, complies with the conditions of the Council's consent.

General Conditions Governing Design

The structural steel must comply with the British Standard Specification No. 15, and electrodes complying with the requirements for class A in the British Standard Specification No. 639 will probably be demanded. In accordance with the practical conditions under which the work shall be done, the Council will agree the maximum working stresses. In Table 26 values are given to indicate the probable maximum stresses which will be permitted :

Table 26

Classification of Stress in Welded Connections	Maximum permissible Stress in tons per sq. in.
Tension and compression in butt welds	8
Shearing in butt welds in webs of plate girders and joists . .	6
Shearing in butt welds other than webs of plate girders and joists	5
Stress in end fillet welds	6
Stress in side fillet welds, diagonal fillet welds and tee fillet welds	5

From these stresses, Tables 27 to 31 inclusive have been compiled for each of the classes. For the purposes of Tables 27 to 31, see Figs. 120 to 130.

Table 27

PERMITTED LOADS IN TENSION AND COMPRESSION IN ALL BUTT WELDS OTHER THAN J AND BEVEL WELDS

Plate Thickness, in.	Tons per Linear Inch of Weld
$\frac{1}{8}$	1
$\frac{3}{16}$	1·5
$\frac{1}{4}$	2
$\frac{5}{16}$	2·5
$\frac{3}{8}$	3·0
$\frac{7}{16}$	3·5
$\frac{1}{2}$	4
$\frac{5}{8}$	5
$\frac{3}{4}$	6
$\frac{7}{8}$	7
1	8
$1\frac{1}{4}$	10
$1\frac{1}{2}$	12
$1\frac{3}{4}$	14
2	16

For plates of other thicknesses the permitted loads are in direct proportion.
For J and bevel welds these permitted loads should be reduced by a quarter.

TABLE 28

PERMITTED SHEARING LOADS IN BUTT WELDS IN WEBS OF PLATE GIRDERS AND JOISTS

Plate or Web Thickness, in.	Tons per Linear Inch of Weld
$\frac{1}{4}$	1·5
$\frac{5}{16}$	1·88
$\frac{3}{8}$	2·25
$\frac{7}{16}$	2·63
$\frac{1}{2}$	3·0
$\frac{5}{8}$	3·75
$\frac{3}{4}$	4·5
$\frac{7}{8}$	5·25
1	6

For plates of other thicknesses the permitted loads are in direct proportion.

TABLE 29

PERMITTED SHEARING LOADS IN BUTT WELDS OTHER THAN IN WEBS OF PLATE GIRDERS AND JOISTS

Plate or Web Thickness, in.	Tons per Linear Inch of Weld
$\frac{1}{8}$	0·63
$\frac{3}{16}$	0·94
$\frac{1}{4}$	1·25
$\frac{5}{16}$	1·56
$\frac{3}{8}$	1·87
$\frac{7}{16}$	2·18
$\frac{1}{2}$	2·50
$\frac{5}{8}$	3·12
$\frac{3}{4}$	3·74
$\frac{7}{8}$	4·36
1	5·00

For plates of other thicknesses the permitted loads are in direct proportion.

TABLE 30

PERMITTED LOADS IN END FILLET WELDS

Fillet Size, in.	Tons per Linear Inch of Weld
$\frac{3}{16}$	0·79
$\frac{1}{4}$	1·05
$\frac{5}{16}$	1·31
$\frac{3}{8}$	1·57
$\frac{7}{16}$	1·84
$\frac{1}{2}$	2·10
$\frac{5}{8}$	2·62
$\frac{3}{4}$	3·15
$\frac{7}{8}$	3·68
1	4·20

For fillet welds of other sizes the permitted loads are in direct proportion.

Table 31

PERMITTED LOADS IN SIDE, DIAGONAL AND TEE FILLET WELDS

Fillet Size, in.	Tons per Linear Inch of Weld
$\frac{3}{16}$	0·65
$\frac{1}{4}$	0·87
$\frac{5}{16}$	1·09
$\frac{3}{8}$	1·31
$\frac{7}{16}$	1·53
$\frac{1}{2}$	1·75
$\frac{5}{8}$	2·19
$\frac{3}{4}$	2·62
$\frac{7}{8}$	3·06
1	3·5

For fillet welds of other sizes the permitted loads are in direct proportion.

It will be noted from these Tables that where a single or double J or bevel butt weld is required the reduction of one-fourth of the given safe loads must be made.

It will be observed also that the loads are given in each instance for 1 linear inch of weld. Permitted stresses have been defined in the Regulations as so many tons per square inch, and the effective area of the weld must be made great enough to ensure that the actual stress does not exceed the specified stress. In practice, however, it is inconvenient to calculate the required weld sizes by this method and it is much simpler to divide the total load which must be transmitted by the length available for welding, and to see from the appropriate table which weld size will afford at least this load per inch of run.

Particular attention should be given to the requirement for a run of weld metal on the back of a single V, U, J or bevel butt weld, because where this is not done it is necessary for the permitted stresses to be reduced by one-half, provided, however, that if there is another steel surface at the back of the joint and if the plates are cut to a sharp corner at the bottom of the groove with a gap of at least $\frac{1}{8}$ in., and if the first run is then made with an electrode not exceeding No. 8 gauge, this reduction need not be made.

In Figs. 118 and 119:

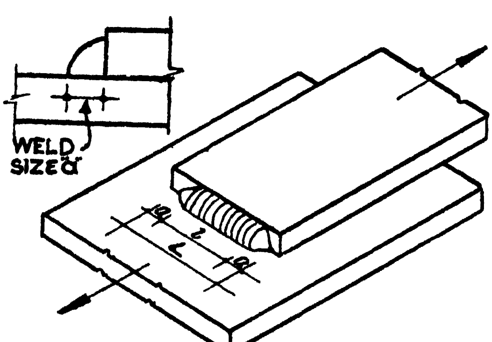


Fig. 118

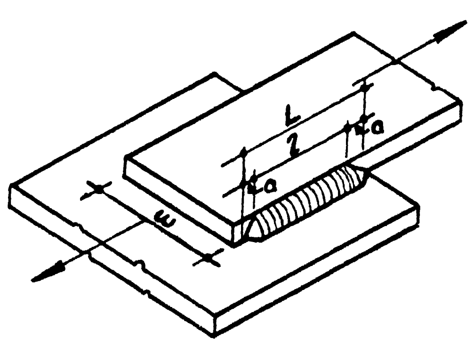


Fig. 119

$l = L - 2a$

l must not be less than $6a$ } whichever is the greatest.
l ,, ,, ,, ,, 2 in.
and in Fig. 119 l ,, ,, ,, ,, w

The strength of a fillet weld is to be calculated on the effective throat thickness, which is a dimension of 0·7 of the weld size, this size being the length of the shorter leg. The effective or calculation length is to be taken as the overall length minus twice the weld size, and in no case should the effective length be less than 2 in. or six times the weld size, whichever is the greater. When the angle between the surfaces actually to be welded is less than 60° or more than 110°, fillet welds should not be relied on to transmit any load.

The effective cross-section of the weld is defined as the effective length multiplied by the effective throat thickness in the case of fillet welds, and by the plate thickness in the case of butt welds.

It is specified that any eccentricity which introduces bending moments must be provided for in the calculations, and for this purpose the section modulus of the weld has to be used, this being the moment of inertia of the effective cross-section divided by the distance between its neutral axis and the edge of the cross-section farthest from it. This applies to the actual weld itself.

As in ordinary structural mechanics, direct stresses are found by dividing the load by the effective sectional area, and bending moment stresses by dividing the bending moment by the section modulus. Where both direct and bending stresses occur, the resultant or vectorial sum must be taken as the limiting stress. For bending moments and direct loads on groups of welds this requirement has been investigated further in Chapter 26.

The effects of bending moments and of other influences arising from the rigidity of a welded joint must be taken into account in the design of the rest of the structure. By-laws 88 and 89 give some indication as to how this is to be done, but a fuller investigation is given in Chapter 15.

It is recommended that welds should be arranged so that as far as possible the distribution of stresses may be known.

General Conditions Governing Detailed Preparation and Weld Sizes

For the most part these conditions are set out diagrammatically in Figs. 120 to 130 inclusive. Attention, however, should be given to the requirement, as far as it affects design stresses, for a run of weld metal on the back of a single V, U, J or bevel butt weld, as is pointed out in the previous section, treating of design

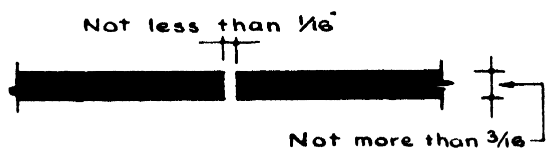


Fig. 120.—Square Butt

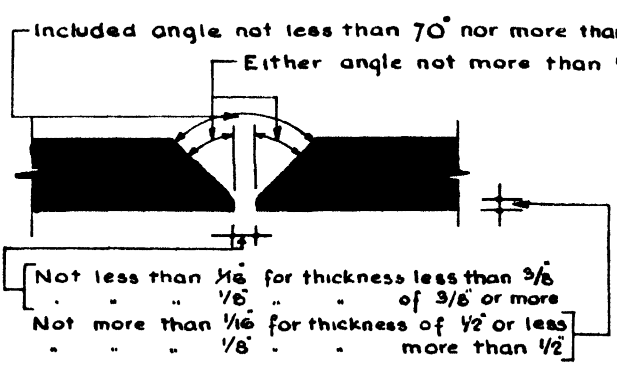


Fig. 121.—Single V Butt

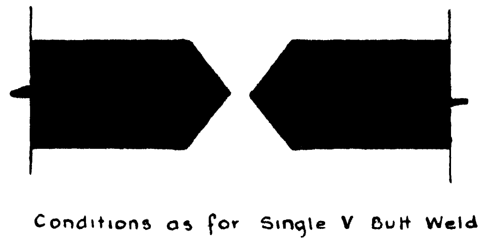


FIG. 122.—Double V Butt

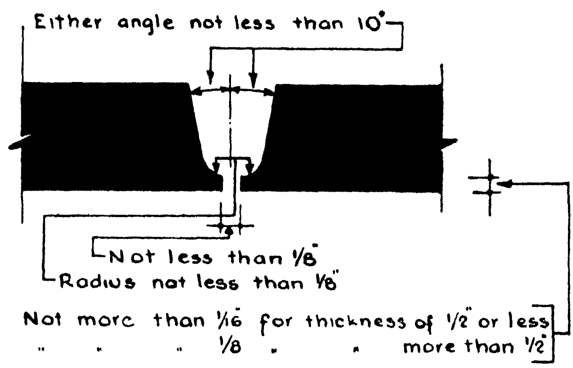


FIG. 123.—Single U Butt

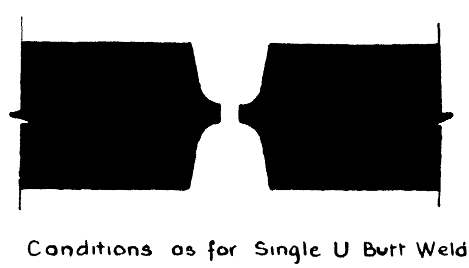


FIG. 124.—Double U Butt

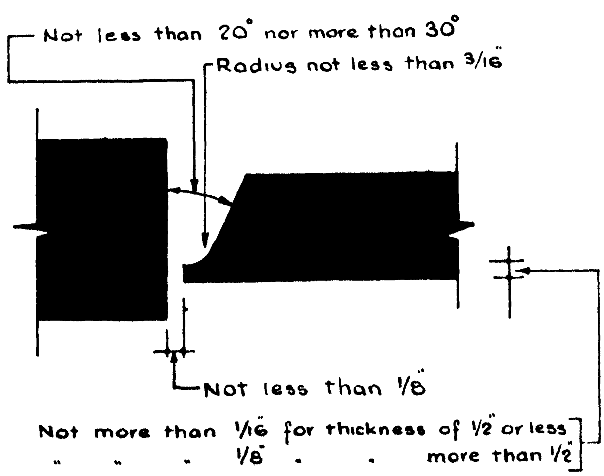


FIG. 125.—Single J Butt

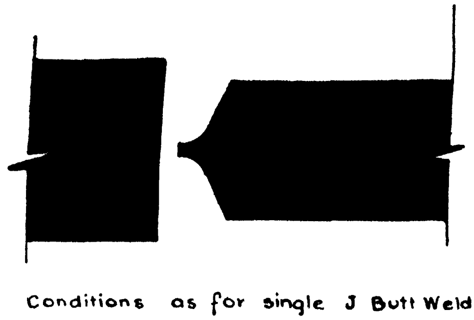


FIG. 126.—Double J Butt

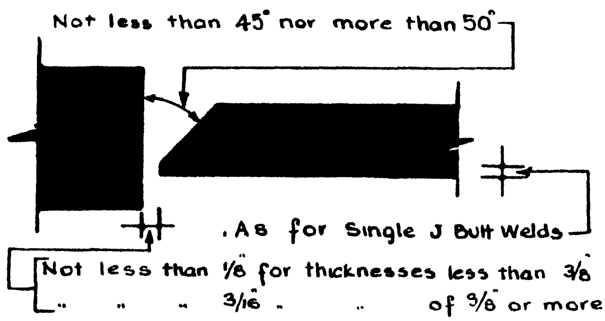


FIG. 127.—Single Bevel Butt

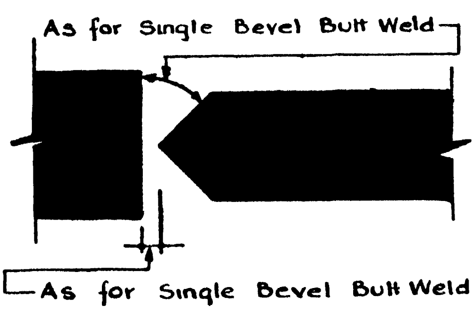


FIG. 128.—Double Bevel Butt

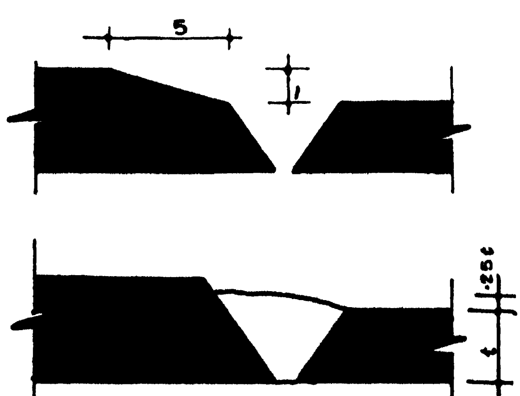


FIG. 129.—Butt Welds between Plates of Different Thicknesses

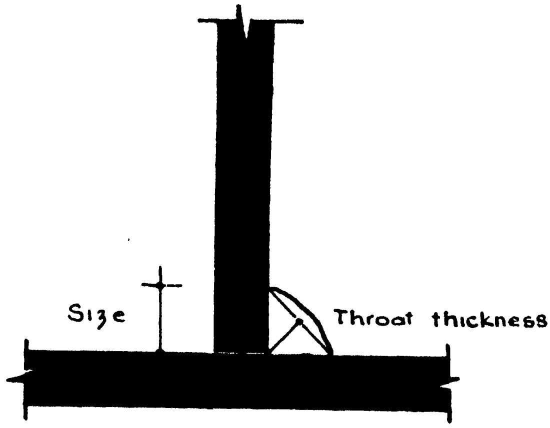


FIG. 130.—Fillet Weld

conditions. It is required that butt welds should be reinforced or built up at the centre to a height at least 10 per cent. greater than the thickness of the steel plates and when a flush surface is required, that the weld first be built up and then dressed flush, no reduction in stress being required when this is done.

Where the calculations do not call for continuous welds, intermittent fillets may be used; but, however, butt welds should always be continuous. The length of unwelded plate between intermittent fillet welds should not be more than twelve times the thickness of the thinner plate for compression members, nor more than sixteen times this thickness for tension members.

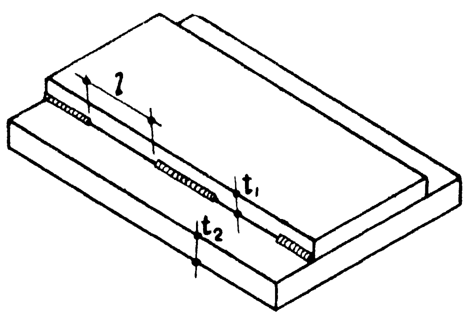


FIG. 131

For compression members:

$$\left.\begin{array}{l} l \text{ must not be greater than } 12t_1 \\ \text{or } l \quad ,, \quad ,, \quad ,, \quad ,, \quad 12t_2 \end{array}\right\} \text{whichever is the less.}$$

For tension members:

$$\left.\begin{array}{l} l \text{ must not be greater than } 16t_1 \\ \text{or } l \quad ,, \quad ,, \quad ,, \quad ,, \quad 16t_2 \end{array}\right\} \text{whichever is the less.}$$

Holes or slots may be used to increase the length available for fillet welds, but such holes or slots should not be so filled with weld metal that there is a direct connection between the welds on opposite sides. The dimensions of such holes and slots are specified in relation to plate widths and thicknesses, as shown in Fig. 132.

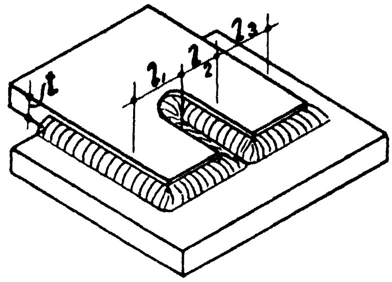


FIG. 132

$$\left.\begin{array}{l} l_2 \text{ must not be less than } 2t \\ l_2 \;\;,, \;\;\;\; ,, \;\;\;\; ,, \;\;\;\; ,, \;\; 1 \text{ in.} \end{array}\right\} \text{whichever is the greater.}$$

l_1 must not be less than $2t$
l_3 ,, ,, ,, ,, $2t$

$$\left.\begin{array}{l} r \text{ must not be less than } t \\ r \;\;,, \;\;\;\; ,, \;\;\;\; ,, \;\;\;\; ,, \;\; \tfrac{1}{2} \text{ in.} \end{array}\right\} \text{whichever is the greater.}$$

where r = radius of interior corners of slot or hole.

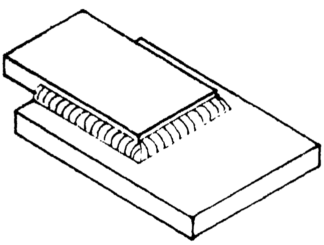


FIG. 133

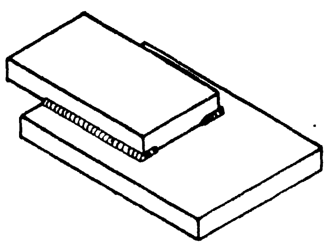


FIG. 134

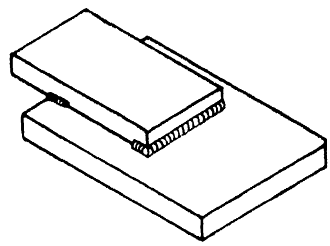


FIG. 135

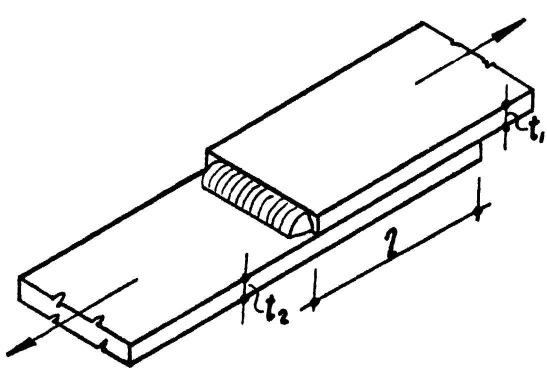
It is recommended that side and end fillets in combination should be adopted in preference to such single welds alone, and that the ends of welds should be returned round the ends of plates wherever possible. A single end fillet should not be used alone. See Fig. 133.

The ends of side and end fillets used alone are to be returned whenever possible. Returned lengths are not to be taken into account in the calculations.

The welds attaching bracing members which are not determined by calculations should be sufficient to develop the actual effective strength of the members.

The amount of lap of plates, where such a lap is adopted, must not be less than four times the thickness of the thinner plate. See Fig. 136.

FIG. 136



l must not be less than $4t_1$ or $4t_2$ whichever is the less.

In a member consisting of a composite section built up from plates or from other rolled sections, continuous welds should be used at the ends, and the authors recommend also at intermediate points where there are connections to be made to the member; and the lengths of such continuous welds should not be less than the greatest width of plate or section which is used. In such built-up members the width of the plate which is unsupported between the welds down its edges should not be greater than forty times the plate thickness. Welds joining web plates to flanges in any member must be sufficient to transmit the whole of the shearing forces due to bending moments or other causes, and the working stresses must not be exceeded. A uniform shear stress for the full depth of the girder may be assumed in the splices of webs of a plate girder, and such web plates should not be less in thickness than one-sixtieth of the depth unless they are specially stiffened.

The ends of the separate lengths of a column which must be spliced should be properly machined to give proper bearing, and if there is no tension on any part of the column section the welds need be sufficient only to hold the separate lengths accurately in position. Where there is tension in any part of the section the welds must be sufficient to resist the whole of the forces due to the bending moment causing that tension.

Members and joints should be designed so that all points are readily accessible for welding and for inspection and painting or other maintenance. Where joints are to be exposed to the weather or to any other detrimental influence they should be sealed by a light run of weld round all edges of contact or, alternatively, the surfaces must be held in sufficiently close contact by the welds to prevent the entry of moisture.

General Conditions Covering Workmanship

These clauses, which are in agreement with good engineering practice, specify that the surfaces of the materials for a distance of ½ in. from the joint should be cleaned of dirt and grease, paint and heavy rust, etc., and that the method of preparing the fusion faces should be such that a uniform result obtains and that the parent metals are not damaged. It is required, also, that the pieces should be held securely during the welding operations, for which the sequence should be such that, as far as possible, distortion is eliminated. Good clean weld metal of even contour and free from cavities is required, and undercutting is to be avoided, or, if it occurs to an unreasonable extent, it is to be made good. The slag is to be chipped from finished welds which, with the adjacent portions of the parent metal, may be coated with clean linseed oil, which will permit their inspection. Adverse weather or other conditions which will hinder the operator at his work are to be overcome by suitable screens and staging.

The principles of welded steel design are set out in Part 7, Vols. II and III, together with some examples and calculations.

CHAPTER 9

PART VI. REINFORCED CONCRETE. BY-LAWS 92–118

In By-laws 92 to 118 there are set out all the requirements for the design and construction of reinforced concrete to be used in buildings. The theory of reinforced concrete and suggestions for design, together with tables and graphs, based upon the following regulations are set out in Part 5, Vol. II, whilst this chapter gives only an explanation of the actual by-law requirements.

In the following pages the order of these by-laws has been altered slightly so as to bring together into groups the several by-laws relating to the one subject; but where this has been done a footnote has been made to draw the attention of the reader to the change.

By-laws 115 to 118, which refer to the rights of the district surveyor to modify or to supplement the by-laws, are given at the beginning of this chapter, because they specify the conditions under which the by-laws are to be applied.

By-laws 115 and 116 refer to the design of a building and By-laws 117 and 118 to the construction.

By-law 92. Reinforced Concrete to conform to the By-laws.—The formal demand is made that reinforced concrete must not be allowed to carry or transmit any load unless it is in accordance with the by-laws.

All concrete to be used as reinforced concrete must be at least equal to concrete III (in By-law 14) as regards the composition and strength. That is to say, concretes I, II and III and concretes IA, IIA and IIIA may be used, together with any concrete intermediate between concretes IA and III, but no other.

By-law 93. Supports for Reinforced Concrete.—" Construction which will support or transmit loading supported, collected or transmitted by reinforced concrete, shall comply with the requirements of these by-laws."

That is to say, all walls or piers or any other construction on which the loads carried by reinforced concrete are imposed, and the reinforced concrete itself, must be in accordance with the particular by-laws for such walls or piers or the other construction or for reinforced concrete.

By-law 115.[1] **Deflections, etc.**—" Reinforced concrete subjected to bending actions in a building shall possess adequate stiffness to prevent such deflection or deformation as might, in the opinion of the district surveyor, affect adversely the stability of such building or of any part thereof."

Any reinforced concrete member subject to bending deflects, and if such deflection is greater than a certain amount the member loses a very great part of its resistance to axial forces; in other words, occasional axial forces which might be set up by unequal settlement, vibration, unequal loading, etc., and which are not normally taken into consideration in the calculations, might produce high and important stresses in the member, which already would be subjected to the stresses due to the bending action.

Danger will be avoided if the proportion of the deflection to the length is kept within strict limits. Such a limit has been prescribed for steelwork (By-law 84);

[1] Introduced here from its numerical position.

but it is unfortunate that for reinforced concrete this important matter has been left entirely to personal discretion.

The calculation of deflection is a little more complicated for reinforced concrete than for steel, due to the fact that such deflection is influenced to a great extent by continuity and rigidity as well as by the load and by its own moment of inertia, which again generally varies considerably throughout the length of the member; but this difficulty can be overcome. The authors recommend the following rules:

1. Unless the calculated deflection of freely supported slabs is less than $\frac{1}{300}$ of the length, the ratio of the effective depth to the length should not be less than $\frac{1}{27}$.

2. Unless the calculated deflection of a loaded panel of a continuous slab (under the assumption that alternate spans are loaded) is less than $\frac{1}{300}$ of the length, the ratio of the effective depth to the length should not be less than $\frac{1}{36}$ for centre panels nor less than $\frac{1}{32}$ for end panels.

3. The effective depth of a flat slab, if drop panels are provided, should never be less than $\frac{1}{40}$ of the span for centre panels nor less than $\frac{1}{36}$ of the span in end panels.

The effective depth of a flat slab, if drop panels are not provided, should never be less than $\frac{1}{36}$ of the span in centre panels nor less than $\frac{1}{32}$ of the span in end panels.

4. Unless the calculated deflection of a freely supported beam is less than $\frac{1}{325}$ of the length, the ratio of the effective depth to the length should not be less than $\frac{1}{20}$.

5. Unless the calculated deflection of a continuous beam is less than $\frac{1}{375}$ of the length, the ratio of the effective depth to the length should not be less than $\frac{1}{24}$. (For notes on the calculations of deflections see Chapter 20.)

By-law 116.[1] **Restraint of Compression Members.**—" Reinforced concrete subjected to compression in a building shall possess adequate stiffness or be provided with adequate restraint to prevent such lateral flexure as might, in the opinion of the district surveyor, affect adversely the stability of such building or of any part thereof."

In most cases, where buildings either are designed to take wind stresses or (see By-law 6) have a height of less than twice the width and are properly stiffened by walls and floor slabs at each floor, the condition that adequate lateral support for columns must be provided, will be fulfilled. There are, however, exceptions, most commonly in industrial buildings, where compression members are supported laterally in a way which is open to criticism. For such cases (as, for example, the support of a crane girder in a factory) the reader is referred to the explanation of By-law 102 and to Chapter 15, in which the lateral stiffness of columns is discussed.

By-law 117.[1] **Construction of Reinforced Concrete Work.**—" The fabrication and erection of reinforced concrete shall be such as will ensure that the assumptions upon which the stresses in such concrete and its reinforcement have been calculated shall be fulfilled adequately at all times in the building of which such reinforced concrete forms part."

Thus the workmanship must be kept to a certain standard.

This standard is defined in By-laws 9, 15 and 96 (iii) as far as reinforced concrete is concerned, and Schedules I, II, III describe the tests for which the district surveyor is entitled to ask. The ultimate decision on the actual standard for any particular case rests with the district surveyor.

By-law 118.[1] **Tests.**—" Where the district surveyor finds substantial reason for doubt as to the sufficiency or suitability of the reinforced concrete for its purposes under these by-laws, the builder shall make such test or tests on such concrete as the district surveyor may require; and if such testing proves, in the opinion of the district surveyor, that such concrete is insufficient or unsuitable for its purposes under these by-laws, such concrete shall be removed and replaced with reinforced concrete which complies with these by-laws."

[1] Introduced here from its numerical position.

The tests mentioned in this by-law (By-law 118) are not those described in Schedules I, II, III; but are any additional tests which the district surveyor may consider necessary.

For example, such tests may be considered necessary if the concrete test cubes made in accordance with By-law 14 and tested after 28 days show a much lower strength than that which was expected, so that as a result the quality of the concrete placed at the same time as the cubes were made is in doubt; or if after the shuttering has been struck the concrete is shown not to be completely round the steel rods; or if the work is not in accordance with calculations or drawings passed by the district surveyor; or if concrete has been cast at too low a temperature; or for any other similar reason.

The tests referred to under this by-law may be loading tests, but only when it is clear that no further deterioration of the quality of concrete will take place later. They may also be chemical tests if the presence of any deleterious material or if a deficiency of cement or waterproofing agent is suspected.

Further, the cutting of holes in the concrete, to ascertain whether the concrete is properly rammed behind the reinforcement or if the required reinforcement is really present, may be necessary.

If as a consequence to such tests, part of the concrete of a structure is to be removed and replaced, care must be taken that:

(1) no temporary overstressing takes place in other parts which have not had to be cut out; and

(2) the whole of the concrete acts as a monolithic unit afterwards.

Because the stability of a building as a whole suffers by the cutting out and replacing of concrete, district surveyors will be reluctant to order such measures unless they are absolutely necessary, but it should always be kept in mind, first, that "prevention is better than cure," and, secondly, that the repair of a faulty concrete structure is an expensive item and therefore it pays to take the necessary precautions in the first place.

All those precautions are set out in By-laws 9 to 15.

By-law 94. Foundations for Reinforced Concrete.—" Loading supported, collected or transmitted by reinforced concrete shall be distributed upon the earth by concrete which shall:

" (*a*) comply with the requirements of By-law 32 in the same manner as is required for concrete which is to support walls or piers;

" (*b*) if plain, be of composition and quality not inferior to that designated V in By-law 14.

" The angle of dispersion through such plain concrete shall be taken at not less than 45° with the horizontal; and such plain concrete shall not be relied upon to resist shearing or tensile stresses otherwise than in accordance with this by-law; and

" (*c*) if reinforced, comply with the requirements of these By-laws.

" The pressure upon such distributing concrete shall be calculated, and such concrete, if plain, shall comply in all respects with the requirements of By-law 35 in the same manner as is required for plain concrete which is to support walls or piers."

By-laws 32 and 35 refer to foundations, and, as pointed out under By-law 60, the permitted stresses in such members are often much smaller than those for walls and piers of the same slenderness ratio. Therefore it may be assumed, if a reinforced concrete member is supported by concrete or brick walls or piers and not directly by foundations, that the stresses given by By-law 60 can be adopted.

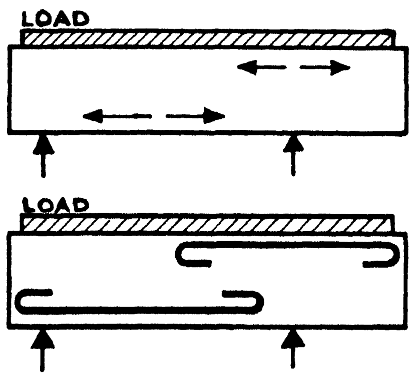
Reinforced concrete can be carried also by structural steel work, as is very frequently done in the case of floor slabs; but in order to carry any other reinforced concrete work on steel By-laws 91 and 113 would have to be observed.

By-law 95. Protection of Metal Reinforcement.—Protection must be provided for any metal reinforcement against any damage which might reduce the stability of the building or any part of it. The minimum cover of concrete as required by By-law 97 will normally afford this protection, except that in foundations, etc., the minimum cover should be 3 in.

By-law 96. Reinforcement.—(i) *Material.*—Reinforcement is to consist of steel or other metal complying with the requirements of By-law 15, under which it will be seen that reinforcement is to be steel in accordance with the British Standard Specification No. 15—1936, except when certain conditions, prescribed for a particular case by the Council, are observed, and in this case steel complying with the British Standard Specification No. 165—1929 may be used for slabs only, or other steel or metal may be used generally for that particular case. The regulations for applications to use any other reinforcement than that of steel of British Standard Specification No. 15 standard are given in the explanation of By-law 15.

(ii) *Uses.*—The reinforcement shall be so combined with the concrete that it will:

(*a*) Resist all tensile forces.



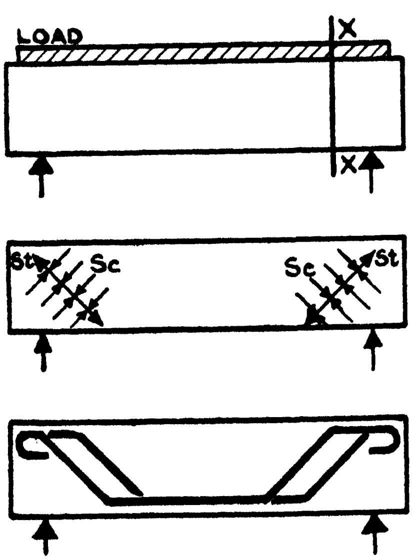
The vertical arrows represent the supports of a beam.

The horizontal arrows represent tension due to the bending action.

Tensile reinforcement.

FIG. 137

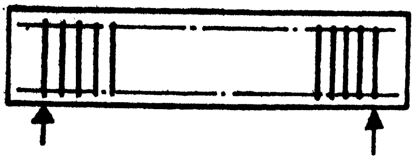
(*b*) Resist the tensile components of the whole of the shear forces in cases where the permitted shear stresses in the concrete (as defined in By-law 99) are exceeded.



The diagonal arrows represent the compressive and the tensile components of the shear force.

Shear reinforcement to take tensile components.

FIG. 138



Shear reinforcing stirrups to take tensile components.

Fig. 138 (*contd.*)

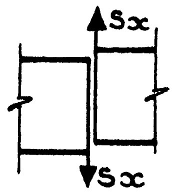


FIG. 139

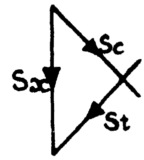


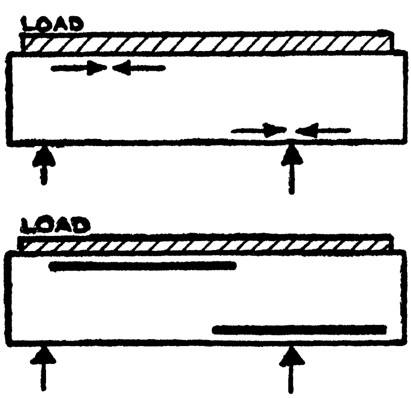
FIG. 140

These figures indicate purely diagrammatically the types of stress which occur.

It is shown in Chapter 20, Volume II, that any shear force can be regarded as a combination of a compressive stress and a tensile stress acting at right angles to each other and at 45° to the shear force. As long as the shear stress lies within the permitted limit, the tensile stress, being small, may be considered to be resisted by the concrete. If the shear stress exceeds the permitted value, then only the compressive stress can be assumed to be resisted by the concrete and steel reinforcement must be introduced to take all tensile stresses. An exact investigation is given in Chapter 20, Volume II, where the use of stirrups is also described.

(*c*) Assist the concrete to resist compressive forces in certain cases.

These cases are those of columns (for which a minimum amount of steel is specified whether it is actually required to assist the concrete or not) and those of beams in which the compressive stresses due to the bending moments would be greater than the values allowed in By-law 99. Such a use of beam compressive steel is shown in Fig. 141.



The vertical arrows represent the supports of a beam.

The horizontal arrows represent compression due to the bending action.

Compressive reinforcement.

FIG. 141

In such cases the stresses to be used in determining the reinforcement are to be taken as fifteen times the calculated stresses in the concrete which is displaced. Because the reduction of the concrete area due to this displacement is not generally given as a factor in the expression for the concrete section, the steel area can be assumed to be equivalent to only $(15 - 1) = 14$ times the displaced concrete area. Thus for a section under compressive stresses which are due to bending action, the steel stress must be taken as only fourteen times the stress in the concrete where the steel is located, and not as fourteen times the extreme fibre stress.

The reinforcement may be used also to take the whole of the compression forces in a beam, the resistance of the concrete being neglected, and in such a case the working stress in the steel may be taken at a higher value, namely 18,000 lb. per sq. in.

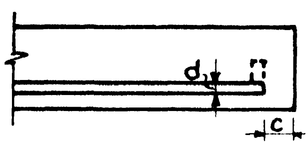
In the foregoing examples the reinforcement has been shown in the form of rods or bars; but there is nothing in the by-laws to preclude the use of rolled steel sections as reinforcement. Provision for such reinforcement has been made in By-law 83.

(iii) *Preparation.*—"Reinforcement shall, immediately before being placed in the concrete[1], be free from loose mill scale, loose rust, oil, or other matter which might affect adversely the proper combination of such reinforcement with such concrete."

By-law 97. Concrete Cover to Reinforcement.—All reinforcement must be embedded in the concrete which it reinforces. The thickness of solid concrete between any part of the reinforcement and the face of the structural concrete (i.e. not to the face of the plaster or any decorative finish) must not be less than that given for each case below. The symbol *c*, etc., has been used to represent the thickness of the concrete cover, and *d*, etc., to represent the diameter of the reinforcing bar or rod.

(*a*) At ends of any rods:

(i) not anchored by hooks:



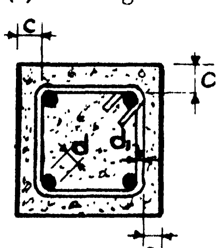
c must not be less than $2d$ } whichever is the greater.
c ,, ,, ,, ,, 2 in.

FIG. 142

(ii) anchored by hooks:

c must not be less than the cover required at any other point on the bar.

(*b*) On longitudinal or main rods in columns (Fig. 143):

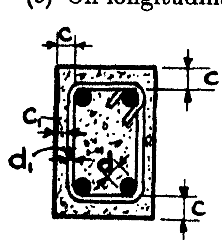


c must not be less than *d* } whichever is the greater.
c ,, ,, ,, ,, $1\frac{1}{2}$ in.

FIG. 143

In special cases waivers may be granted to allow a cover on column steel of 1 in.

(*c*) On longitudinal or main rods in a beam (Fig. 144):



c must not be less than *d* } whichever is the greater.
c ,, ,, ,, ,, 1 in.

FIG. 144

[1] That is to say before any concrete is placed round reinforcement.

(*d*) On tensile, compressive and shear rods in slabs (Fig. 145):

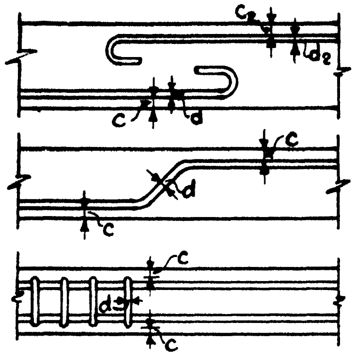


FIG. 145

$$\left.\begin{array}{l} c_2 \text{ must not be less than } d_2 \\ c_2 \quad ,, \quad ,, \quad ,, \quad ,, \quad \tfrac{1}{2} \text{ in.} \end{array}\right\} \text{whichever is the greater.}$$

$$\left.\begin{array}{l} c \quad ,, \quad ,, \quad ,, \quad ,, \quad d \\ c \quad ,, \quad ,, \quad ,, \quad ,, \quad \tfrac{1}{2} \text{ in.} \end{array}\right\} \text{whichever is the greater.}$$

(See (*f*) below.)

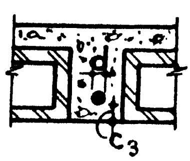
(*e*) On all rods other than those specially detailed above and other than bindings:

$$\left.\begin{array}{l} c_1 \text{ must not be less than } d_1 \\ c_1 \quad ,, \quad ,, \quad ,, \quad ,, \quad \tfrac{1}{2} \text{ in.} \end{array}\right\} \text{whichever is the greater.}$$

This requirement will apply most commonly to stirrups and links of all kinds.

(*f*) Special rules are given for the cover on reinforcement at the lower side of hollow-tile floor slabs. They are contained in Clauses (viii) and (ix) of the resolution passed by the Council on 23 June, 1925, when conditions were approved under which hollow tile floors and roofs could be regarded as fire-resisting for the purposes of the First Schedule to the Act. These requirements remain in force. (See p. 232).

When no slip tiles, etc., are used:



$$\left.\begin{array}{l} c_3 \text{ must not be less than } d \\ c_3 \quad ,, \quad ,, \quad ,, \quad ,, \quad \tfrac{3}{4} \text{ in.} \end{array}\right\} \text{whichever is the greater.}$$

FIG. 146

When slip tiles, etc., are used:

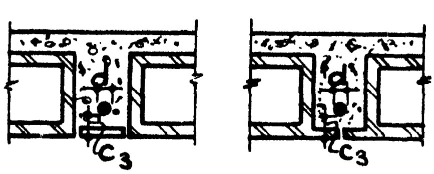


FIG. 147

$$\left.\begin{array}{l} c_3 \text{ must not be less than 1 in.} \\ c_3 \quad ,, \quad ,, \quad ,, \quad ,, \quad c_4 + c_5 \end{array}\right\} \text{whichever is the greater.}$$

$$\left.\begin{array}{l} c_4 \quad ,, \quad ,, \quad ,, \quad ,, \quad \tfrac{1}{2} \text{ in.} \\ c_4 \quad ,, \quad ,, \quad ,, \quad ,, \quad d \end{array}\right\} \text{whichever is the greater.}$$

$$c_5 \quad ,, \quad ,, \quad ,, \quad ,, \quad \tfrac{1}{2} \text{ in.}$$

For further requirements for hollow-tile floors, see Chapter 18, Volume II.

By-law 114.[1] **Restriction on Welding of Reinforcement.**—With the one exception given in the next paragraph, welding of reinforcement for reinforced concrete may be adopted only by special permission of the Council and under the provisions of the Council's consent. Application to use welding must be made under the Welding Regulations (given at the end of Part V) and a separate application must be made for each individual case.

Special permission does not have to be obtained from the Council, however, where a welded mesh reinforcement is to be used in a solid concrete slab, if that mesh consists of round or square bars not greater than ·4 in. in diameter or side respectively, and if the bars are arranged transversely to each other and welded together at the points of contact by the electrical fusing of the metals. Thus metallic arc, carbon arc, or resistance welding may be used for such reinforcement without a special application to the Council being required.

In other cases it may be to advantage to butt weld the ends of main reinforcement so as to obtain continuity of rod without the difficulties which sometimes arise when the rods have to be lapped, as in a normal splice. It may also be to advantage to weld special plates to the ends of rods to provide anchorage or even to weld them on to other main rods or to structural steel sections.

The details of such joints as those of beams to columns, in rigid frames, may be simplified to a very great extent by the use of welding.

By-law 113.[1] **Reinforced Concrete not specifically governed by By-laws 99 to 112.**—Where reinforced concrete serves any structural purpose whatever in a building in which all the loads and stresses are transmitted down through each storey length to the foundations in any other way than either entirely by a skeleton framework of reinforced concrete or partly by such a skeleton framework and partly by a party wall or party walls, then such reinforced concrete must be to the satisfaction of the district surveyor, and it must have a standard of stability and quality not less than that required for reinforced concrete under By-laws 92 to 112 .

That is to say, that whilst all the conditions prescribed for reinforced concrete skeleton buildings must be observed, the district surveyor may if he considers it necessary impose additional special requirements. However, sound and stable construction will generally be achieved by observance of the normal requirements.

By-law 98. Reinforced Concrete in completely framed Buildings or in combined Frame and Party Wall Buildings.—Where all the loads and stresses in a building are transferred down through each storey height to the foundations either entirely by a skeleton framework of reinforced concrete or partly by a steel framework of reinforced concrete and partly by a party wall or party walls, then such reinforced concrete must be designed and constructed in accordance with the requirements of By-laws 99 to 112.

By-law 99. Permissible Stresses in Reinforced Concrete.—The compressive, shear and bond stresses must be calculated for the concrete used in any reinforced concrete structural member; and the member must be so proportioned that these calculated stresses do not exceed the values given in Tables 32 and 33 for Ordinary and Quality A concretes respectively. The index figures given at the tops of certain columns refer to the numbers of the notes which are given immediately after the tables.

The modular ratio m has now been fixed for all cases at the value 15 ; and as a result calculations have been considerably simplified as compared with those required previously, when different values of m were to be adopted for different cases.

The design data in Part V Vol. 2 have all been based on the constant value of m, namely, 15.

[1] Introduced here from its numerical position.

TABLE 32

PERMISSIBLE STRESSES IN ORDINARY CONCRETE [7]

L.C.C. Designation	Popular Reference	Cubic Feet of Aggregate to 112 lb. of Cement		Compression in Concrete, lb. per. sq. in.			Shear, lb. per sq. in.	Punching Shear, lb. per sq. in.	Maximum Shear in Concrete, lb. per sq. in.	Bond, lb. per sq. in.		β
		Fine	Coarse	Bending	Direct (1)	(2)	(3)	(4)	(5)	Normal (8)	(6)	
I	1 : 1·0 : 2·0	$1\frac{1}{4}$	$2\frac{1}{2}$	975	780	10,920	98	196	392	123	246	·05467
	1 : 1·1 : 2·2	$1\frac{3}{8}$	$2\frac{3}{4}$	950	760	10,640	95	191	382	120	241	·05351
	1 : 1·2 : 2·4	$1\frac{1}{2}$	3	925	740	10,360	93	186	371	118	236	·05236
	1 : 1·3 : 2·6	$1\frac{5}{8}$	$3\frac{1}{4}$	900	720	10,080	90	180	361	115	230	·05120
	1 : 1·4 : 2·8	$1\frac{3}{4}$	$3\frac{1}{2}$	875	700	9,800	88	175	350	113	225	·05004
II	1 : 1·5 : 3·0	$1\frac{7}{8}$	$3\frac{3}{4}$	850	680	9,520	85	170	340	110	220	·04889
	1 : 1·6 : 3·2	2	4	830	664	9,296	83	166	332	108	216	·04800
	1 : 1·7 : 3·4	$2\frac{1}{8}$	$4\frac{1}{4}$	810	648	9,072	81	162	324	106	212	·04711
	1 : 1·8 : 3·6	$2\frac{1}{4}$	$4\frac{1}{2}$	790	632	8,848	79	158	316	104	208	·04622
	1 : 1·9 : 3·8	$2\frac{3}{8}$	$4\frac{3}{4}$	770	616	8,624	77	154	308	102	204	·04533
III	1 : 2·0 : 4·0	$2\frac{1}{2}$	5	750	600	8,400	75	150	300	100	200	·04444

TABLE 33

PERMISSIBLE STRESSES IN QUALITY A CONCRETE [7]

L.C.C. Designation	Popular Reference	Cubic Feet of Aggregate to 112 lb. of Cement		Compression in Concrete, lb. per. sq. in.			Shear, lb. per sq. in.	Punching Shear, lb. per sq. in.	Maximum Shear in Concrete, lb. per sq. in	Bond, lb. per sq. in.		β
		Fine	Coarse	Bending	Direct (1)	(2)	(3)	(4)	(5)	Normal (8)	(6)	
IA	1 : 1·0 : 2·0	$1\frac{1}{4}$	$2\frac{1}{2}$	1250	1000	14,000	125	250	500	150	300	·06667
	1 : 1·1 : 2·2	$1\frac{3}{8}$	$2\frac{3}{4}$	1220	976	13,664	122	244	488	147	294	·06533
	1 : 1·2 : 2·4	$1\frac{1}{2}$	3	1190	952	13,328	119	238	476	144	288	·06400
	1 : 1·3 : 2·6	$1\frac{5}{8}$	$3\frac{1}{4}$	1160	928	12,992	116	232	464	141	282	·06267
	1 : 1·4 : 2·8	$1\frac{3}{4}$	$3\frac{1}{2}$	1130	904	12,656	113	226	452	138	276	·06133
IIA	1 : 1·5 : 3·0	$1\frac{7}{8}$	$3\frac{3}{4}$	1100	880	12,320	110	220	440	135	270	·06000
	1 : 1·6 : 3·2	2	4	1070	856	11,984	107	214	428	132	264	·05867
	1 : 1·7 : 3·4	$2\frac{1}{8}$	$4\frac{1}{4}$	1040	832	11,648	104	208	416	129	258	·05733
	1 : 1·8 : 3·6	$2\frac{1}{4}$	$4\frac{1}{2}$	1010	808	11,312	101	202	404	126	252	·05600
	1 : 1·9 : 3·8	$2\frac{3}{8}$	$4\frac{3}{4}$	980	784	10,976	98	196	392	123	246	·05467
IIIA	1 : 2·0 : 4·0	$2\frac{1}{2}$	5	950	760	10,640	95	190	380	120	240	·05333

Note (1).—These direct stresses are the maxima permitted for any compression in a structural member due to direct loading. They do not apply to compressive stresses set up in a member by bending actions. They refer, for example, to the stress in a column in which the slenderness ratio is not greater than 50 (the slenderness ratio being determined by By-law 102), and they must be reduced where necessary in accordance with the requirements of By-law 101 for slenderness ratio stresses.

When bending occurs either by itself or in combination with a direct force the permissible stresses given under the heading " Bending " may be adopted.

Note (2).—The figures given under the heading (2) are the stresses in the steel reinforcement of concrete under direct compressive forces when that concrete is stressed up to its full permitted value. They have been based on the values of the concrete stresses and on the modular ratio.

For example, if the gross cross-sectional area of concrete in a column (composed of a 1 : 1·3 : 2·6 Quality A concrete and having a slenderness ratio not greater than 50) is *Ac*, and if the total cross-sectional area of the longitudinal reinforcement is *As*, then that column would be permitted by these By-laws to carry a total load of ($Ac \times 928$) lb. + ($As \times 12{,}992$) lb. (see Table 33).

Note (3).—If a shear stress on any cross-section of a member (this stress being calculated by dividing the total shear force at that section by the width of the section multiplied by the depth of the lever arm) exceeds the value given for the appropriate concrete, then the member must be provided with shear reinforcement. The width of the section is to be taken as the smallest width occurring between the neutral axis and the main reinforcement.

Thus in Figs. 148 and 149 :

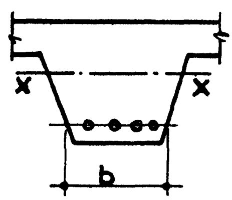


FIG. 148

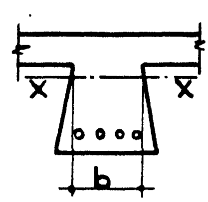


FIG. 149

b is to be taken as the width.

Note (4).—These values may be taken for the punching shear in footings and similar members which are constructed of reinforced concrete complying with these By-laws. They do not apply to punching shear in flat slabs. The circumference of the member which brings the load on to the concrete subjected to punching shear is to be taken for determining the area under such punching shear.

Note (5).—When it is necessary to provide shear reinforcement in a member because the shear stress exceeds that permitted in the concrete (see Note 3) the whole of the shear force must be resisted by this reinforcement. In other words, in such cases the shear resistance of the concrete is neglected in the calculations.

At the same time the shear area as defined in Note 3 must be such that the computed shear stress on this area for the whole of the shear force does not exceed four times the permitted shear stress. That is to say, even when the whole of the shear is taken by reinforcement the calculated stress must not be greater than that given under columns 5 in Tables 32 and 33. If such a value would be exceeded the section of the member must be increased until these requirements are satisfied.

Note (6).—The figures given under the heading (6) are the maxima permitted local bond stresses on plain rods when such rods are under tensile stresses due to bending actions only and when that tension varies between one point and another due to the change in bending moment between those points. In order to verify that the bond stress in such rods does not exceed these permitted stresses the appropriate values given for β may be used thus : The sum (D) of the diameters of all such tensile rods must not be less than β times the bending moment divided by the total shear force at the same point, or D must not be less than $\frac{\beta M}{S}$.

Note (7).—By-law 99 gives the permitted stresses specifically for concretes IA, IIA, IIIA, I, II and III only ; but it permits proportional stresses for intermediate mixes. For convenience, certain of these mixes have been given already in Tables 32 and 33. Further interpolation may be made for a mix within the upper and lower limits. Under By-law 14 the district surveyor has the authority to order or to agree a variation in the portion of fine to coarse aggregate within the limits 1 : 1·5 and 1 : 2·5, and the stresses for any such concrete are to be found from the proportion of cement to the total amount of fine and coarse aggregate, each being measured separately.

For example, it may be found that for particular materials the most suitable concrete is a Quality A concrete mixed in the proportion of 2 cu. ft. of fine aggregate and 4·4 cu. ft. of coarse aggregate to 112 lb. of cement. This is a 1 : 1·6 : 3·52 concrete, i.e. it has 5·12 cu. ft. of fine and coarse aggregate to 1 cu. ft. of cement. It is therefore near enough to a 1 : 1·7 : 3·4 concrete, in which there are 5·1 cu. ft. of fine and coarse aggregate to 1 cu. ft. of cement. The stresses given in Table 33 for the latter mix could, therefore, be adopted.

Note (8).—The " normal " bond stress defines the minimum length permitted for any bar between the beginning of the anchorage (i.e. excluding the anchorage) and any point at which it has developed tensile or compressive stress. Alternatively if the length is already fixed by practical considerations the " normal " bond stress determines the maximum diameter of the bar.

By-law 100. Permissible Stresses in Reinforcement.—The tensile and compressive stresses in the steel reinforcement of reinforced concrete must be calculated and the amount of reinforcement (within the limits required by other By-laws, such as By-law 104) must be such that these calculated stresses are not greater than those given for the different cases below :

(*a*) Tension in all reinforcement (other than helical reinforcement in columns), i.e. tension in the main bars, bent up bars, and stirrup in beams, slabs, etc. :

18,000 lb. per sq. in.

(*b*) Tension in helical reinforcement in columns (for the calculation of such reinforcement see Chapter 21, Volume 2) :

13,500 lb. per sq. in.

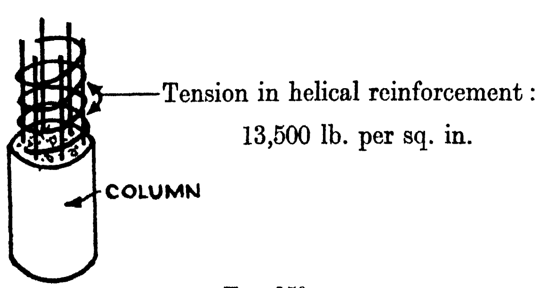


Fig. 150

(*c*) Longitudinal compression in a beam in which the compressive resistance of the concrete is neglected :

18,000 lb. per sq. in.

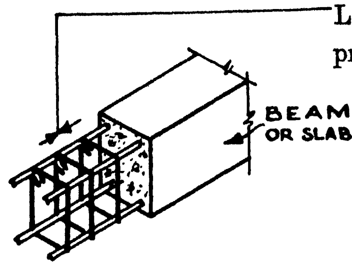


FIG. 151

It is considered that the above requirements of case (*c*) would apply also to the corresponding reinforcement in a slab, although only beams are mentioned in the By-laws.

(*d*) Longitudinal compression, direct, or due to bending in a member, in which the compressive resistance of the concrete is taken into account:

$$15 \times fc \text{ lb. per sq. in.}$$

fc = calculated compressive stress in surrounding concrete.

(See the special remark made under By-law 96 (ii) (*c*), and Note 2, Tables 33 and 34.)

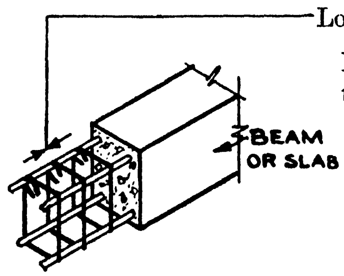


FIG. 152

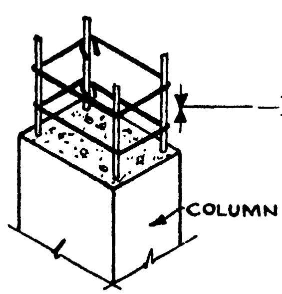


FIG. 153

By-law 101. Permissible Stresses in Reinforced Concrete Columns.—The permissible stress in a reinforced concrete column or in any part of it is to be determined by the slenderness ratio. The slenderness ratio is the ratio of the effective length to the radius of gyration, and the manner in which the effective length is to be found is explained under By-law 102, where it is indicated also which radius of gyration is to be adopted.

The area and the radius of gyration are to be taken from the overall dimensions, except in the case of spirally reinforced columns.

The slenderness ratio of no reinforced concrete column must be greater than 120.

TABLE 34

ORDINARY CONCRETE

Working Stresses in Concrete and Steel Reinforcement in Columns

l/r	I		$1:1\frac{1}{8}:2\frac{1}{4}$		$1:1\frac{1}{4}:2\frac{1}{2}$		$1:1\frac{3}{8}:2\frac{3}{4}$		II		$1:1\frac{5}{8}:3\frac{1}{4}$		$1:1\frac{3}{4}:3\frac{1}{2}$		$1:1\frac{7}{8}:3\frac{3}{4}$		III	
50	780	10,920	755	10,570	730	10,220	705	9,870	680	9,520	660	9,240	640	8,960	620	8,680	600	8,400
55	741	10,374	717	10,042	694	9,709	670	9,377	646	9,044	627	8,778	608	8,512	589	8,246	570	7,980
60	702	9,828	680	9,514	657	9,198	634	8,884	612	8,568	594	8,316	576	8,064	558	7,812	540	7,560
65	663	9,282	642	8,986	621	8,687	599	8,391	578	8,092	561	7,854	544	7,616	527	7,378	510	7,140
70	624	8,736	604	8,458	584	8,176	564	7,898	544	7,616	528	7,392	512	7,168	496	6,944	480	6,720
75	585	8,190	566	7,930	548	7,665	529	7,405	510	7,140	495	6,930	480	6,720	465	6,510	450	6,300
80	546	7,644	529	7,402	511	7,154	493	6,912	476	6,664	462	6,468	448	6,272	434	6,076	420	5,880
85	507	7,098	491	6,874	475	6,643	458	6,419	442	6,188	429	6,006	416	5,824	403	5,642	390	5,460
90	468	6,552	453	6,346	438	6,132	423	5,926	408	5,712	396	5,544	384	5,376	372	5,208	360	5,040
95	429	6,006	415	5,818	402	5,621	387	5,433	374	5,236	363	5,082	352	4,928	341	4,774	330	4,620
100	390	5,460	378	5,290	365	5,110	353	4,940	340	4,760	330	4,620	320	4,480	310	4,340	300	4,200
105	351	4,914	340	4,762	329	4,599	317	4,447	306	4,284	297	4,158	288	4,032	219	3,906	270	3,780
110	312	4,368	302	4,234	292	4,088	282	3,954	272	3,808	264	3,696	256	3,584	248	3,472	240	3,360
115	273	3,822	264	3,706	256	3,577	247	3,461	238	3,332	231	3,234	224	3,136	217	3,038	210	2,940
120	234	3,276	227	3,178	219	3,066	212	2,968	204	2,856	198	2,772	192	2,688	186	2,604	180	2,520

TABLE 35

QUALITY "A" CONCRETE

Working Stresses in Concrete and Steel Reinforcement in Columns

l/r	IA		1 : 1⅛ : 2½		1 : 1¼ : 2½		1 : 1⅜ : 2¾		IIA		1 : 1⅝ : 3¼		1 : 1¾ : 3½		1 : 1⅞ : 3¾		IIIA	
50	1,000	14,000	970	13,580	940	13,160	910	12,740	880	12,320	850	11,900	820	11,480	790	11,060	760	10,640
55	950	13,300	922	12,901	893	12,502	865	12,103	836	11,704	807	11,305	779	10,906	750	10,507	722	10,108
60	900	12,600	873	12,222	846	11,844	819	11,466	792	11,088	765	10,710	738	10,332	711	9,954	684	9,576
65	850	11,900	825	11,543	799	11,186	774	10,829	748	10,472	722	10,115	697	9,758	671	9,401	646	9,044
70	800	11,200	776	10,864	752	10,528	728	10,192	704	9,856	680	9,520	656	9,184	632	8,848	608	8,512
75	750	10,500	728	10,185	705	9,870	683	9,555	660	9,240	637	8,925	615	8,610	592	8,295	570	7,980
80	700	9,800	679	9,506	658	9,212	657	8,918	616	8,624	595	8,330	574	8,036	553	7,742	532	7,448
85	650	9,100	631	8,827	611	8,554	592	8,281	572	8,008	552	7,735	533	7,462	513	7,189	494	6,916
90	600	8,400	582	8,148	564	7,896	546	7,644	528	7,392	510	7,140	492	6,888	474	6,636	456	6,384
95	550	7,700	534	7,469	517	7,238	501	7,007	484	6,776	467	6,545	451	6,314	434	6,083	418	5,852
100	500	7,000	485	6,790	470	6,580	455	6,370	440	6,160	425	5,950	410	5,740	395	5,530	380	5,320
105	450	6,300	437	6,111	423	5,922	410	5,733	396	5,544	382	5,355	369	5,166	355	4,977	342	4,788
110	400	5,600	388	5,432	376	5,264	364	5,096	352	4,928	340	4,760	328	4,592	316	4,424	304	4,256
115	350	4,900	340	4,753	329	4,606	319	4,459	308	4,312	297	4,165	287	4,018	276	3,871	266	3,724
120	300	4,200	291	4,074	282	3,948	273	3,822	264	3,696	255	3,570	246	3,444	237	3,318	228	3,192

For columns having a slenderness ratio of 50 or less the permissible stresses may be taken as those given in Tables 32 and 33; but for columns with slenderness ratios between 50 and 120, the permissible stresses are given in Tables 34 and 35.

It will be seen that there are two values given for each concrete mix and slenderness ratio. The first of these is the permissible concrete stress and the second is the appropriate steel stress which may be used in the manner described under Note (2) for By-law 99.

For concretes of mixes intermediate between those of concretes IA and III which may not be given in Tables 34 and 35 and for columns of intermediate slenderness ratios the permissible stresses may be obtained by interpolation in the same manner as has been given in Note (7) to By-law 99.

The By-laws do not refer specifically to slender columns which have to resist bending moments in addition to the direct loads; but the Council will agree to the following manner of treating such cases: The permitted stress, which may be taken as that given in Table 32 or 33 under the heading " Bending," is to be reduced by a coefficient depending on the slenderness ratio of the column. These coefficients are given in Table 36.

TABLE 36

Slenderness Ratio	Coefficient
50	1·0
60	0·9
70	0·8
80	0·7
90	0·6
100	0·5
110	0·4
120	0·3

Thus if the slenderness ratio of a column composed of 1 : 1·4 : 2·8 Quality A concrete is 90 and if the column must resist bending in addition to the direct load. then the permitted stress may be taken as $1{,}130 \times 0{\cdot}6 = 678$ lb. per sq. in.

By-law 102. Effective Lengths and Radii of Gyration of Reinforced Concrete Columns. —Six conditions for end fixing and continuity of columns are given in this by-law, and the proportion of the actual column length which must be taken for each of these conditions and which must be regarded as the " effective length " for the purposes of calculating the slenderness ratio is given. The by-law points out that the values given are only for the few typical cases and that they are based on the general principles on which the effective length must be ascertained to the district surveyor's satisfaction for the actual end conditions in any particular case.

There is no further indication in the by-laws as to how such intermediate effective lengths are to be determined; but in Chapter 15 the authors have given certain rules by which the amounts of positional and rotational restraints may be computed. Tables 52 to 54 have also been compiled setting down the effective lengths of columns in terms of the actual lengths for these different proportions of end restraints.

By-law 102 actually specifies the effective lengths given in Tables 37 and 38 (the illustrations, which indicate purely diagrammatically the end conditions intended for each case, and the index letters and references to other parts of this volume have been introduced by the authors):

TABLE 37

COLUMNS OF ONE STOREY

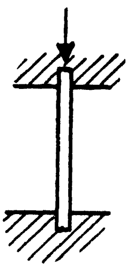
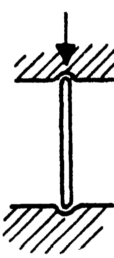
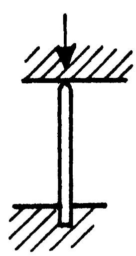
	Type of Column	Effective column length
A	Properly restrained at both ends in position and direction  FIG. 154	0·75 of the actual column length
B	Properly restrained at both ends in position but not in direction  FIG. 155	Actual column length.
C	Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end  FIG. 156	A value intermediate between the actual column length and twice that length, depending upon the efficiency of the imperfect restraint

TABLE 38

COLUMNS CONTINUING THROUGH TWO OR MORE STOREYS

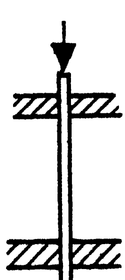
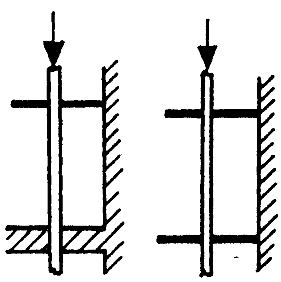
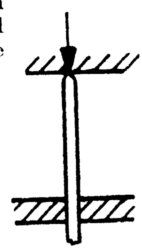
	Type of Column	Effective column length
D	Properly restrained at both ends in position and direction  FIG. 157	0·75 of the distance from floor level to floor level

TABLE 38—*continued*

COLUMNS CONTINUING THROUGH TWO OR MORE STOREYS—*continued*

	Type of Column	Effective column length
E	Properly restrained at both ends in position and imperfectly restrained in direction at one or both ends  FIG. 158 FIG. 159	A value intermediate between 0·75 and 1·00 of the distance from floor level to floor (or roof) level, depending upon the efficiency of the directional restraint
F	Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end  FIG. 160	A value intermediate between the distance from floor level to floor (or roof) level and twice that distance, depending upon the efficiency of the imperfect restraint

The correct effective length having been ascertained in any case, it is usually necessary only to take the least radius of gyration of the section to obtain the slenderness ratio from which the permissible column stress may be found under By-law 101. Cases frequently occur, however, in which a column is braced laterally either for its full height in one direction or at certain points in that direction. Thus in Fig. 161 if l is the effective length of a column unsupported in one direction and if in the other direction there is an intermediate lateral support which is so attached that the effective length of the column above and below are l_1 and l_2 respectively, then the value of whichever of $\frac{l}{r_x}$, $\frac{l_1}{r_y}$, $\frac{l_2}{r_y}$ is the greatest must be taken as the slenderness ratio by which the permissible column stress will be determined. In these three expressions for the slenderness ratios the symbols r_x and r_y refer to the radii of gyration of the column section about its xx and yy axes respectively.

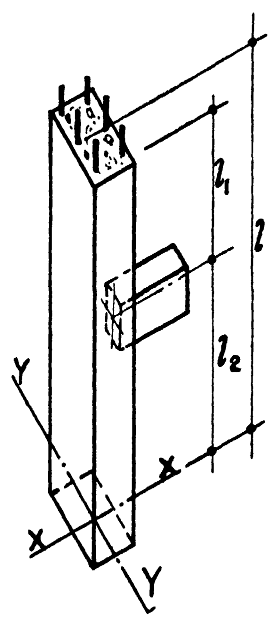


FIG. 161

By-law 103. Increase in Working Stresses for Wind Loads.—Where the calculated stresses due to vertical and all other loads except wind loads in any reinforced concrete member other than a secondary floor beam or a member in the roof construction are increased by the wind loads on the building, the working stresses allowed by By-laws 99, 100 and 101 may be increased by an amount up to but not more than 33⅓ per cent. This increase is not permitted for stresses caused by any other influence than that of the wind, nor is it permitted for secondary floor beams or

for structural members in the roof construction above the topmost floor of the building.

It should be noted that the exceptions to this by-law are different from those of By-law 90 for steelwork.

By-law 104. Percentage of Column Reinforcement.—" A reinforced concrete column shall have longitudinal steel reinforcement, and the cross-sectional area of such reinforcement shall not be less than 0·8 per cent. nor more than 8 per cent. of the gross cross-sectional area of the column required to transmit all the loading in accordance with these By-laws."

The wording of this by-law is very ambiguous and can be interpreted to have a meaning which is not intended. The intention is that:

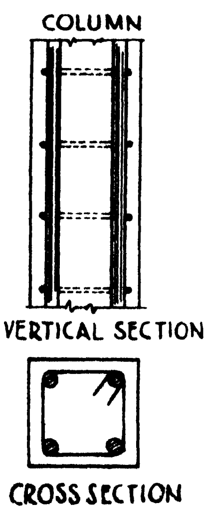
(*a*) No reinforced concrete column shall have reinforcement of greater sectional area than 8 per cent. of the gross sectional area of the column.

(*b*) No reinforced concrete column in which the whole of the gross sectional area is stressed up to the permitted value shall have reinforcement of less sectional area than 0·8 per cent. of the gross sectional area of the column.

And that

(*c*) No reinforced concrete column in which the stress calculated on the whole of the gross sectional area is less than the permitted value shall have reinforcement of less sectional area than 0·8 per cent. of the area which would be required to resist the load at the full permitted stress. It will be noted in this case that it is not the gross sectional area of the column on which this area of reinforcement is to be computed. The amount of reinforcement may therefore be less than 0·8 per cent. of the gross area; but it must not be less than 0·8 per cent. of the area required to transmit the load at the full permitted stress. (See Chapter 21, for practical formulæ.)

The last condition is of most importance in the case of reinforced concrete bearing walls, because it allows such walls to be treated as columns without, as formerly, requiring an excessive and unnecessary amount of reinforcement in them.



In Fig. 162, if

A = gross overall area of column section,

As = total cross sectional area of all longitudinal reinforcement,

and Ac = area of concrete required at the full working stress to carry the portion of the total column load which is not taken by the steel,

then As must not be less than $0{\cdot}008Ac$.

As must not be more than $0{\cdot}08A$.

FIG. 162

For purposes of calculation, the overall sectional area of the column may be used. Where helical reinforcement is used in a column there must be provided not less than six longitudinal rods which must be spaced at equal distances inside the helical reinforcement and in contact with it.

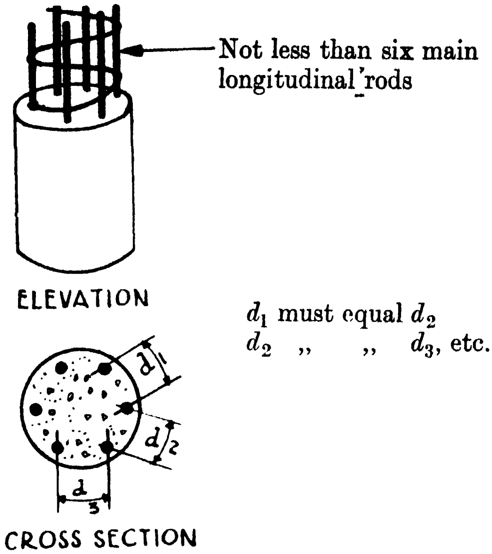
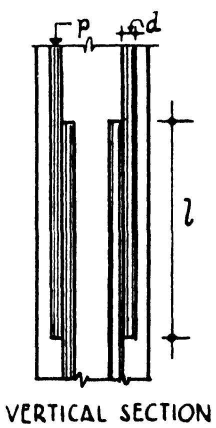


FIG. 163

Where it is necessary to splice the longitudinal reinforcement in a column the rods must be lapped in contact with one another, and the length of the lap must be not less than 24 in. unless the loads in the rods can be transmitted by a shorter length in which the permissible bond stress (By-law 99) is not exceeded.

Thus in the column in Fig. 164 :

if p = the load in a longitudinal rod,
d = the diameter of the rod, and
u = the permissible normal bond stress from Tables 32 or 33 (By-law 99), then



l must not be less than $24d$ in.
or
l must not be less than $\dfrac{p}{\pi du}$ in.

} whichever is the lower value.

FIG. 164

If fs = the stress in the steel, l must not be less than $\dfrac{fsd}{4u}$, which, if the column is stressed up to the permitted value, is practically identical with $24d$. In other words, when a column is fully stressed the length of a splice is governed by the $24d$ limit, and in other cases this length can be reduced in proportion to the actual stress obtaining.

By-law 105. Column Links.—All the longitudinal rods of a reinforced concrete column must be restrained laterally by transverse links or by helical reinforcement.

The diameter of such transverse links must not be less than $\frac{1}{4}$ in. and the ends must be properly anchored. The longitudinal pitch of the links must not be greater than:

(a) the least lateral dimension of the column,
(b) twelve times the least diameter of longitudinal reinforcement, or
(c) 12 in.

} whichever is the least value.

In Figs. 165 and 166:

if d = diameter of transverse link,
d_1 = diameter of smallest longitudinal rod,
b = smaller lateral dimension of the column (i.e. b is less than b_1), and
p = longitudinal pitch of transverse links,
then

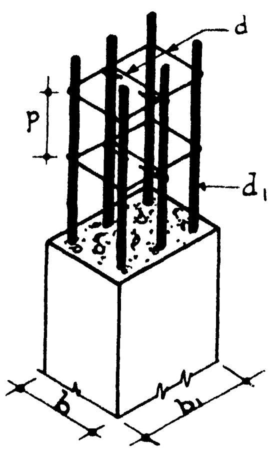


FIG. 165

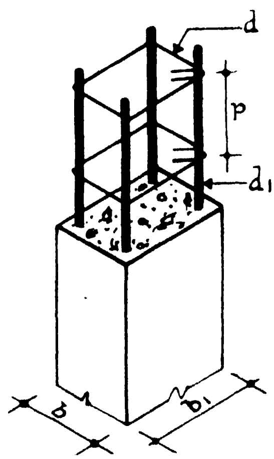


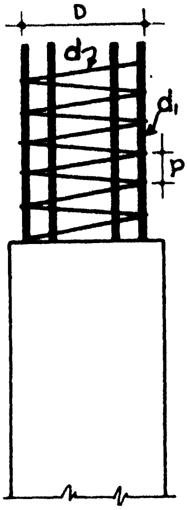
FIG. 166

d must not be less than $\frac{1}{4}$ in.
p must not be greater than b
p ,, ,, ,, ,, $12d_1$ } whichever is the least.
p ,, ,, ,, ,, 12 in.
d_1 ,, ,, ,, ,, 2 in.
d_1 must not be less than $\frac{1}{2}$ in. } By-law 106.

Where helical reinforcement is used in a reinforced concrete column the ends of the helical reinforcement must be properly anchored in the concrete, and the pitch must be constant and not greater than 3 in. or one-sixth of the core diameter, whichever is the less, nor must it be less than 1 in. or three times its diameter, whichever is the greater.

Thus in Fig. 167:

if d = diameter of helical reinforcement,
d_1 = diameter of longitudinal reinforcement,
D = diameter of concrete core inside the helix,
p = uniform pitch of helix,
then



d must not be less than $\frac{3}{16}$ in.
d_1 ,, ,, ,, ,, $\frac{1}{2}$ in. } By-law 106.
d_1 must not be greater than 2 in.

p must not be less than 1 in.
p ,, ,, ,, ,, $3d$ in. } whichever is the greater.

p must not be greater than 3 in.
p ,, ,, ,, ,, $\frac{D}{6}$ in. } whichever is the less.

FIG. 167

In each of the foregoing cases of transverse links and helical reinforcement, it may be considered that proper anchorage will be obtained if the ends are turned round one of the main longitudinal rods so as to project into the concrete core for a distance equal to at least four times the diameter of such links or helical reinforcement.

Very frequently column links in accordance with this by-law provide a lower degree of safety than is called for by the By-laws covering other features of reinforced concrete. The effect of links is not only to restrain the longitudinal rods from buckling but also to prevent splitting of the concrete under transverse stresses.

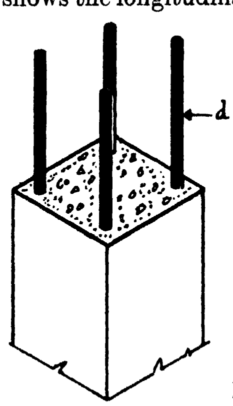
The authors strongly recommend the provision in any part of a column of as many links as are necessary to ensure that the total amount of steel in the links (not including the amount used in anchorages) represents not less than 0·4 per cent. of the cubic contents of that part of the column.

By-law 2 entitles the district surveyor to ask for calculations for stirrups, which calculations will, in fact, result usually in 0·4 per cent. of steel being sufficient. Thus the district surveyor can enforce a greater degree of stability than this by-law would give.

By-law 106. Diameters of Reinforcement.—No steel used in reinforced concrete may be greater than 2 in. nor less than $\frac{3}{16}$ in. in diameter except in the case of mesh reinforcemen tused to resist only tensile stresses, when the diameter must not be less than $\frac{1}{10}$ in.

In addition to these general requirements, there are the special restrictions on column longitudinal reinforcement, which must not be less than $\frac{1}{2}$ in., and on the main reinforcement of beams and slabs, which must not be less than $\frac{1}{4}$ in. except in the case of mesh reinforcement mentioned above.

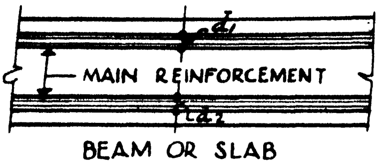
In Fig. 168, which shows the longitudinal reinforcement of a column, the diameter of the rods being d:



d must not be less than $\frac{1}{2}$ in.
d must not be greater than 2 in.

FIG. 168

In Fig. 169, which shows the main reinforcement of a beam or slab, the diameters of the rods being d_1 and d_2 :



d_1 must not be less than $\frac{1}{4}$ in.
d_1 must not be greater than 2 in.
d_2 must not be less than $\frac{1}{4}$ in.
d_2 must not be greater than 2 in.

FIG. 169

By-law 107. Spacing of Rods.—Except in cases (i) and (ii) enumerated in the succeeding paragraphs, the clear distance between rods in reinforced concrete must not be less than the diameter of the rods nor less than the greatest size of the coarse aggregate plus $\frac{1}{4}$ in., whichever of these two dimensions is the greater. When the diameters of two adjacent rods are different, the clear distance between those rods must not be less than the diameter of the larger rod nor less than the greatest size of the coarse aggregate plus $\frac{1}{4}$ in., whichever of these two dimensions is the greater. Thus in Figs. 170 and 171, if a is the greatest size of the coarse aggregate in the concrete and if d is the clear distance between the rods :

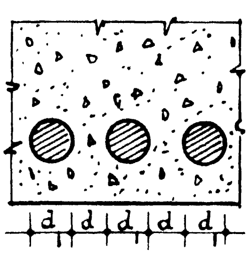


FIG. 170

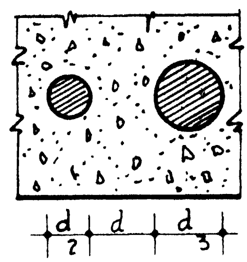


FIG. 171

d must not be less than d_1 ⎫
d ,, ,, ,, ,, $(a + \frac{1}{4})$ in. ⎭ whichever is the greater.

d must not be less than d_3 ⎫
d ,, ,, ,, ,, $(a + \frac{1}{4})$ in. ⎭ whichever is the greater.

(i) The requirements of the preceding paragraph do not apply to the vertical distances between two horizontal main rods, such as in multi-layer tensile or compressive reinforcements in a beam, nor do they apply to the distances between two diagonal rods one above the other. In each of these cases the clear distance vertically or at right angles between the rods must not be less than $\frac{1}{2}$ in.

In Figs. 172 and 173 :

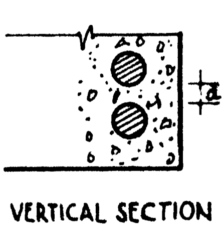


FIG. 172

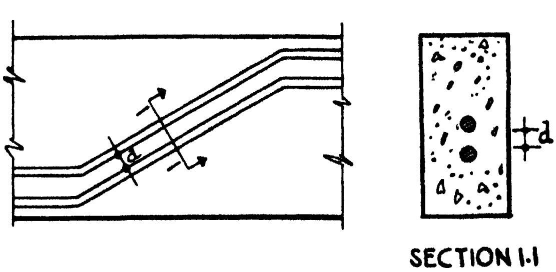


FIG. 173

d must not be less than $\frac{1}{2}$ in.

(ii) When two rods are spliced or when one is transverse to the other, the requirements for a clear distance between them do not apply and they may be in contact, as shown in Figs. 174 and 175.

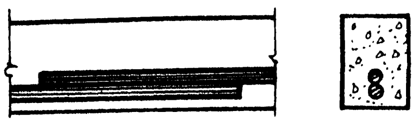


FIG. 174

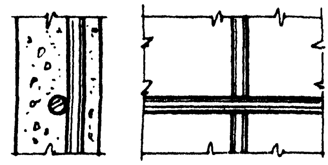


FIG. 175

The distance between the centres of distribution rods in a solid reinforced concrete slab (see By-law 110) must not be more than four times the effective depth of the slab.

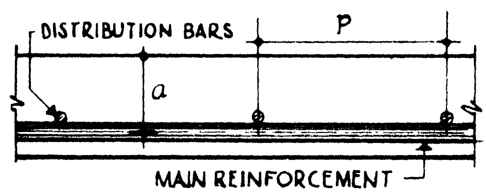


FIG. 176

p must not be greater than $4a$.

If At is the aggregate cross-sectional area of the main rods, and
if Ad ,, ,, ,, ,, ,, ,, distribution rods, then

$$Ad \text{ must not be less than } \frac{At}{10} \text{ (By-law 110).}$$

By-law 108. Shear Reinforcement.—If the shear stress on any cross-section of a member (this stress being calculated by dividing the total shear force at that section by the width of the section multiplied by the depth of the lever arm) exceeds the value given under headings (3) in Tables 32 and 33 (By-law 99) for the appropriate concrete, then the member must be provided with shear reinforcement.

This reinforcement must be capable, within the working stress given under By-law 100, of resisting the tensile components of the shear force at that cross-section in the member in the manner described in clauses (*b*) under By-law 96. A description of the calculations for shear reinforcement is given in Chapter 20, Volume 2.

The reinforcement may be allowed to resist a total shear force of an amount which would produce calculated stresses in the concrete only up to the values given under headings (5) in Tables 32 and 33 (By-law 99). Where the total shear forces would produce yet greater stresses, the concrete section of the member must be so increased that these values would not be exceeded.

For the nature of such shear reinforcement, see By-law 109.

By-law 109. Stirrups.—Shear reinforcement in a member may be provided in either of two common ways. The first is by introducing separate vertical stirrups. These should preferably pass round the main tensile rods, to which they must be wired, but if this should not be possible they may be securely fastened by wiring to a rod round which they do not pass. This latter method should be adopted only in exceptional cases.

The ends of the stirrups must be properly anchored into the concrete of the member, and it is suggested that the best way to do this is to turn each end round a longitudinal main rod, or round a trimmer rod, so that a straight length of *approximately four times the diameter of the stirrup projects into the concrete core. A second way is by turning up at an angle the main reinforcing bars as shown in Fig. 138.*

By-law 110. Distribution Rods in Solid Reinforced Concrete Slabs.—*Distribution rods must be provided at right angles to the main tensile reinforcement of a solid concrete slab which spans in one direction. The aggregate cross-sectional area of such distribution rods must not be less than one-tenth of that of the main tensile reinforcement and they must be disposed as required by By-law 107.*

It is generally sufficient to provide such distribution bars over the main reinforcement for positive bending moments only. No distribution bars are required for hollow tile slabs.

By-law 111. Securing of Compression Rods in Beams.—Every rod taking compressive forces in a beam must be anchored at intervals by secondary ties or stirrups which must pass round or be hooked over that compressive rod and must continue directly through the concrete to the tensile reinforcement round which they must also be fastened. Shear stirrups may be used to serve this purpose, but it must be noted that each compressive rod must be attached to stirrup legs.

When some of the compressive forces due to the bending action are taken by the concrete, the longitudinal distance between the anchorages must not be greater than twelve times the diameter of the compressive rods. When the whole of the compressive forces due to the bending action are taken by the reinforcement, the longitudinal distance between the anchorages must not be greater than eight times the diameter of the compression rods, and in addition the rods must be braced laterally, that is to say, they must be securely tied to the top bars of rectangular stirrups passing round the whole group of rods or they must be fixed laterally in some other suitable manner.

In Figs. 177 and 178 :

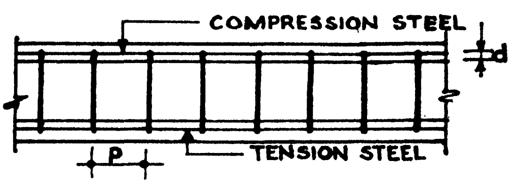
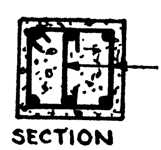


FIG. 177



Stirrups properly hooked or passed round compression steel and anchored to tensile steel.

FIG. 178

If the compressive resistance of the concrete is considered,

p must not be greater than $12d$;

and if the compressive resistance of the concrete is ignored,

p must not be greater than $8d$.

By-law 112. Hooks and Bends.—" Hooks and other anchorages of reinforcement in reinforced concrete shall be of such form, dimensions and arrangement as will ensure their adequacy without overstressing the concrete, or any other material."

There is no indication given in the by-laws as to how such hooks or other anchorages are to be arranged ; but such information is given in the Memorandum and in Chapter 18. Strangely enough the by-laws do not specifically require hooks or other anchorages, although this by-law defines some of the properties which hooks or other anchorages should possess. It seems perfectly clear, however, that the legislators intended that all reinforcement, which could under any condition of loading develop tensile stress, should be provided with hooks or other anchorages at the ends, which anchorages would be additional to any length of bar required for purposes of bond. See Note 8, By-law 99.

Hooks of the form given in Fig. 179 may be used. A straight length not less than fourteen times the bar diameter may also be used for anchorage.

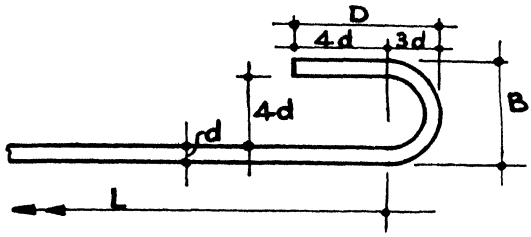


Fig. 179

The extra lengths of rod to be added to the length outside the anchorages are given in Table 39, together with the overall dimensions of hooks.

Table 39

Diameter of Bar, in.	See Fig. 179, in.		Amount to be added to "L," in.	
d	B	D	For two straight lengths	For two hooks
$\frac{1}{2}$	3	$3\frac{1}{2}$	14	12
$\frac{5}{8}$	$3\frac{3}{4}$	$4\frac{3}{8}$	17·5	15
$\frac{3}{4}$	$4\frac{1}{2}$	$5\frac{1}{4}$	21	18
$\frac{7}{8}$	$5\frac{1}{4}$	$6\frac{1}{8}$	24·5	21
1	6	7	28	24
$1\frac{1}{8}$	$6\frac{3}{4}$	$7\frac{7}{8}$	31·5	27
$1\frac{1}{4}$	$7\frac{1}{2}$	$8\frac{3}{4}$	35	30
$1\frac{3}{8}$	$8\frac{1}{4}$	$9\frac{5}{8}$	38·5	33
$1\frac{1}{2}$	9	$10\frac{1}{2}$	42	36

The exact shape of a bend in a rod will obviously depend on the actual stress in the rod and on the permissible compressive stress in the concrete. The stress in the rod must be found, by calculation, from the external forces.

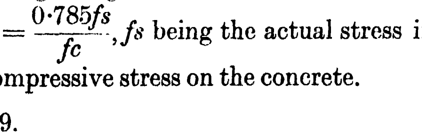


Fig. 180

The radius r at the beginning of such a bend should not be less than xd as shown in Fig. 180, in which $x = \frac{0{\cdot}785 fs}{fc}$, fs being the actual stress in the steel and fc being the permissible direct compressive stress on the concrete.

By-law 113.—See p. 129.

By-law 114.—See p. 129.

By-law 115.—See p. 122.

By-law 116.—See p. 123.

By-law 117.—See p. 123.

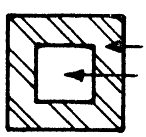
By-law 118.—See p. 123.

CHAPTER 10

PART VII. CHIMNEY-SHAFTS. BY-LAWS 119 to 131

The Construction of Chimneys and Flues.—In the by-laws the requirements are given only for chimney-shafts built of brickwork; but a footnote to the title of Part VII states that chimney-shafts to which the requirements of the by-laws are not applicable can still be constructed with the approval of the Council under the provisions of Part VII of the London Building Act, 1930 (Special Structures). Thus chimney-shafts of concrete, of reinforced concrete, steel or any other structural material approved by and used in the manner permitted by the Council can be adopted. Some notes on the uses of these other materials are given at the end of this Part, i.e. after By-law 131.

By-law 119.—A chimney-shaft is defined as the material surrounding a vertical flue which is greater than 80 sq. in. in area and through which smoke or other products of combustion pass to reach the open air.



Chimney shaft. Material surrounding flue.
Vertical duct. Not less than 80 sq in. in area through which smoke or other products of combustion pass to reach the open air.

FIG. 181

A chimney-shaft may be either free standing or bonded to a building. In the former case it must comply with By-laws 120 to 131. In the latter case, that is to say, when it is within or when it forms part of a building, it must be constructed to the satisfaction of the district surveyor, but

(*a*) the standard of stability must not be less than that required by By-laws 120 to 131. By-laws 120 to 123 are the equivalent of By-laws 27, 30, 32, 34 and 35, and must always be satisfied if a chimney-shaft is combined with a building. In many cases of such chimney-shafts, By-laws 124, 125, 126, 128 and 130 may be ignored, but the requirements of By-laws 127, 129 and 131 must always be observed;

and

(*b*) proper precautions are to be taken to prevent damage to the building through heat or through corrosion of structural steel.

In the majority of chimney-shafts in domestic buildings the 9 in. of brickwork required round the flues will afford sufficient protection for all adjacent structural materials, including steelwork. In special cases, as, for instance, for furnaces, it is advisable to line the inside of the flue, for a certain height, with fireclay bricks or other similar materials.

By-law 120. Site Treatment and Filling.—" Before the construction of a chimney-shaft is commenced:

" (1) The site of such shaft shall be cleared of all such material as in the opinion of the district surveyor would, if not removed, affect adversely the stability of such shaft or of any part thereof; and

"(2) all excavations, voids or cavities in the said site shall be filled or otherwise treated as the district surveyor may require for the purpose of ensuring the stability of such shaft."

Filling shall consist of material which has the district surveyor's approval as being suitable ; plain concrete may be used, provided that it is not inferior to concrete of designation VII in By-law 14. For permitte ıessures see By-law 34.

By-law 121. Foundation Bearing Pressures.—" The pressure upon earth intended or likely to support or restrain a chimney-shaft shall be calculated to the satisfaction of the district surveyor ; and the intensity of such pressure shall not exceed that allowed by the district surveyor.

" The builder shall furnish all such evidence as the district surveyor may require in order that he may decide the intensity of such pressure."

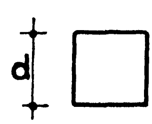
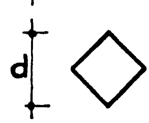
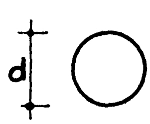
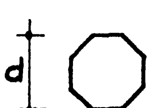
An indication of reasonable bearing pressures is given in Table 11 under By-law 30.

By-law 122. Pressures on Concrete Filling.—Where plain concrete is used for filling, as required by By-law 120, the pressure due to all loads coming on the filling from above must be calculated and such calculated stress must not exceed the value permitted for the appropriate concrete in By-law 34, Table 12.

The angle of dispersion of load through such plain concrete filling must not be taken as being greater than 45° measured from the vertical, and except for the tensile components of the shear such concrete must not be considered to resist any tensile forces.

By-law 123.—Concrete Foundations.—All the loads of a chimney-shaft must be calculated, and, accordingly, concrete foundations which must be provided to transmit

TABLE 40

Shape of Shaft	Direction of Wind	Coefficient α
	←	1·0
	←	0·5
	←	0·67
	←	0·70
	←	0·75

these loads to the ground must conform to the requirements of By-law 33. In the London County Council publication of the by-laws it is stated that such concrete must comply with By-law 32; but whilst it seems quite clear that By-law 33 is intended and that there is a printers' error, attention is formally drawn to the matter here. The requirements for plain and reinforced concrete when used as such foundations are given in the explanation of By-law 33.

The forces at the foundations of a chimney-shaft due to the wind loads on the shaft are very considerable, in the majority of cases, and particularly in free-standing shafts. The wind load should be taken as 25 lb. per sq. ft. on the total exposed area, but the shape of the shaft and its inclination to the wind will have a considerable influence on the actual wind load imposed. In the calculations it is easier to modify the actual exposed width than the intensity of the wind pressure, and therefore, if the height of a shaft is h and if the average exposed width is d, the total force of the wind on the shaft will be αdh 25 lb., in which α is the coefficient by which the exposed width is modified to allow for the shape and inclination of the shaft. In Table 40 suggested values of α are given for common shapes.

By-law 124. Shape.—The cross-sectional shape of a chimney-shaft may be:

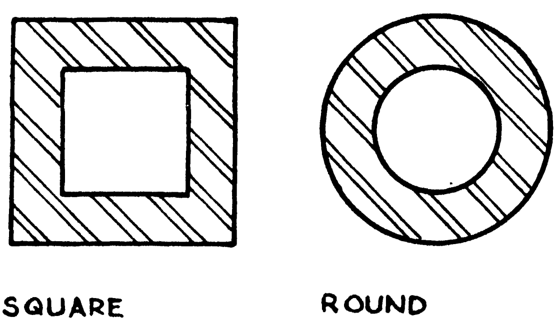


Fig. 182

or that of:

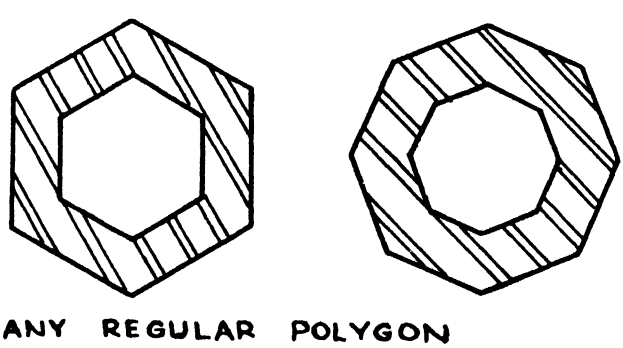


Fig. 183

By-law 125. Chimney-shafts to be of Brickwork and Mortar.—A chimney-shaft and its footings must be constructed of brickwork, the bricks and the mortar both being of a suitable nature. The shaft must have a batter of at least $1\frac{1}{4}$ inches for every 10 ft. of height.

Fig. 184 is purely diagrammatic.

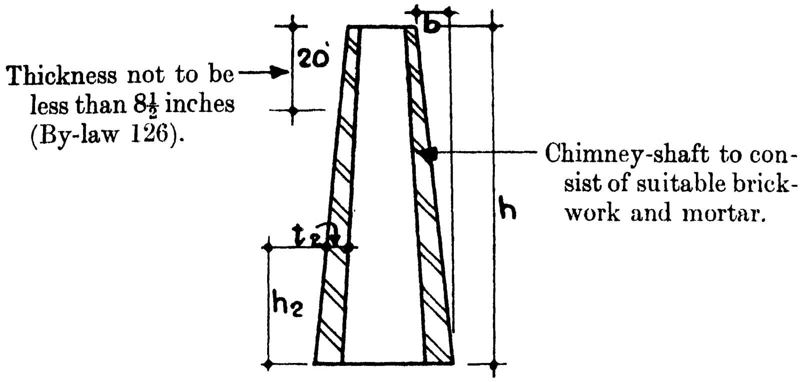


FIG. 184

b must not be less than $\dfrac{h}{96}$.

By-law 126. Thickness of Brickwork.—The thickness at any height of the brickwork forming a chimney-shaft must not be less than that given in Table 41 for that appropriate height.

TABLE 41

Chimney Height, ft.	Thickness in Inches at Various Heights above the Base							
	0–20 ft.	20–40 ft.	40–60 ft.	60–80 ft.	80–100 ft.	100–120 ft.	120–140 ft.	140–160 ft.
20	8½							
40	13	8½						
60	17½	13	8½					
80	22	17½	13	8½				
100	26½	22	17½	13	8½			
120	31	26½	22	17½	13	8½		
140	35½	31	26½	22	17½	13	8½	
160	40	35½	31	26½	22	17½	13	8½

Alternatively, in Fig. 184, if t_2 is the thickness at a height h_2, then t_2 is given by

$$t_2 = 8\tfrac{1}{2}\text{ in.} + \left(\frac{h - h_2}{20} - 1\right) 4\tfrac{1}{2}\text{ in.} \qquad (20)$$

in which the expression in brackets, if it is not already an exact whole number, must always be taken as the higher adjacent whole number.

Example: Determine the thickness of the brickwork of a chimney stack 110 ft. high at a level 65 ft. above the base.

$$h = 110,\ h_2 = 65$$

therefore

$$t_2 = 8\tfrac{1}{2}\text{ in.} + \left(\frac{110 - 65}{20} - 1\right) 4\tfrac{1}{2}\text{ in.}$$

$$= 8\tfrac{1}{2}\text{ in.} + (1{\cdot}25)4\tfrac{1}{2}\text{ in.}$$

which must be taken as

$$8\tfrac{1}{2}\text{ in.} + (2 \times 4\tfrac{1}{2}\text{ in.})$$

$$= 17\tfrac{1}{2}\text{ in.}$$

By-law 127. Projections from Normal Brickwork.—" Any cap, cornice, pedestal, plinth, string course, or other variation from plain brickwork in a chimney-shaft shall be provided as additional to the thickness of brickwork required for compliance with By-law 126 and shall be of proper construction, stability and security."

It may be necessary, according to the circumstances, to anchor such projecting cornices, etc., by means of external metal bands or by reinforcement within the brickwork, and all bands or reinforcements must be protected against every source of corrosion or damage.

By-law 128. Footings.—Brick footings must be provided continuously round the base of chimney-shafts and must project by regular off-sets to distances on each side of not less than the thickness of the brickwork forming the shaft. The space enclosed by the footings must be filled in solid as the work proceeds.

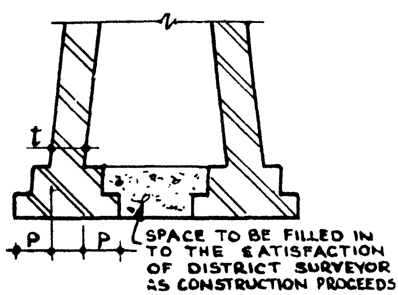


FIG. 185

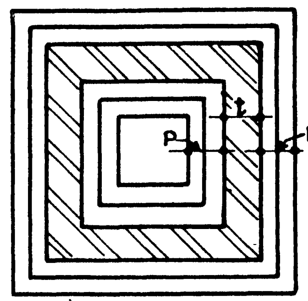


FIG. 186

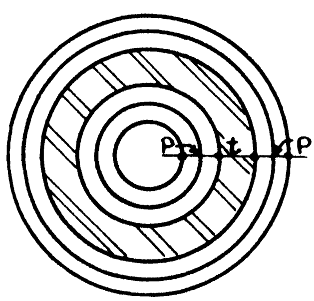


FIG. 187

In Figs. 185, 186 and 187 :

p must not be less than t.

By-law 129. Protection of Metal.—" Where metal is used in connection with the construction of a chimney-shaft or the footings thereof, proper protection shall be provided to prevent damage to such metal which, in the opinion of the district surveyor, might affect adversely the stability of such shaft."

By-law 130. Height of Chimney-shaft and Width of Base.—The height of a square chimney-shaft must not be greater than ten times the width of its base, and the height of a circular or regular polygonal shaft must not be greater than twelve times the diameter or the least width, respectively.

The height is to be taken as the distance from the base to the top.

In Figs. 188 to 191 :

h = height
w = width governing the height.

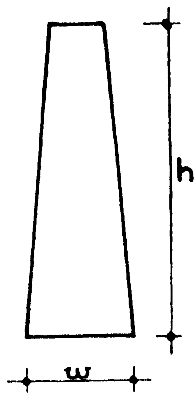


FIG. 188

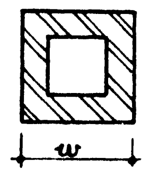


FIG. 189

h must not be greater than $10w$.

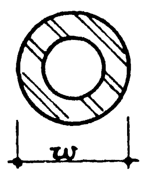


FIG. 190

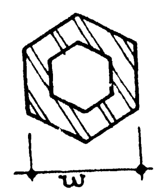
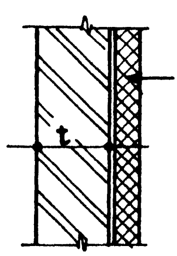


FIG. 191

h must not be greater than $12w$.

By-law 131. Internal Lining.—" Any internal lining in a chimney-shaft shall be provided as additional to and independent of the thickness of the enclosing brickwork, and shall not be bonded with such brickwork."



Internal linings must not be included in the required thickness " t " and they must be independent of and not bonded to the chimney-shaft.

FIG. 192

Notes on Chimney-shafts not Covered by the By-laws

When chimney-shafts are constructed from concrete, reinforced concrete, steel or such materials other than brickwork, they are to be considered as " special structures," and must, therefore, be approved by the London County Council.

Concrete used for chimney-shafts should not be inferior to concrete of designation IV (By-law 14, Table 8), and when the dimensions of the concrete comply with By-laws 126 and 130 the concrete shaft will be accepted. In all other cases a calculation must be provided showing that the combined stresses of the compressions due to the dead load and bending moment caused by the wind are nowhere greater than the permitted limits.

The permitted combined compression stress can be taken from By-law 60, Table 19, and a tensile stress equal to 10 per cent. of the compressive stress is likely to be permitted.

A reinforced concrete chimney can be constructed to a considerably greater height, in relation to the base width, than that set out in By-law 130. The dimensions of such chimneys are to be determined entirely by calculation. The stresses can be taken as those given in By-laws 99 to 102, but without any increases as allowed by By-law 103.

With regard to buckling, a shaft is to be considered as a column of one storey, rigidly fixed at the level at which the section is to be investigated, and unrestrained in both position and direction at the point where the load or the resultant of the loads is applied. Thus the length of the shaft for the purpose of the slenderness ratio is to be taken as twice the distance between the section under investigation and the point where the resultant load is applied. However, because all wind and vertical loads are generally applied uniformly through the height of the shaft in most cases, it is correct to take the slenderness ratio height as the distance between the section under investigation and the top of the shaft.

The fact that the maximum permitted slenderness ratio is 120 limits the height to about 40 times the diameter at the base if a circular shape is used, and to about 48 times the width if a square shape is used.

Every reinforced concrete chimney should have outer and inner reinforcement, both vertically and horizontally.

The volume of the horizontal reinforcement should not be less than 0·8 per cent. of the volume of the concrete. The vertical reinforcement can be determined from the formulæ given in Chapter 22.

The inside cover on reinforcement should not be less than 2 in., but if the heat or chemical content of the gases should have any dangerous influence, a fireclay or other lining should be provided. When such a lining is used the inside concrete cover to the reinforcement can be reduced to 1 in. on horizontal and ½ in. on vertical bars, the same cover being kept to under all circumstances on the outside. According to By-law 129 the ultimate decision on protection of metal lies with the district surveyor in spite of the fact that it is the Council who must approve the construction.

Whilst By-law 125 has no direct bearing on the construction of reinforced concrete chimneys, a certain batter is to be recommended, and this should be not less than 1 in. in 12 ft. 6 in.

By-law 131 will apply to reinforced concrete chimneys in fact, if not directly, in that only the concrete area, but not any lining, is to be taken into account for statical purposes.

Steel chimneys also can be used. They are especially useful on the tops of buildings such as factories, etc. They are to be lined inside by proper materials, as, for example, fire-clay not less than 4 in. thick. The same rules obtain with regard to the slenderness ratio as for reinforced concrete chimneys, unless, however, such steel chimneys are properly guyed at the top. It is suggested that the ratios of height to width of base be kept the same as for the reinforced concrete chimney, namely 40 times the diameter at the base of a circular-shaped shaft and 48 times the width of a square-shaped shaft.

The plates from which the wall is constructed are to be properly stiffened.

CHAPTER 11

PART VIII. MISCELLANEOUS. BY-LAWS 132 TO 151

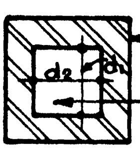
THERE are twenty by-laws in this part (By-laws 132 to 151), and they relate principally to the precautions which must be made in the construction of a building against possible damage by fire, and they specify where incombustible materials are to be used. In addition, the requirements for habitable rooms, such as dimensions and windows, are given. The actual order of the by-laws has been re-arranged here with the object of bringing together into groups all those by-laws relating to the same subject; but where this has been done a footnote has been made to draw attention to the fact.

By-law 132. Chimneys and Flues.—In this by-law there are ten clauses (2 to 11 inclusive) defining the conditions which must be observed in the construction and arrangement of chimneys, flues (which are defined in clause 1) and fireplaces. They refer to all chimneys, flues and to constructions in connection with them, built of bricks, blocks or of concrete. Chimneys of which the sizes of the ducts bring them under the class of chimney-shafts as defined in Part VII of the by-laws must also be in accordance with the requirements of By-laws 119 to 131, unless Part VII of the Act should apply to them (see note on p. 148). Where flues are constructed from other materials than those specified above they are classified as pipes, and in such cases By-laws 134 to 136 apply.

Chimneys and flues used for a furnace, close fire or for a range or other cooking apparatus used for the purpose of a trade or gain must comply with the requirements of By-law 133 (3) in addition to those of clauses 2 to 11 of this By-law.

Flues for open gas-fires are exempted from the requirements of clauses 2 (*a*) (i), *d*, *e*, *f*, *g* and (7), but By-law 134 (1–5) must be observed. Clauses (3), (4), (5) are to be observed in the case of gas fires where such a fireplace construction is used.

Clause 1.—Clauses 2 to 11 cover chimneys of brick, blocks or concrete. If the ducts of such chimneys do not exceed 80 sq. in. in area in the clear, they are called flues.



Chimney of bricks, blocks or concrete.

Duct not greater than 80 sq. in. through which smoke or other products of combustion pass to reach the open air, called "flue"

FIG. 193

Clause 2: *Flues and Chimneys in, against or detached from Walls.*—If the following ten conditions, (*a*) to (*j*), are observed, chimneys may be constructed to be free standing or to be in contact with walls and flues may be included in walls. The conditions are that:

(*a*) *Thickness and height of chimneys.*—(i) Every flue must be surrounded by not less than 4 in. of solid bricks or blocks or of concrete. The bricks or blocks must be properly bonded, bedded and jointed in mortar, and any concrete which is used, but which is not in the form of blocks, must be cast *in situ*. Metal may be used in conjunction with these materials, but not in the case of flues in party walls.

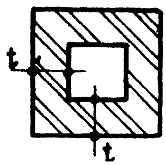


FIG. 194

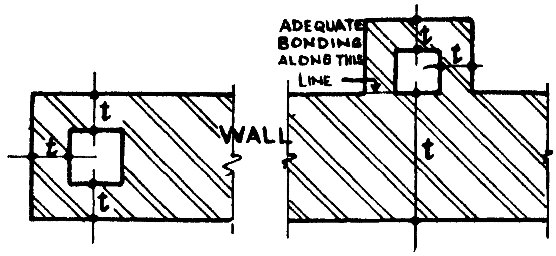
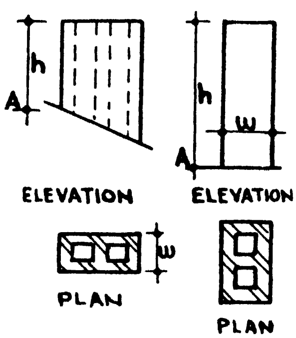


FIG. 195

t must be less than 4 in.

The foregoing provisions are for normal cases; but if other by-laws specify different conditions for special cases, as for instance for flues for gas fumes, then the former requirements need not be observed. See Clause 5.

(ii) When two or more chimneys are combined they must be carried up to the following heights above the adjoining roof, flat or gutter:



w = the less dimension of the stack at the lower level from which h is measured.

FIG. 196

For a flue of a gas fire or a gas-fired appliance:

- h must not be less than 18 in.
- h must not be greater than $6w$ (unless the chimney or stack is braced against overturning).

For all other flues:

- h must not be less than 3 ft.
- h must not be greater than $6w$ (unless the chimney or stack is braced against overturning).

Owing to the wide range of special materials which are available for gas flues it is not possible to give a general indication of the height at which bracing is first required; but for the common 9-in. flue consisting of $4\frac{1}{2}$ in. brickwork, an unbraced height up to 9 ft. may be used. Where it is necessary, bracing can usually be provided economically by metal bands, fastened round the stack, to which are attached guy rods or bars. If such bracing is used on only one side, it should consist of a stiff member capable of resisting a certain amount of compression without buckling. If rods are used on opposite sides they can be of very much smaller section, having to resist only tension. Obviously all struts or tee members used to brace a chimney must be securely anchored to the rest of the structure, and if possible they should be attached to walls running in the same direction as the ties.

(*b*) *Supports for chimney.*—Every chimney must be carried in one of the three following manners :

(i) on foundations which are adequate and which have the approval of the district surveyor ;

(ii) on corbels of brick, stone or other incombustible material of which the projection is not greater than the thickness of the wall below ; or

Corbel of brick stone or other incombustible material.

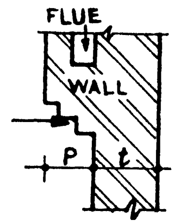


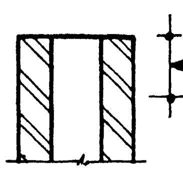
FIG. 197

p must not be greater than t

(iii) on structural steelwork or reinforced concrete constructed in accordance with the By-laws.

(*c*) *Cement mortar at the top.*—If the chimney or stack is built of bricks or blocks (which must be properly bonded, bedded and jointed with mortar as required by condition (*a*) above), the mortar used for the top six courses must be cement mortar.

Brick or block chimney :



Cement mortar must be used for the top six courses.

FIG. 198

(*d*) *Minimum interior dimension.*—The smaller dimension of any flue must not be less than $7\frac{1}{2}$ in.

In Fig. 193 :

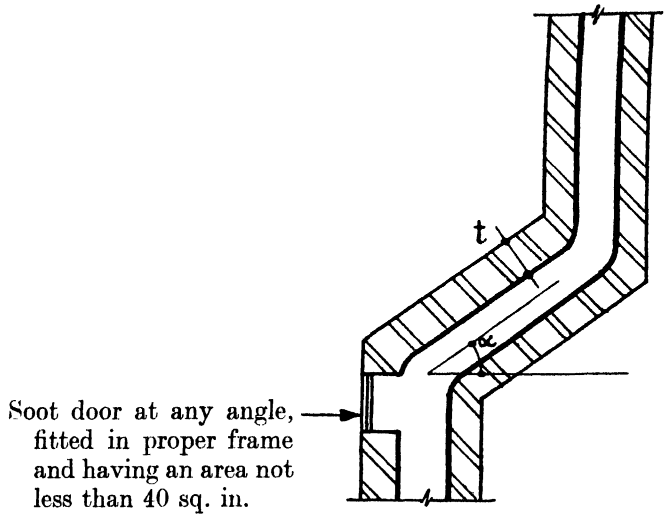
$d_1 \times d_2$ must not be greater than 80 sq. in.,
d_1 must not be less than $7\frac{1}{2}$ in., and
d_2 must not be less than $7\frac{1}{2}$ in.

This provision will actually cover the majority of cases ; but if other By-laws specify different conditions for special cases, as for instance for flues for gas fumes, then this requirement need not be observed.

(*e*) *Inclination of flues.*—All flues for gas fumes and all other flues having proper soot doors and suitably rounded interior angles may be constructed at any inclination to the horizontal.

Thus in Fig. 199, α can have any value for all flues, and in Fig. 200, α can have any value for flues for gas fires or other gas-fired appliances.

The soot door must be fitted in a proper frame of incombustible materials and have an area not less than 40 sq. in. It can be arranged at any suitable angle, but obviously the best position and angle are those which will give the greatest facilities for removing the soot from the inclined length of flue. See Clauses (8*b*) and (9). For the thickness of material on the upper side of flues inclined at an angle less than 45° see condition (*f*).

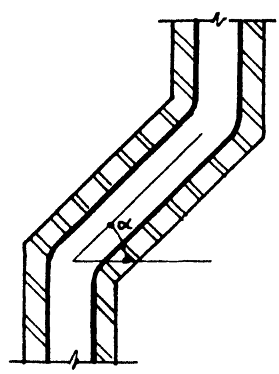


VERTICAL SECTION

FIG. 199

In Fig, 199 ; for all flues : α can have any value.

If a soot door conforming to the preceding requirements is not fitted, the inclination of any flue, other than one for gas fumes, must not be less than 45° to the horizontal. The interior angles must be properly rounded.



VERTICAL SECTION

FIG. 200

In Fig. 200, for flues for gas fumes : α can have any value.

For all other flues : α must not be less than 45°.

(*f*) *Thickness of inclined flues.*—Except in the case of flues for gas fumes, the thickness of solid material on the upper side of a flue, inclined at an angle of less than 45°, must not be less than 8½ in.

Thus in Fig. 199, when α is less than 45° for any flue other than one for gas fumes, *t* must not be less than 8½ in.

(*g*) *Interior treatment.*—Unless a chimney other than one for gas fumes, is lined with fireclay, stoneware or other similar material to a thickness not less than ⅝ in., its interior must be rendered or pargeted. All such linings must be placed in position as the construction of the chimney proceeds, and any spandrel angles or voids which may occur, as for instance between a stonework lining and the stack, must be filled with brickwork, mortar or other incombustible material.

(*h*) *Exterior treatment.*—" Where a chimney passes through any floor or roof within 9 in. of any combustible material or behind or against any woodwork, the outside of such chimney shall be properly rendered or pargeted."

(*i*) *Protection of adjacent steelwork, etc.*—" Where a chimney is in proximity to any constructional steelwork or reinforced concrete in a building, adequate precautions shall be taken to ensure that the effects of heat or corrosion arising through the proximity of such chimney shall not affect injuriously such steelwork or reinforced concrete."

Normally, the 4 in. of material which is required to surround the flue is in itself sufficient protection ; but if for any reason this is interrupted where the structural steel or reinforced concrete occurs, it is considered that at least the same amount of cover, which can be provided by the normal steel casing or by concrete at the sides and ends of the reinforcement, should be provided. In special cases where the steel is at the lower end of the flue and near to the source of heat a special fireclay lining to the flue or an external application of an incombustible insulating material will eliminate the risk of damage.

(*j*) *Separate flue for each fire.*—With the two exceptions, (i) and (ii) which follow, every fire and every heating apparatus which requires ducts for the removal of fumes or currents or air must be provided with a separate and independent flue. However, if :

(i) in a building of which the construction was started or completed before 1 January, 1938, it is impracticable to provide a separate flue for any new fire or heating apparatus and if the district surveyor approves, then for that new fire, etc., a separate flue need not be provided and connection can be made to a suitable existing flue ; or

(ii) if there are more than one fire or heating apparatus within the same room or within the same enclosed space, then these fires, etc., can have a common flue. In certain cases, for example when the fireplaces, etc., are on different sides of the room, it is obviously impracticable to use the same flue, and therefore separate flues would be provided ; but in large rooms or in lounges where fires can be arranged back to back, advantage can be taken of this provision for a common flue.

Clause 3 : *Support for Chimneys over Fireplace Openings.*—The weight of the material enclosing the chimney over every fireplace opening must be transferred to the jambs on each side by one of the three following methods :

(*a*) a brick or stone arch ; (Fig. 201)
(*b*) a steel or reinforced concrete lintel ; (Fig. 202) or
(*c*) a bar of wrought iron. (Fig. 203)

The minimum width of jamb on either side is specified in Clause 4 as $8\frac{1}{2}$ in. ; but if either actual width is less than $17\frac{1}{2}$ in., and if at the same time the material

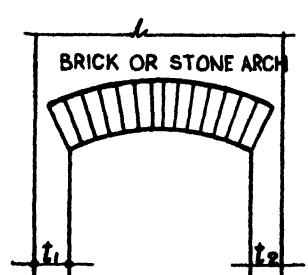


FIG. 201

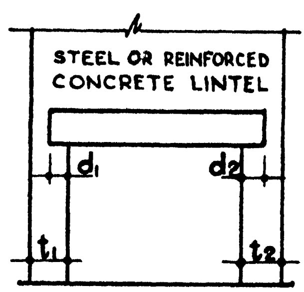


FIG. 202

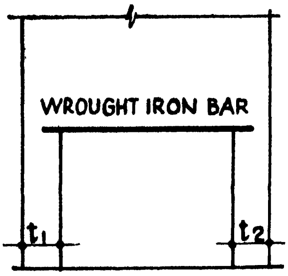


FIG. 203

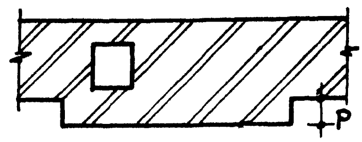


FIG. 204

enclosing the chimney over the fireplace projects more than 4 in. from the face of the wall, then, except in case (*b*) above, the two jambs must be tied together by means of an iron bar or by several iron bars, which must project into each jamb at least 8½ in. and which must, beyond this point, be turned up and down to obtain a secure hold on the jamb. The bar in case (*c*) above may be used for this purpose.

Thus in Figs. 201, 202, 203 and 204 :

if t_1 is less than 17½ in., or if t_2 is less than 17½ in.,
and at the same time if p is greater than 4 in.,
then, iron ties must be introduced as in Fig. 205 such that :

d_3 must not be less than 8½ in. and d_4 must not be less than 8½ in.

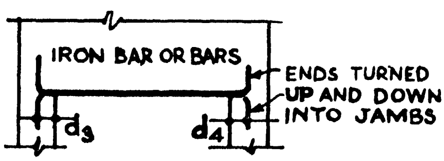


FIG. 205

Also in Fig. 202 and 204 :

if t_1 is less than 17½ in. or if t_2 is less than 17½ in.
and if at the same time
p is greater than 4 in.
and if at the same time d_1 is less than 4 in. or d_2 is less than 4 in.
then (but only under the immediately preceding conditions) iron ties must be introduced as in Fig. 205 such that :

d_3 must not be less than 8½ in. and d_4 must not be less than 8½ in.

Clause 4 : *Width of Fireplace Jambs.*—" The jambs of every fireplace opening shall be at least 8½ in. wide on each side."

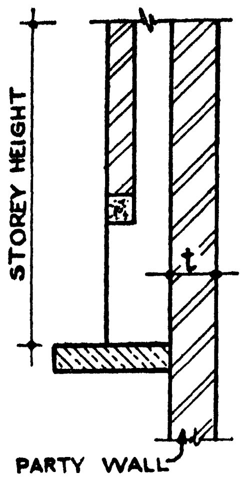
Thus in Figs. 201, 202 and 203 :

t_1 must not be less than 8½ in. and
t_2 must not be less than 8½ in.

Clause 5 : *Thickness of Party Wall behind Fireplace.*—Where a flue in a party wall is not back to back with another flue, the thickness of solid material behind the fireplace must not be less than 8½ in. and this thickness must be carried up for the full height from the fireplace to the level of the next floor above.

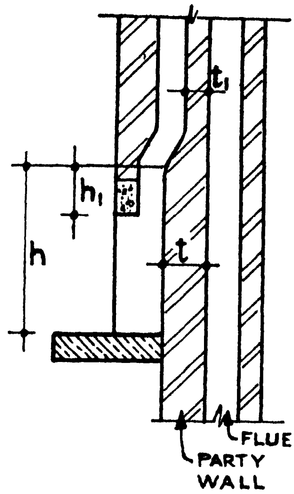
In Fig. 206 : t must not be less than 8½ in. throughout the storey height.

Where a flue in a party wall is back to back with another flue, the thickness of solid material behind the fireplace must not be less than 8½ in., and this thickness must be carried up from the level of the hearth to a height which is at least 12 in. above the level of the fireplace opening.



Flue not back to back with another in a party wall.

FIG. 206



Flue back to back with another in a party wall.

FIG. 207

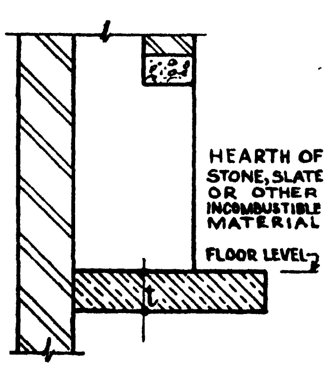
In Fig. 207 : t must not be less than $8\frac{1}{2}$ in. throughout the height h such that h_1 is not less than 12 in.

Clause 6 : *Conversion of External Walls into Party Walls.*—" Where any external wall is converted into a party wall, all chimneys in such wall shall be made to comply with the provisions of this by-law."

Clause 7 : *Hearths.*

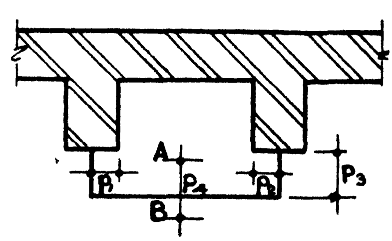
(*a*) *Open fireplaces.*—A hearth or slab of stone, slate or other incombustible material must be laid level with the floor in front of every fireplace opening. Any bedding of brick or other incombustible material on which the hearth is laid may be included in the required thickness, which must nowhere be less than 6 in. of solid material. Normally the slab must extend for a distance not less than 6 in. past either end of the opening and for a distance of not less than 18 in. in front.

If the fire will project in front of the opening, for example, to a line A in Fig. 209, then the projection P_3 from the opening must be increased until the hearth extends to a line B such that P_4 is not less than 18 in.



VERTICAL SECTION

FIG. 208



HORIZONTAL SECTION

FIG. 209

t must not be less than 6 in., but t may include any incombustible material on which the hearth is bedded.

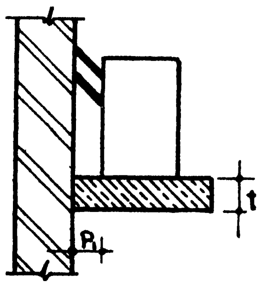
P_1 must not be less than 6 in.
P_2 must not be less than 6 in.
P_3 must not be less than 18 in.

Closed stoves.—*A hearth or slab of stone, slate or other incombustible material must be laid level with the floor beneath every closed stove which*

(i) has been designed, constructed and is used for the slow combustion only of anthracite or other solid smokeless fuel; and which

(ii) is used only for the direct heating of the room, hall or passage in which it is located.

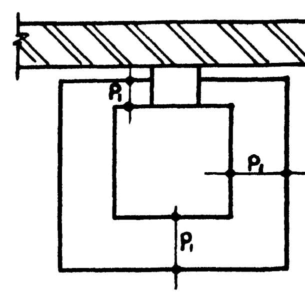
Any bedding of brick or other incombustible material on which the hearth is laid may be included in the required thickness, which must nowhere be less than 6 in. of solid material. Normally the slab must extend for a distance not less than 12 in. from the stove round all sides.



VERTICAL SECTION

FIG. 210

t must not be less than 6 in., but *t* may include any incombustible materials on which the hearth is bedded.

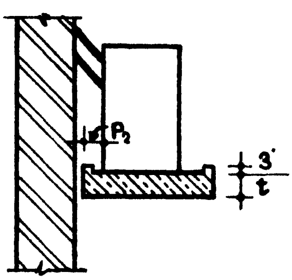


PLAN

FIG. 211

P_1 must not be less than 12 in.

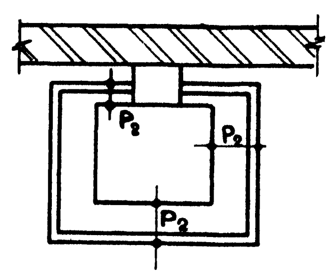
Alternatively, if the hearth is fitted with a fixed curb of incombustible material not less than 3 in. high, the projection on all sides may be reduced to a minimum of 6 in.



VERTICAL SECTION

FIG. 212

t must not be less than 6 in., but *t* may include any incombustible materials on which the hearth is bedded.



PLAN

FIG. 213

P_2 must not be less than 6 in.

If the fire will project in front of the stove the projection of the hearth must be increased to such an extent that in the two preceding cases it is still 12 in. or 6 in., respectively, in front of the fire. This increased projection can be of a thickness reduced to a minimum of 2 in., provided that immediately underneath the stove there is a normal hearth conforming to the preceding requirements.

(*b*) *Supports for hearth.*—" The said hearth or slab shall be laid wholly upon stone or iron bearers or upon brick trimmers or other incombustible material or upon timber fillets not less than 1 in. square in section secured along their whole length to the sides of the floor trimming joists, but in the lowest storey it may be bedded on the concrete covering the site or on solid incombustible materials placed on that concrete."

Clause 8 : *Reasons for cutting into Chimney Jamb or Breast.*—" No part of any chimney, including any jamb or breast, shall be cut into except for the purpose of repair or of any of the following operations :

(*a*) Letting in or removing or altering flue pipes or funnels for the conveyance of smoke, products of combustion from a gas fire or gas-fired appliance, hot air or steam, or letting in, removing or altering smoke jacks.

(*b*) Forming or opening for a soot door, such opening to be fitted with a close iron door and frame.

(*c*) Forming an opening for the insertion of a ventilating valve.

(*d*) Forming an opening for the purpose of complying with the provisions of paragraph (2) (*j*) of this by-law."

Clause 9 : *Soot Doors and Ventilating Valves.*—" All soot doors and frames shall be at least 15 in. and all ventilating valves shall be at least 12 in. distant from any woodwork or other combustible material."

Clause 10 : *Proximity of Woodwork to Flues, etc.*—(*a*) Except for wooden plugs, no woodwork may be nearer than 9 in. to the inside of any flue or fireplace opening.

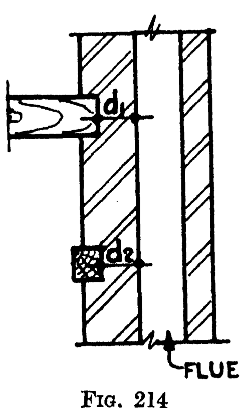


FIG. 214

d_1 must not be less than 9 in.
d_2 must not be less than 9 in.
See Clause 10 (*a*) above.

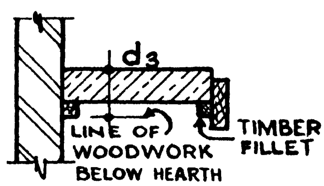


FIG. 215

d_3 must not be less than 10 in.
See Clause 10 (*b*), p. 164.

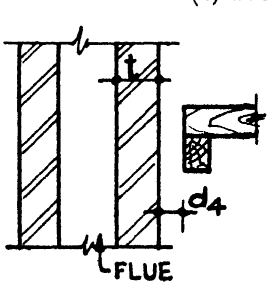


FIG. 216

If t is less than $8\frac{1}{2}$ in., d_4 must not be less than 2 in.
See Clause 10 (*c*), p. 164.

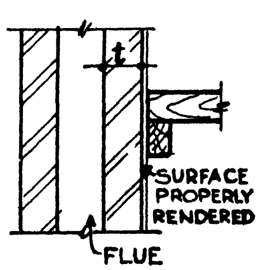


FIG. 217

Unless :
when t is less than $8\frac{1}{2}$ in. the outside of the chimney is properly rendered.
See Clause 10 (*c*), p. 164.

(*b*) Except for wooden plugs and for any timber fillets supporting the hearth of a fireplace opening no woodwork below may be nearer than 10 in. to the upper surface of the hearth. See Fig. 215, p. 163.

(*c*) If the material of which a chimney is constructed is less than 8½ in. thick and if at the same time the outside is not properly rendered, then no woodwork except wooden plugs may be nearer than 2 in. to the outer face of the chimney. See Figs. 216 and 217, p. 163.

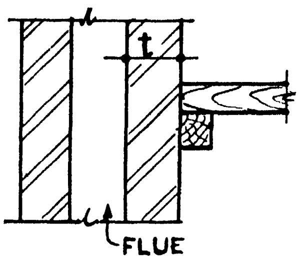


FIG. 218

When t is not less than 8½ in. the outside need not be rendered.
See Clause 10 (*c*) above.

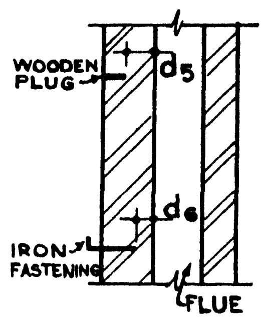


FIG. 219

d_5 must not be less than 6 in.
d_6 must not be less than 2 in.
See Clause 11 below.

Clause 11 : *Wooden and Iron Plugs in Chimneys.*—" Wooden plugs shall not be driven into any wall or chimney nearer than 6 in., and iron holdfasts or other iron fastenings shall not be driven nearer than 2 in. to any flue or to the inside of any fireplace opening."

By-law 133. Ovens, Coppers, Boilers, Stoves. *Clause* 1 : *Open-base Electric and Gas Ovens.*—" Under every open-base oven heated by gas or electricity which has a flame or material heated to incandescence within 9 in. of a floor, a slab of incombustible material of a thickness of not less than 2 in. shall be provided so as to extend for a distance of not less than 6 in. on all sides of such oven or up to any adjacent wall or walls whichever be the lesser distance. Where the flame or material heated to incandescence of such oven is 9 or more inches from the floor, the slab of material required need only be ½ in. in thickness."

Clause 2 : *Other Ovens.*—" Under every other oven, copper, steam boiler or stove a slab of stone, slate, concrete, bricks or tiles shall be provided so as to extend for a distance of at least eighteen inches on all sides of such oven, copper, steam boiler or stove or up to an adjacent wall or walls, whichever be the lesser distance. Such slab shall be solid for a thickness in every part of not less than 6 in., including any bedding consisting of incombustible materials. Provided that the slab may be not less than 2 in. thick if it is constructed of incombustible material and so as to have a resistance to the transmission of heat not inferior to that of a slab which would otherwise be required by this paragraph. Such slab shall be deemed to be part of the floor unless it is superimposed on such floor.

" Provided that :

" (*a*) In the case of an oven (other than an open base oven), copper, steam boiler or stove heated by gas or electricity, the thickness of required material may be as follows :

" Where the temperature at any point at floor level does not exceed : 270° F., 2 in. and shall extend not less than 9 in. beyond such oven, copper, steam boiler or stove or up to any adjacent wall or walls, whichever be the lesser distance.

" 170° F., ½ in. and shall extend not less than 6 in. beyond such oven, copper, steam boiler or stove or up to any adjacent wall or walls, whichever be the lesser distance.

" At temperatures of 150° F. or below, no slab need be provided.

" Any material other than the required slab which may be interposed between the floor and the flame or material heated to incandescence for screening such flame or material and which may have the effect of reducing the temperature at floor level shall be of incombustible material and shall be fixed permanently in position.

" (*b*) In the case of a close slow-combustion domestic oven, copper, steam boiler or stove designed, constructed and used solely for the burning of anthracite or other solid smokeless fuel and situate in a position not provided for by By-law 132 (7), the requirements of this by-law shall be deemed to be complied with :

" (i) if such slab is extended around the said domestic oven, copper, steam boiler or stove in every direction to a distance of at least 12 in. or up to any adjacent wall or walls, whichever be the lesser distance ; or

" (ii) if such slab is extended around the said domestic oven, copper, steam boiler or stove in every direction to a distance of at least 6 in. and is surrounded with a fixed curb of incombustible material at least 3 in. in height.

The requirements of this by-law as to the provision of a slab shall not apply if the floor under such oven, copper, steam boiler or stove is of such material and of such thickness and extent as would otherwise be required by this by-law."

Clause 3 : *Pipes, Stoves, Ranges, etc., for Trade Purposes.*—" The provisions of By-law 132 shall apply to chimneys and flues adapted or used for any oven, furnace boiler or close fire or for a range or cooking apparatus used for the purpose of trade or gain subject to the following modifications :

" (i) That a flue exceeding 80 sq. in. in area shall be enclosed by a chimney-shaft complying with the requirements of By-law 119 ; and

" (ii) That a flue not exceeding in area 80 sq. in. shall be surrounded with brickwork not less than 8½ in. in thickness from the floor on which such oven, boiler or close fire, range or cooking apparatus is situate :

" (*a*) to the level of the underside of the floor forming the ceiling of the room next above the room in which it is situate and where such floor is not of incombustible construction to the upper surface of such floor, and

" (*b*) when there is no such room above to the outer surface of the roof.

Provided that an existing flue may be adapted or used in connection with a close fire having an outlet exceeding 20 sq. in. but not exceeding 80 sq. in. in area if the flue is surrounded by brickwork not less than 4 in. in thickness and there is provided inside such brickwork and for the protection thereof a pipe of such thickness and of such incombustible material as the district surveyor may consider necessary for such height as the surrounding brickwork would otherwise be required to be 8½ in. in thickness."

Clause 4 : *Flues for Engines, Steam Boilers, etc.*—" A chimney shall not be used in connection with an internal combustion engine or the furnace of a steam boiler or hot air engine unless the chimney is at least 20 ft. in height measured from the ground level and is at least 20 ft. away from any other building which is a domestic building."

By-law 134. Gas Fires.—" The provisions of By-law 132 shall apply to chimneys and flues used or to be used in connection with ordinary open domestic gas fires subject to the following modifications :

" (1) The flue shall be surrounded with incombustible material not less than 1 in. in thickness exclusive of any plastering. The mortar or other bonding material shall be so finished as not to project into the flue.

" Any void in the material enclosing the flue may for the purpose of this by-law be considered to be incombustible material, subject to such void not being connected with the flue and to the district surveyor being satisfied in respect of stability. Subject to the flue with an incombustible adaptor fixed thereto being carried up to the roof space, it may be continued by a flue-pipe of incombustible material on the slope under the roof tiles to the ridge and thence ventilated to the open air by a ventilating cap on the ridge.

" (2) The cross-sectional area of the flue shall not be less than 20 sq. in., but shall in no case be less than the flue spigot of the gas fire. The flue shall be provided with a terminal cap as a precaution against down-draught unless the flue complies with the provisions of By-law 132.

" (3) In a building constructed before the coming into operation of this by-law a flue may, subject to the approval of the district surveyor being first obtained, deliver into an existing flue.

" (4) A hearth constructed of material conforming with the British Standard Specification No. 476—1932 for materials for hearths and not less than 2 in. thick and extending at least 6 in. beyond the front and sides of the fire or up to any adjacent wall or walls, whichever be the lesser distance shall be provided.

" Provided that in the case of a panel gas fire which has the lowest portion of the flame or incandescent material not less than 9 in. from the floor level and the heating elements properly protected to prevent any defective portion from falling on to the floor such hearth need not be provided.

" (5) A flue shall not be used in connection with any other fire unless it conforms with the requirements of these by-laws relating to flues and chimneys used in connection with such fires."

By-law 135. Gas Water Heaters.—" The following provisions shall have effect with regard to flue pipes used or to be used in connection with gas-heated geysers or other gas appliances of the rapid water-heating type (with the exception of small appliances of the rapid water-heating type used solely for the intermittent supply of hot water to sinks and lavatory basins) and the ventilation of rooms in which such appliances are fitted.

" (1) A flue pipe of incombustible material shall be provided to every geyser or other such appliance and shall be constructed so as to allow all products of combustion to flow away in conditions of normal weather.

" (2) There shall be provided in the room in which the geyser or other such appliance is situate a baffler capable of preventing interference with the combustion of the gas if conditions of abnormal weather temporarily prevent all products of combustion flowing away, and such baffler if not contained within the appliance shall be fitted close to such appliance and in no case more than 1 ft. away.

" (3) Subject to the provisions of sub-paragraph (*j*) of paragraph (2) of By-law 132, the flue pipe shall discharge either into a chimney not communicating with any other room or directly into the external air, in which case the flue pipe shall be fitted with a suitable terminal or outside windguard, or, where neither of the foregoing methods is reasonably practicable, into a space immediately below the roof which does not form part of any room and is itself freely ventilated to the external air.

" (4) The internal diameter of the flue pipe and the flue pipe connections of the baffler and terminal cap shall not be less than that of the flue spigot at the top of the geyser or such other appliance.

" (5) Every room in which a geyser or such other appliance is fitted shall be ventilated by means of a window capable of being opened."

By-law 136. Pipes for Smoke, Steam, etc.—" (1) (*a*) A pipe for conveying smoke or other products of combustion shall not, except as permitted by By-law 133 (3), be provided to act within the building as a flue except within the room in which the fire for which it acts as a flue is situate, but this provision shall not prevent such pipe being carried through an external wall of such room in which it is fitted direct to the open air.

" (*b*) A pipe for conveying smoke or other products of combustion shall be of cast iron conforming with the British Standard Specification (Cast-iron Spigot and Socket Flue and Smoke Pipes) No. 41—1908 or of mild steel plates not less than $\frac{3}{16}$ of an inch in thickness.

" Provided that in the case of a pipe used in connection with a gas-fired appliance such pipe may be of asbestos cement conforming with the British Standard Specification (Asbestos Cement Spigot and Socket Flue Pipes and Fittings for Gas-Fired Appliances) No. 567—1934.

" Provided also that pipes used in connection with domestic cooking ranges, gas fires or geysers and similar hot-water appliances may be constructed of sheet metal.

" Provided, further, that in the case of a pipe used in connection with a close slow-combustion domestic oven, copper, steam boiler or stove designed, constructed and used solely for the burning of anthracite or other solid smokeless fuel, such pipe may be constructed of asbestos cement to the satisfaction of the district surveyor.

" (2) A pipe for conveying smoke or other products of combustion heated air, steam or hot water shall not be fixed against any building on the face adjoining any street or public way.

" (3) A pipe for conveying smoke or products of combustion other than a pipe fitted to a gas fire or a gas-fired appliance shall not be fixed nearer than 9 in. to any combustible material.

" A pipe for conveying the products of combustion from a gas fire or gas-fired appliance shall not be fixed nearer than 4 in. to any combustible material. Provided that this requirement shall not apply if the pipe for conveying the products of combustion from a gas fire or gas-fired appliance where in proximity to combustible material is carried in a metal sleeve with a space of 1 in. between the sleeve and the pipe and such space is packed with asbestos packing or other suitable non-heat-conducting material.

" (4) A pipe for conveying heated air or steam shall not be fixed nearer than 6 in. to any combustible material.

" (5) A pipe conveying hot water shall not be placed nearer than 3 in. to any combustible material.

" (6) Every pipe provided for collecting or carrying off any vapour or fumes from apparatus used in any fried fish or other cook-shop, eating-house, restaurant, hotel or any like premises for cooking food or for any purpose in connection therewith, shall be of metal or asbestos cement and shall not be fixed nearer than 9 in. to any combustible material.

" (7) The restrictions imposed by this by-law with respect to the distance at which pipes for conveying heated air, hot water or steam may be placed from any combustible materials shall not apply in the case of pipes for conveying any of the following:

"(*a*) Heated air for the purpose of heating or ventilating a building.

"(*b*) Steam at a pressure not exceeding 15 lb. to the sq. in. above atmospheric pressure.

"(*c*) Hot water where provided with a free blow-off or with a temperature not exceeding 200° F.

"(8) A pipe coming within the foregoing provisions of this by-law shall not when used externally exceed 4 in. in diameter.

"(9) The restrictions contained in paragraph (1) (*a*) of this by-law shall not apply to a pipe or pipes fitted to a gas fire or gas-fired appliance if such pipes be enclosed with 3 in. of incombustible material conforming with the British Standard Specification (Definition of Fire-resistance, etc.) No. 476—1932, exclusive of any plaster, and the pipe or pipes are carried up 18 in. above the roof flat or gutter in the same manner as is required under By-law 132 (2) (*a*) for a chimney."

By-law 138.[1] **Floors over Furnaces or ovens, etc.**—"The floor immediately over or, if there be no floor, the roof over any room or enclosed space in which a furnace is fixed, and any floor or roof within 18 in. from the crown of an oven shall be constructed of incombustible materials. Provided that:

"(*a*) the foregoing provisions of this by-law shall not apply to the enclosed fire of an independent water boiler or to a kitchen range if the top of such boiler or range is not within 5 ft. of the ceiling of the room or enclosed space in which such boiler or range is placed and is not more than 3 ft. above the floor of such room or enclosed space;

"(*b*) in lieu of the incombustible floor or roof required by the first paragraph of this by-law there may be provided a false ceiling constructed of incombustible material not less than 3 in. thick supported or suspended by incombustible material."

By-law 140.[1] **Fireproofing in Buildings over Certain Sizes.**—"In every building which exceeds 25 squares in area or of which the cubical extent exceeds 125,000 cu. ft. which is constructed or adapted to be tenanted by different persons:

"(*a*) (i) The whole of the floors; and

(ii) All lobbies, corridors, passages, landings and stairs used in common by the tenants shall be constructed of fire-resisting materials and shall be carried by supports of fire-resisting materials. Provided that in the case of self-supporting flights of stairs constructed of reinforced concrete or steel no additional support shall be required if such support is not necessary for the purpose of stability.

"(*b*) All such lobbies, corridors, passages, landings and stairs shall be enclosed with terra-cotta, brick, concrete or other incombustible material not less than 3 in. thick. Provided that there may be constructed in any such enclosure such doorways as are necessary for communicating between the different parts of the building if such doorways be fitted with fire-resisting doors in frames constructed of fire-resisting materials.

"For the purposes of this by-law 'area' shall not include the area of any underground cellar at the front or return front of the building when such cellar is entirely separated from the main building or is connected therewith only by an opening not exceeding 5 ft. in width."

Cubical extent is defined in Section 5, Pt. I, of the Act, and the definition is reproduced on p. 212.

[1] Introduced here from its numerical position.

This definition of cubical contents is not valid for any legal dealings under Part XIV of the Act (District surveyors and Superintending architect). See also Section 60 of the Act for cubical contents over 250,000 cub. ft.

By-law 147.[1] **Incombustible Coverings to Roofs, etc.**—" (1) The roof flat and gutter of every building and every turret, dormer, lantern light, skylight or other erection placed on the flat or roof thereof shall be covered externally with slates, tiles, metal or other incombustible materials. This requirement shall not apply to wooden cornices and barge-boards to dormers not exceeding 12 in. in depth and the door frames, windows and window frames of such turrets, dormers, lantern lights, skylights or other erections. Provided that:

" (i) on the covering required by the foregoing provisions of this by-law there may be superimposed a layer not exceeding $\frac{1}{8}$ in. in thickness of impervious material containing not more than 40 per cent. by weight of bitumen or other like combustible material;

" (ii) where a roof is not at an angle of more than twenty degrees with the horizontal:

" (*a*) the external covering may consist of rock asphalte containing not more than 17 per cent. by weight of bitumen;

" (*b*) in the case of a roof otherwise constructed entirely of incombustible material the external covering may consist of a layer or layers of compressed and impervious material having an aggregate thickness of not more than $\frac{3}{8}$ in. bedded down solidly on to the roof by a viscous material;

" (*c*) flashings and vertical sides to gutters may be constructed of the materials described in (*a*) and (*b*) above;

" (*d*) the external covering may consist of a layer not less than $\frac{1}{2}$ in. and not more than 1 in. in thickness of bitumen macadam composed of not less than 93 per cent. of stone chippings or other inert and incombustible material and not more than 7 per cent. of bitumen, provided that where necessary a layer or layers of compressed and impervious material be interposed between such covering and the roof and that where such compressed and impervious material is not incombustible it shall not exceed three layers bedded down solidly on to the roof by a viscous material, and the total thickness of such layers and bedding material shall not exceed $\frac{3}{8}$ in.

" (*e*) The requirement in paragraphs (*b*) and (*d*) of this proviso that material shall be bedded down solidly on to the roof by a viscous material need not be complied with in the case of a roof which is not at an angle of more than 1° with the horizontal.

" (2) The aggregate area measured in the plane of the roof of all turrets, dormers, lantern lights, skylights and any other erections or windows placed in or on or projecting from a roof or flat shall not exceed one-half of the aggregate area measured in the same plane of the roof or flat in or on which they are so placed or from which they so project.

" Subject to the provisions of Section 98 of the principal Act, the area of the glass in the case of a roof constructed entirely of incombustible material shall not be restricted if the whole of the glazing is of wired glass in hard metal frames."

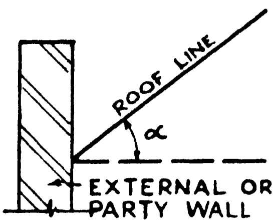
Section 98 of the Act relates to the roofs over shops which project beyond the main fronts of any buildings.

[1] Introduced here from its numerical position.

(3) The roof of a building other than of the warehouse class must not rise from a party wall or from an external wall at a steeper angle than 75° measured from the horizontal.

In the case of a warehouse building this angle must not be greater than 47° unless the roof, constructed entirely of incombustible materials, rises from an external wall bounding a street or open space.

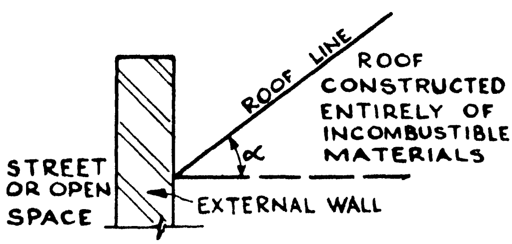
Thus in Fig. 220 :



For any building other than a warehouse :

α must not be greater than 75°.

FIG. 220



For a warehouse building :

α must not be greater than 47° except for the conditions given in Fig. 221, when α must not be greater than 75°.

FIG. 221

The foregoing requirements do not apply to towers, turrets or spires.

By-law 150.[1] **Fire-resisting Floors, Walls and Supports in Framed or Partially Framed Buildings.**—" All floors and staircases (together with their enclosing walls) in buildings wherein the loads and stresses are transmitted through each storey to the foundations :

(i) wholly by a skeleton framework of structural steel ; or
(ii) partly by a skeleton framework of structural steel and partly by a party wall or party walls ; or
(iii) wholly by a skeleton framework of reinforced concrete ; or
(iv) partly by a skeleton framework of reinforced concrete and partly by a party wall or party walls,

shall be constructed throughout of fire-resisting materials carried upon supports of fire-resisting materials. Provided that in the case of self-supporting flights of stairs constructed of reinforced concrete or steel, no additional support shall be required if such support is not necessary for the purpose of stability."

The first Schedule of the Act, p. 227, defines the materials which may be considered to be " fire-resisting."

By-law 151.[1] **Structural Metals not to Carry Electrical Currents.**—" Structural metal shall not be used for conducting electrical currents. Provided that such metal may be used as part of a sufficient and properly earthed apparatus for protection of the building against damage by lighting."

By-law 137. Party Floors, Arches, Partitions, etc.—Every arch or floor which is a party structure or which is over a public way or passage-way leading to premises

[1] Introduced here from its numerical position.

in different occupation must be constructed of brick, stone or other incombustible materials and have a thickness of solid material everywhere of not less than 5 in.

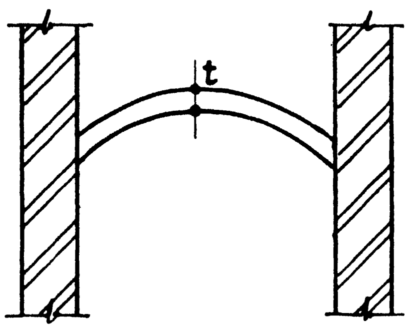


FIG. 222

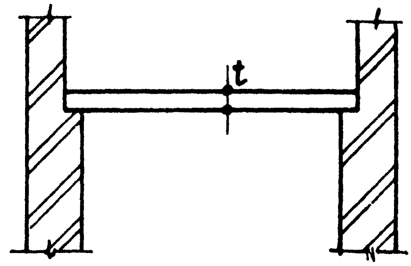


FIG. 223

t must not be less than 5 in.

When a partition in a building separates two portions which are in different occupation it must be constructed of incombustible material everywhere of not less than 3 in. if it is not higher than 10 ft. nor less than $4\frac{1}{2}$ in. if it is higher than 10 ft. In addition to the required thickness of solid materials, the partition must be rendered on each side. The stability of such a partition must be to the satisfaction of the district surveyor.

By-law 138.—See p. 168.

By-law 139. Party Floors, Arches, etc.—" (1) Every arch or other construction under any passage leading to premises in other occupation shall be formed of brick or stone or other incombustible materials not less than 5 in. in thickness in the aggregate and shall be of adequate stability to the satisfaction of the district surveyor to support such load as may be likely to be imposed on it.

" (2) Every arch or other construction under any public way or intended public way (other than a passage leading to premises in other occupation) shall be constructed in a manner similar to that prescribed in paragraph (1) of this by-law. Provided, however, the foregoing requirements of this paragraph shall not prohibit the formation of pavement lights in any such construction."

By-law 140.—See p. 168.

By-law 141.—See p. 176.

By-law 142.—See p. 176.

By-law 143.—See p. 180.

By-law 144. Separation of Buildings or Parts of Same Building.—" (1) *External or Party Wall required between Buildings.*—Every building shall be separated either by an external wall or by a party wall from any adjoining building. See Figs. 224 and 225, p. 172.

" (2) *Trade and Domestic Parts of Same Building.*—In every building exceeding 10 squares in area used in part for purposes of trade or manufacture and in part as a dwelling-house, the part used for purposes of trade or manufacture shall be separated from the part used as a dwelling-house by walls and floors constructed of fire-resisting materials and all passages, staircases and other means of approach to the part used as a dwelling-house shall be constructed throughout of fire-resisting materials. Provided that there may be constructed in the walls of such staircases and passages or in any wall which is subject to the provisions of this paragraph such doorways as are necessary for communicating between the different parts of the building if such doorways be fitted with fire-resisting doors in frames constructed of fire-resisting materials.

"For the purposes of this by-law 'area' shall not include the area of any underground cellar at the front or return front of the building when such cellar is entirely separated from the main building or is connected therewith only by an opening not exceeding 5 ft. in width."

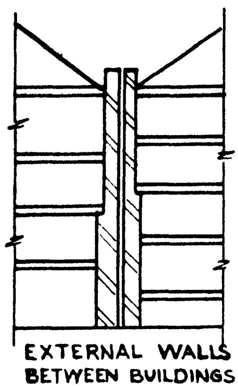


FIG. 224

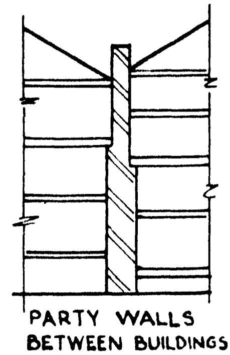
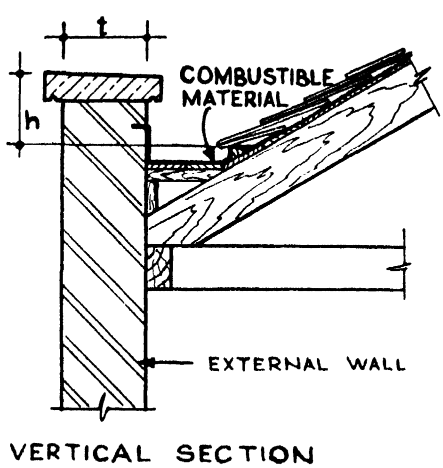


FIG. 225

See Clause (1), By-law 144, p. 171.

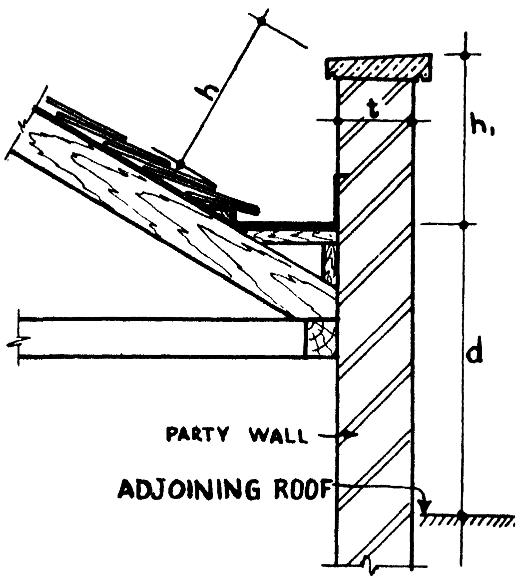
By-law 145. Parapets for External Walls.—"No gutter any part of which is formed of combustible materials shall adjoin an external wall unless such wall is carried up so as to form a parapet 1 ft. at the least above the highest part of such gutter and the thickness of such parapet shall be at least $8\frac{1}{2}$ in. throughout."



t must not be less than $8\frac{1}{2}$ in.
h must not be less than 12 in.

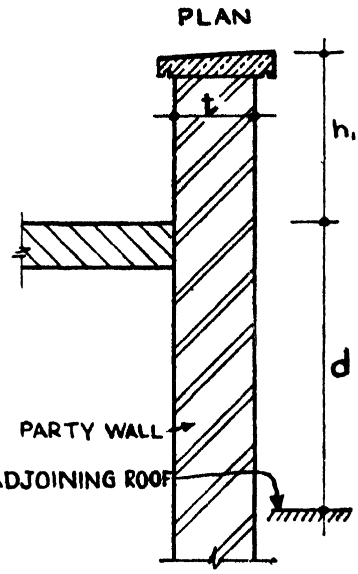
FIG. 226

By-law 146. Heights of Party Walls above Roofs.—(1) *Warehouse Buildings.*—Every party wall which encloses a warehouse building must, unless every roof abutting on the wall consists of at least 5 in. of solid incombustible material or, unless the difference in level between abutting roofs is greater than 3 ft., project above the level of the roof of the higher building, and the thickness of the projecting parapet must not be less than that required for such warehouse party wall by any by-law (see Part IV). The heights of the parapets are given in Figs. 227 and 228.



VERTICAL SECTION

FIG. 227



VERTICAL SECTION

FIG. 228

In Figs. 227 and 228 :

if d is not greater than 3 ft. or if any abutting roof does not consist of at least 5 in. of solid incombustible material, then, in warehouse buildings higher than 30 ft. :

t must not be less than that required for a warehouse party wall.
h_1 must not be less than 3 ft.
h_1 must not be less than the height required under clause 3, p. 175. } whichever is the greater.
h must not be less than 3 ft.

In Figs. 227 and 228 :

if d is not greater than 3 ft. or if any abutting roof does not consist of at least 5 in. of solid incombustible material, then, in warehouse buildings not higher than 30 ft. :

t must not be less than that required for a warehouse party wall.
h_1 must not be less than 15 in.
h_1 must not be less than the height required under clause 3, p. 175. } whichever is the greater.
h must not be less than 15 in.

Such parapets are not required if d is greater than 3 ft. or if all abutting roofs consist of at least 5 in. of solid incombustible material.

For further conditions, see Clause (3) p. 175.

(2) *Other Buildings.*—With the two exceptions, (*a*) and (*b*). given at the end of this paragraph, every party wall enclosing buildings other than of the warehouse class must, unless every roof abutting consists of at least 5 in. of solid incombustible material or unless the difference in level between abutting roofs is greater than 3 ft., project above the level of the higher roof, and the thickness of the projection must not be less than 8½ in. The heights of the parapets are given below. The symbols all refer to Figs. 227 and 228.

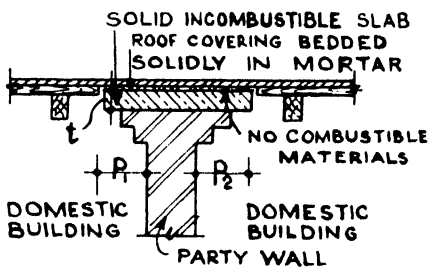
If d is not greater than 3 ft. or if any abutting roof does not consist of at least 5 in. of solid incombustible material, then, in all buildings other than of the warehouse class :

t must not be less than $8\frac{1}{2}$ in.
h_1 must not be less than 15 in.
h_1 must not be less than the height required under clause 3, p. 175. } whichever is the greater.
h must not be less than 15 in.

Such parapets are not required if d is greater than 3 ft. or if all abutting roofs consist of at least 5 in. of solid incombustible material.

(*a*) The above parapet is not required between two domestic buildings if an oversailing slab of incombustible material is provided at the top of the wall such that:

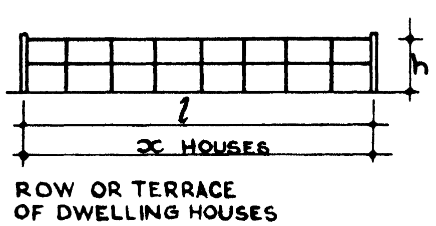
(i) it is not less than 4 in. in thickness; and
(ii) it continues for the full length of wall common to the two buildings; and
(iii) it is supported on brick corbels to the satisfaction of the district surveyor; and
(iv) it projects not less than 9 in. on each side of the party wall; and
(v) no combustible materials of the roof are carried over it; and
(vi) the roof covering is carried over it solidly bedded in mortar.



t must not be less than 4 in.
P_1 must not be less than 9 in.
P_2 must not be less than 9 in.

FIG. 229

(*b*) The above parapet is not required between any two dwelling houses of a row or terrace in which:



l is not greater than 150 ft.,
x is not less than 2,
x is not greater than 8, and
h is not greater than 2 storeys,

FIG. 230

and in which no combustible materials of the roof are carried across any party wall, the roof covering being carried across, solidly bedded in mortar for the full width and length of wall common to two houses, as shown in Fig. 231.

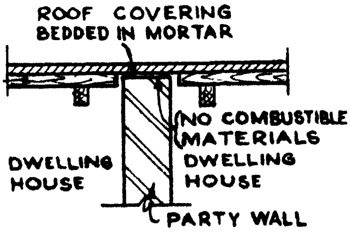
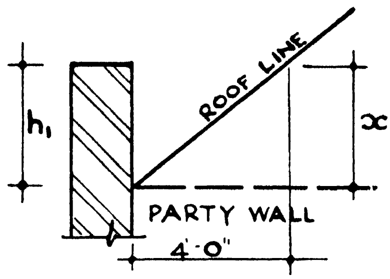


FIG. 231

(3) *Buildings of all Classes.*—(i) Every party wall parapet which is required by the preceding conditions in clauses 1 or 2 must not be less than the height determined in those clauses, nor, in the case of a parapet along a sloping roof which does not consist of 5 in. of incombustible materials, less than the height shown in Fig. 232.



h_1 must not be less than x nor less than the height required in Clauses 1 or 2.

FIG. 232

(ii) Where there is any turret, dormer, lantern light, skylight or erection constructed of combustible materials on a flat or sloping roof of a building, within 4 ft. of a party wall, then that party wall must be carried up to a level not less than 12 in. higher than the highest point of such turret, etc., and it must extend for a length 12 in. longer at each end than the length of the turret, etc. If the height required by this condition is less than that required under either clauses 1, 2 or 3 (i), then the latter must be taken as the determining height :

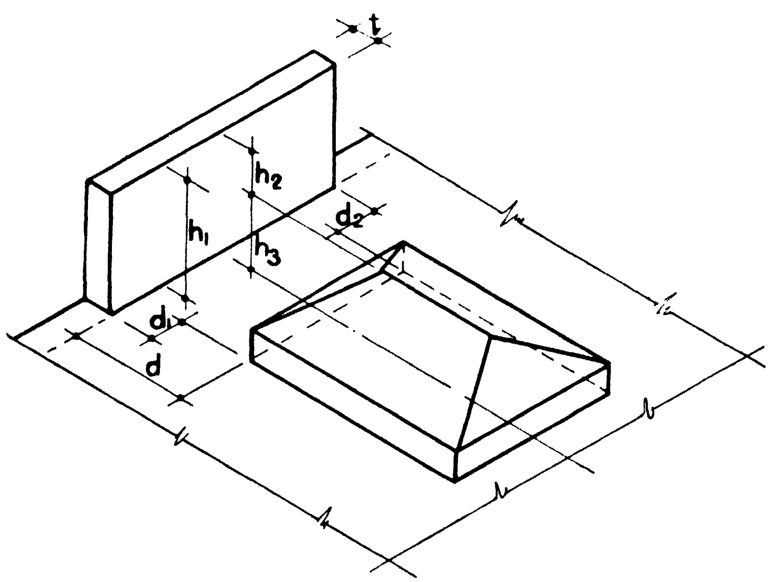


FIG. 233

If h_3 is the height from the roof to the highest point of a turret, dormer, lantern light, skylight or erection constructed of combustible materials, and if d is less than 4 ft. then :

h_2 must not be less than 12 in.
d_1 must not be less than 12 in.
d_2 must not be less than 12 in.
h_1 must not be less than $h_2 + h_3$ }
h_1 must not be less than the height required under clauses 1, 2 or 3 (i) } whichever is the greater.
t must not be less than the thickness specified in clauses 1 or 2 for the particular class of building.

. It is obvious that a strict observance of these requirements (Clause 3 (ii)) in every case would call for work which would give an unnecessarily high degree of protection in certain instances; that is to say, the protection would be very much higher than that obtaining in the rest of the structure. For instance, if such an erection on the roof were wide and shallow at the base, and if a portion of the base, nevertheless, came within 4 ft. of the party wall, it would still be necessary for the party wall to be carried up to a height 12 in. above the top of the erection for a length equal to the base plus 2 ft., in spite of the fact that the erection might both narrow to the top and slope away rapidly from the party wall. In such cases a waiver of the foregoing requirements should be sought.

It would appear that in such cases requirements should apply only to those lengths and heights of such turrets, etc., which lie within 4 ft. of the party wall, provided that the slope of the surface adjacent to the party wall is not greater than that given in By-law 147 (3).

If the slope is greater it seems reasonable to apply the requirements without modification.

By-law 142.[1] **Woodwork in External Walls.**—Except for any woodwork in windows of a shop on the ground storey of a building and except in the cases following, all woodwork in an external wall of a building must be set back at least 4 in. from the external face of the wall.

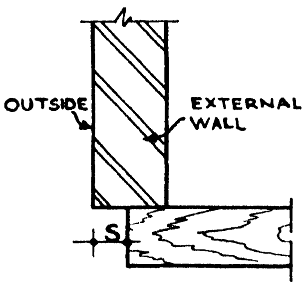


FIG. 234

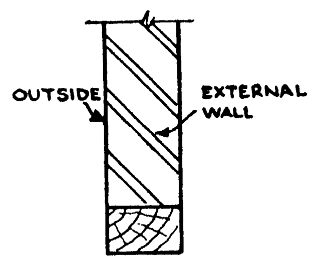


FIG. 235

S must not be less than 4 in.

However, timber storey posts, loophole frames, door or window frames or wooden beams carrying part of the external wall need not be set back as required above, but they must not project in front of the external face of the wall.

By-law 141.[1] **Requirements for Habitable Rooms.**[2]—(1) *Heights of Rooms.*—Every room which is to be used as an office or for habitation must have a clear height from the floor to the ceiling as specified in Figs. 236 and 237 and the following notes.

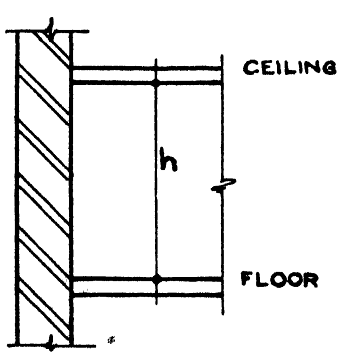


FIG. 236

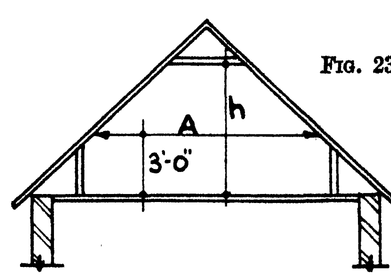


FIG. 237

[1] Introduced here from its numerical position.
[2] For the definition of habitable room, see "inhabited," Section 5 of the Act, p. 213.

For all storeys below the top storey :

h must not be less than 8 ft. 6 in.

For the top storey :

h must not be less than 8 ft.

For any room wholly or partly in the roof :

h must not be less than 8 ft. for an area at least equal to $\frac{A}{2}$, where A is the area of the room measured at a height of 3 ft. above the floor.

(2) *Timber Ground Floors.*—(*a*) " In any case where a room used for the purpose of an office, factory, workshop or workroom or for habitation forms the lowest storey of the portion of the building in which it is situate and has a floor constructed of wood other than solid wood bedded on concrete there shall be provided below the level of the underside of the floor-joists of the floor of such room a space at least 6 in. deep and such space shall be ventilated to the open air to the satisfaction of the district surveyor by means of air bricks or otherwise." The provision of concrete to cover the side is frequently waived (By-law 28).

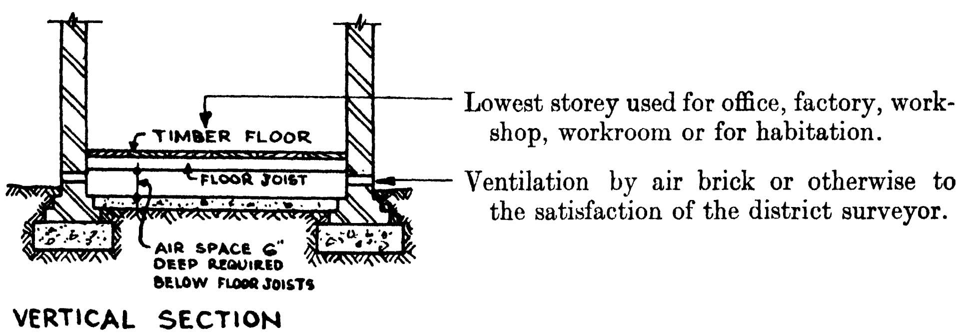


FIG. 238

In the preceding case (*a*) if there is an internal wall which is not a party wall nor a party partition but which extends below the timber floor then the portion below the floor must be perforated at intervals to the satisfaction of the district surveyor in order to afford proper cross ventilation for the full width of building in which the timber floor occurs.

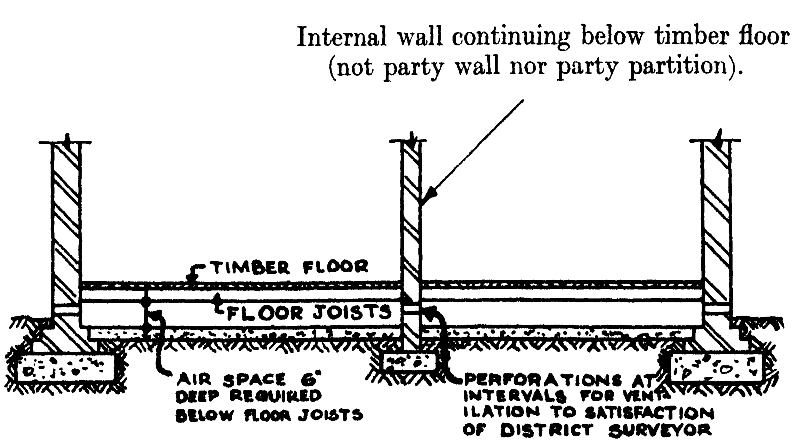


FIG. 239

(3) *Garages or Stables in Same Building.*—" In the case of a building in which there is a garage or stable and also a room used for the purpose of an office, factory, workshop or workroom, or for habitation, such garage or stable shall be separated from the remaining portion of the building as follows :

" (*a*) The vertical separation shall be by walls at least 8½ in. thick, constructed of solid incombustible material ; and

" (*b*) the horizontal separation (not being a party structure) shall be so constructed that every part thereof not occupied by a joist or beam has a layer at least 3 in. thick either of concrete pugging of good quality or of some other solid incombustible material properly supported and finished smooth as to its upper surface, and the underside of such separation shall be properly ceiled in the case of a stable with lath and plaster of good quality or other solid construction and in the case of a garage with asbestos cement or similar sheeting.

" Any openings in any such vertical separation shall be fitted with self-closing fire-resisting doors in frames constructed of fire-resisting materials."

(4) *Bathrooms.*—" Every room constructed or adapted to be used as a bathroom after the date on which this by-law comes into operation shall be ventilated to the satisfaction of the district surveyor."

The requirements of By-law 149 are not applicable to bathrooms. However, the district surveyor will generally be satisfied if Clauses 2 to 6 or Clause 7 of By-law 149 are observed. These remarks might reasonably be held to apply also to kitchens, corridors, and lavatories.

(5) *Only Rooms complying with Requirements to be Inhabited.*—" No person shall suffer to be inhabited any room constructed after the 31st day of December, 1894, unless such room is in conformity with the provisions of this by-law and By-law 149 or is by virtue of Section 233 of the principal Act deemed to be constructed in compliance with those provisions."

Section 233 of the Act permits buildings constructed before the commencement of the Act to be considered to be erected in accordance with the Act, provided first that the work had been constructed in accordance with any Act in force at the time of erection, and secondly that no provisions in the Act relating to the alteration of buildings would prevent such an interpretation.

For ventilation of staircases, see Section 74 of the Act, p. 215.

By-law 149.[1] **Windows and Ventilation in Habitable Rooms.**—There are seven clauses in the by-law relating to the natural lighting and to the ventilating of habitable rooms. The sixth of these has been brought forward to make it clear at the outset what are the requirements of some of the subsequent clauses.

Where the environs of a habitable room make it undesirable or impossible to ventilate that room in the ways set out in Clauses 1 to 6 (but only on that condition) then the requirements of Clause 7, instead of those of Clauses 1 to 6, must be observed.

" (1) Every room used for the purposes of an office or for habitation shall be provided with one or more windows directly opening into the open air, and such window or windows shall conform to clause (2)

" (6)[1] In this by-law the expression ' superficial area ' in relation to any window means the total clear superficial area of the glass, exclusive of any frames, sash-bars or other obstruction to the light.

" (2) The windows of every such room shall be so constructed that :

" (i) the superficial area of the windows is at least equal in the aggregate to one-tenth of the area of the floor of the room ; and

[1] Introduced here from its numerical position.

" (ii) the windows can, to an extent at least equal in the aggregate to one-twentieth of the area of the said floor, be opened in such manner that the top of the opening in the case of any window in the topmost storey is at least 6 ft. 3 in. above the level of the floor and in every other storey is at least 7 ft. above the level of the floor ; "

That is to say half the total required superficial area of window must be capable of being opened in the manner specified.

" (3) Any such room having no external wall or being wholly or partly in the roof of a building shall be deemed to conform to paragraph (2) of this by-law if it is lighted through the roof by means of :

" (*a*) a dormer window whereof :

"(i) the superficial area is at least equal to one-tenth of the area of the floor of the room ; and

" (ii) a part at least equal in area to one-twentieth of the area of the said floor can be opened in such manner that the top of the opening is at least 6 ft. 6 in. above the level of the floor."

That is to say, half the total required superficial area of window must be capable of being opened in the manner specified.

" or—

" (*b*) a lantern light whereof a part at least equal in area to one-twentieth of the area of the said floor can be opened."

It will be noticed that in this case the total required superficial area need not be any greater than that which must be capable of being opened.

" (4) A window which opens into a conservatory shall be deemed to open directly into the open air if such conservatory is provided with openings into the open air exclusive of doorways of an area equal at least to the combined area of all portions of windows which would but for this proviso be required by this by-law to open into the open air.

" (5) Every room which is to be used for any of the purposes mentioned in paragraph (1) of this by-law shall, unless :

" (i) ventilated by means of a ventilation system installed in the building of which such room forms part ; or

" (ii) provided with a fireplace, with a flue at least seven and a half inches across in every direction properly connected with such fireplace ;

be provided with additional ventilation by means of an aperture or air shaft communicating either direct to the open air or to a lobby or corridor ventilated direct to the open air.

" Every such aperture or air shaft shall have an unobstructed sectional area as follows :

" (*a*) if communicating direct to the open air at least 50 sq. in. ; or

" (*b*) if communicating with a lobby or corridor at least 100 sq. in.

" Provided that if such room has an ordinary open domestic gas fire fitted therein and such gas fire is properly connected to a flue not exceeding 30 ft. in height the sectional area of such flue may be taken as forming part of the required ventilating area.

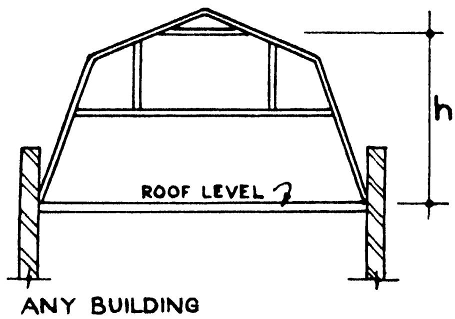
" (7) In any case where owing to the position level or unsuitable surroundings of any room used for any of the purposes mentioned in paragraph (1) of this by-law natural ventilation cannot in the opinion of the district surveyor be provided, other additional means of ventilation shall be provided to the satisfaction of the district surveyor and such ventilation shall be capable of supplying fresh air to all parts of such room at the rate of not less per hour than

750 cu. ft. of air per occupant or per 50 sq. ft. of floor area whichever requires the greater ventilation."

For a room of 8 ft. 6 in. in height this is equivalent approximately to a complete change of air every 35 minutes, or to $1\frac{3}{4}$ changes per hour.

By-law 147.—See p. 169.

By-law 148. Storeys in Roofs.—(1) " Not more than two storeys shall be constructed in the roof of any building."



h must not contain more than two storeys.

FIG. 240

(2) " Any storey constructed in the roof of any domestic building the upper surface of the floor of which storey is at a greater height than 60 ft. above the street level shall be constructed of fire-resisting materials throughout."

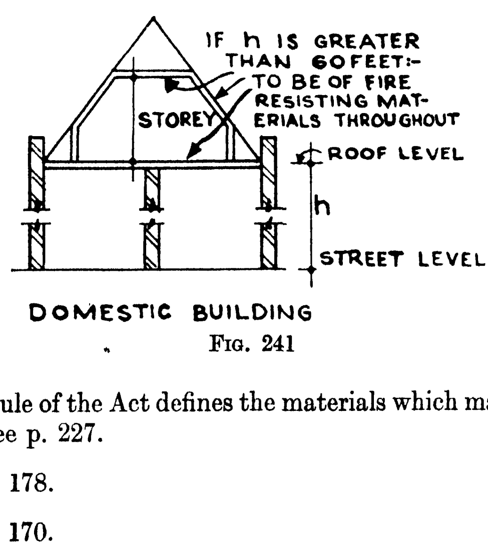


FIG. 241

The First Schedule of the Act defines the materials which may be considered to be " fire-resisting." See p. 227.

By-law 149.—See p. 178.

By-law 150.—See p. 170.

By-law 151.—See p. 170.

By-law 143.[1] **Projections from the Faces of Buildings.**[2]—" The following provisions shall apply to projections from buildings :

" (1) Every coping, cornice, string-course, fascia, window-dressing, portico, porch, balcony, verandah, balustrade, outside landing, outside stairs and outside steps and architectural projection or decoration whatsoever, and also the eaves, barge-boards and cornices to any overhanging roof

[1] Introduced here from its numerical position.

[2] See also Section 79 of the Act, p. 215.

(except the cornices and dressings to the window fronts of shops and except the eaves, barge-boards and cornices to detached and semi-detached dwelling-houses and to other dwelling-houses in which the party walls are corbelled out so as to project 4 in. beyond such eaves, barge-boards or cornices) shall be of brick, tile, stone, artificial stone, slate, iron, concrete, cement or other incombustible material. For the purposes of this paragraph a pair of semi-detached houses shall be deemed to be one building.

" Provided that verandahs, balustrades, outside landings, the treads, strings and risers of outside stairs, outside steps, porticoes and porches may be constructed in accordance with Paragraph II (5) of the 1st Schedule to the principal Act.

" (2) Every balcony, cornice or other projection shall be tailed into the wall of the building and weighted or tied down to the satisfaction of the district surveyor.

" (3) No part of any shop front shall be fixed higher than 25 ft. above the level of the ground immediately in front of the shop."

See Figs. 242 and 243.

(4) (i) Except in accordance with the conditions of paragraph (4) (ii), no part of any shop front, including the cornice or fascia, etc., must be nearer to the centre of a party wall than the amount of the projection of that part from the face of the building to which it belongs nor in any case nearer than 4 in. Thus in Figs. 242 and 243 :

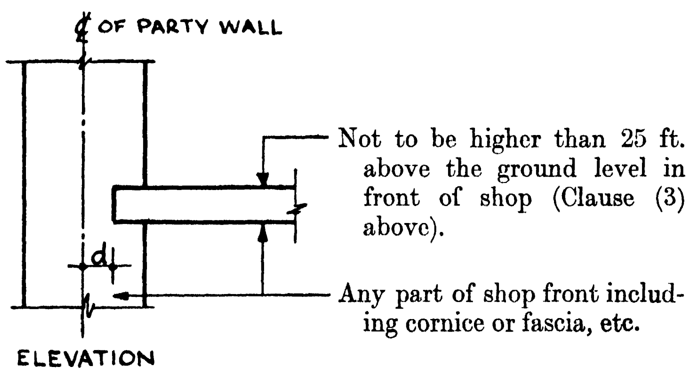


FIG. 242

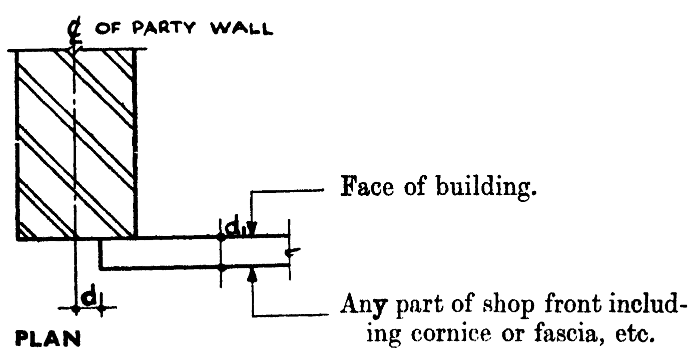


FIG. 243

d must not be less than 4 in. }
d must not be less than d_1 } whichever is the greater.

(ii) If, however, a pier and corbel of brick, stone or other incombustible material is built between the centre line of the party wall and the shop front concerned in such a way that they project not less than 1 in. in front of any part of the shop front, including the cornice, etc., then any such part may be nearer to the centre of the party wall, than the amount of its projection from the building to which it belongs, but not nearer than 4 in.

Thus in Fig. 244.

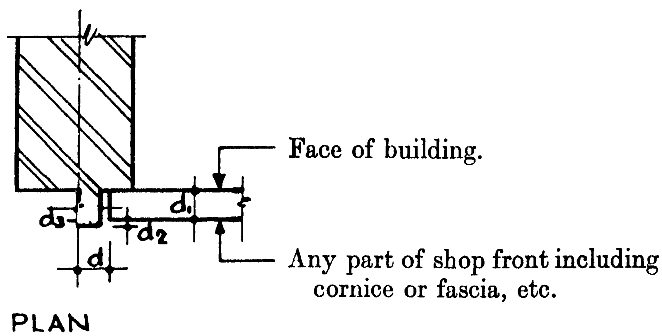


Fig. 244

Where d_1 is greater than 4 in. and if d_3 is not less than 4 in. and if d_2 is not less than 1 in., then d may be less than d_1

but d must not be less than 4 in.

A pier and corbel may be used in the preceding manner only if they do not project over any public way or over land to be given up to the public way.

(iii) Except in accordance with the conditions of paragraph (4) (iv), no part of any shop front, including the cornice or fascia, etc., must be nearer to the adjacent face of an external wall of an adjoining building, than the amount of the projection of that part from the face of the building to which it belongs, nor in any case nearer than 4 in. Thus in Figs. 245 and 246 :

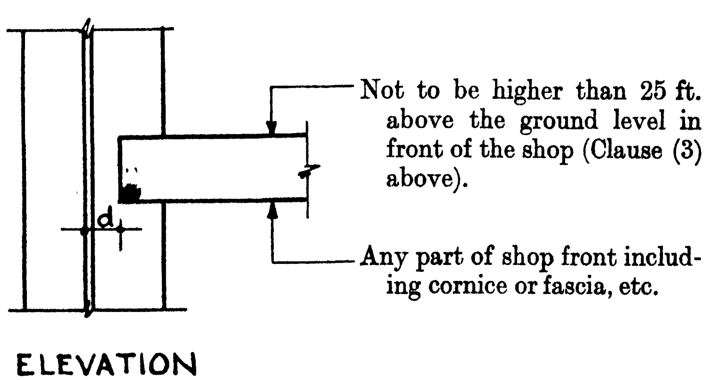


Fig. 245

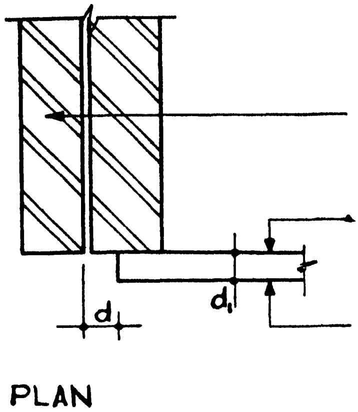


FIG. 246

d must not be less than 4 in
d must not be less than d_1 } whichever is the greater.

(iv) If, however, a pier and corbel of brick, stone or other incombustible material is built between the adjacent face of the external wall of the adjoining building and the shop front concerned in such a way that they project not less than 1 in. in front of any part of the shop front, including the cornice, etc., then any such part may be nearer to the adjacent external wall than the amount of its projection from the building to which it belongs, but not nearer than 4 in.

Thus in Fig. 247 :

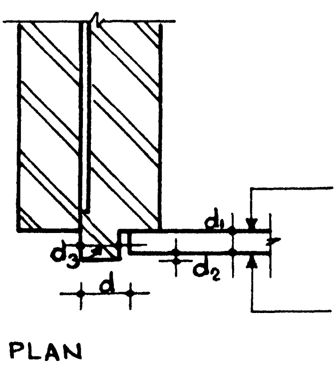


FIG. 247

When d_1 is greater than 4 in. and if d_3 is not less than 4 in. and if d_2 is not less than 1 in. then d may be less than d_1

but d must not be less than 4 in.

A pier and corbel may be used in the preceding manner only if they do not project over any public way or over land to be given up to the public way.

CHAPTER 12

PART IX. GENERAL. BY-LAWS 152 TO 159

By-law 152. Conversions.—" It shall be not lawful to convert a building or any part thereof in such manner that such building or part of a building when so converted will not be in conformity with these by-laws.

" Before any building or part of a building is converted notice in writing of such proposed conversion shall be given to the district surveyor by the owner or occupier of the building or part of the building in question."

By-law 153. Duties of the District Surveyor.—" For the purpose of securing the due observance of the provisions of these by-laws it shall be the duty of the district surveyor to survey any building structure or work affected thereby and he may require the builder to supply him with all such samples of materials as he may reasonably require."

By-law 154. Building Notices to the District Surveyor.—" The building notice required to be served on the district surveyor under Section 161 of the principal Act shall, unless the district surveyor otherwise agrees, be accompanied :

" (*a*) in the case of the erection of a building or part thereof other than a wall constructed in accordance with Section 2 of Part IV of these by-laws or of a chimney-shaft by plans and sections of sufficient detail to show the construction thereof, together with a copy of the calculations of the loads and stresses to be provided for and particulars of the materials to be used, and should such plans, sections, calculations or particulars be, in the opinion of the district surveyor, not in sufficient detail, the person depositing the same shall furnish the district surveyor with such further plans, sections, calculations or particulars as he may reasonably require ; and

" (*b*) in the case of an alteration or addition or other work (other than a wall constructed in accordance with Section 2 of Part IV of these by-laws) by such plans, sections, calculations or particulars as the district surveyor may reasonably require."

By-law 155. Evidence of the Quality of Structural Metal.—" The district surveyor may by notice require reasonable evidence of the quality of the metal used or to be used in the construction or conversion of a building or chimney-shaft or any part thereof, and may require the builder or other person causing or directing the work to be executed to make such tests of such metal and to drill such columns as the district surveyor may deem necessary.

" Any person dissatisfied with any requirement of the district surveyor under this by-law may appeal to a Court of Summary Jurisdiction."

By-law 156. All Work to the Satisfaction of the District Surveyor.—" All work affected by any of the provisions of these by-laws shall be carried out to the satisfaction of the district surveyor in a proper and workmanlike manner."

By-law 157. Penalties for not Complying with the By-laws.—" Every person who contravenes or fails to comply with any of the provisions of these by-laws or of any requirement made thereunder or in pursuance thereof shall be deemed to have committed an offence against these by-laws and shall be liable on summary conviction to a penalty not exceeding the sum of fifty pounds and a daily penalty not exceeding ten pounds."

By-law 158. Replacements of Enactments.—The enactments, by-laws and regulations set out in column 1 below are replaced by the by-laws set opposite thereto in column 2 of this by-law :

By-laws Nos. 1 to 7 made by the Council under Section 16 of the Metropolis Management and Building Acts Amendment Act, 1878, 13 October, 1891, and confirmed on 19 October, 1891.	Parts 1 to VII and By-laws 150 and 157.
By-laws Nos. 1 to 5 made by the Council under Section 31 of the London Council (General Powers) Act, 1890, on 13 October, 1891, and confirmed on 19 October, 1891.	,, ,,
Section 57, sub-section (1) and the Second Schedule to the London Building Act, 1930, Section 58 and the Third Schedule to the London Building Act, 1930, Section 59 and the regulations relating to reinforced concrete construction having effect as if made in pursuance of the London Building Act, 1930, and allowed on 8 September, 1915, and 30 July, 1927, Section 62 and Section 70.	,, ,,
Section 60, sub-section (1), (2), (3) and (5)	,, ,,
Section 222, paragraphs (6) and (7)	,, ,,
Section 61	By-law 142
Section 63	By-law 145
Section 65	By-law 146
Section 66	Part IV
Section 67	By-law 147
Section 68	By-law 148
Section 69, sub-sections (1) to (18) and sub-sections (20) to (22).	By-law 132
Section 71	By-law 136
Section 72	By-law 138
Section 73, the words from " and " where that word first occurs to the word " families."	By-law 140
Section 75	By-law 141
Section 76, sub-sections (1), (2) and (3)	By-law 149
Section 76, sub-section (4)	By-law 141
Section 77	By-law 137
Section 78	By-law 139
Section 79, paragraph (1), in paragraph (2) from the beginning of the paragraph to " and " where that word secondly occurs and paragraph (4).	By-law 143
Section 80, sub-sections (1) and (2)	By-law 144
Section 80 in sub-section (3) from the beginning of the section to " materials."	By-law 140

By-law 159. Appeals.—" In so far as these by-laws control the use of metal skeleton or reinforced concrete construction they relate to the stability of a building or part of a building.

" In pursuance of the provisions of sub-section (5) of Section 4 of the London Building Act (Amendment) Act, 1935, any person dissatisfied with the refusal of the Council to modify or waive the requirements hereinafter specified (in so far as they relate to the use of metal skeleton or reinforced concrete construction) of these By-laws, or dissatisfied with any term or condition attached by the Council to any such modification or waiver, may appeal to the Tribunal of Appeal:

"By-law 3 so far as regards the average weight of reinforced concrete together with plastering, tiles, mosaic, granite or other similar finishing material for such concrete.

"By-laws 4 and 6 so far as regards the pressure of wind and the extent of surface subjected thereto.

"By-law 14 so far as regards the relation between the weight and volume of Portland cement complying with the British Standard Specification No. 12—1931.

"By-law 30 so far as regards the permissible pressure of foundations on the natural ground.

"By-law 39, Conditions (2) and (3) to the proviso.

"By-law 43 so far as regards:

"(a) the thickness of portions of external walls between columns and beams; and

"(b) the thickness of party walls.

"By-laws 59 and 60 so far as regards brickwork which supports steel or reinforced concrete construction.

"By-laws 66, 68 and 97.

"By-law 104, so far as regards the relation between the overlap and the diameter of reinforcement.

"By-law 151 so far as regards reinforcement.

"By-law 154."

CHAPTER 13

TIMBER BY-LAWS

Timber By-law 1. Definitions.—For the purpose of these timber By-laws the following meanings are to be taken for the various terms used unless the context indicates or expressly states alternative meanings.

"Dead loading: means the weight of all walls, floors, roofs, partitions and other like permanent construction.

"Superimposed loading: means all loading other than dead loading." For the loads actually to be taken see Timber By-law 7.

"Joist: means a bridging or common joist in a floor or a ceiling." That is to say, a joist, in a floor or roof, supported on walls, timber binders, steel joists or similar constructions. See Fig. 248.

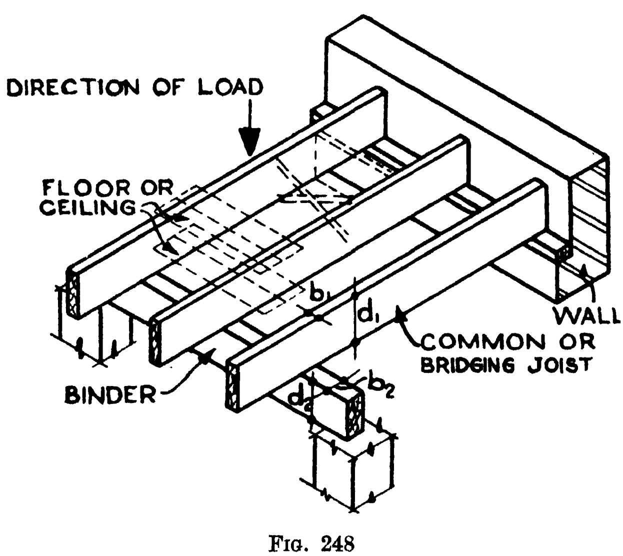


Fig. 248

"Binder: means a beam supporting joists." See Fig. 248.

"Rafter: means a common rafter in a pitched roof." Rafters run in the direction between the ridge and the eaves of the pitched roof and they carry the external roof covering on boards or battens. They are generally supported on a ridge piece and on a wall plate at the ends and intermediately if necessary on purlins. See Fig. 249. Purlins afford intermediate support to rafters in pitched roofs or if a sheet roofing, capable of spanning a reasonable distance by itself is used, purlins may carry such roofing directly, that is to say, without any rafters, boards or battens. Purlins running parallel to the ridge are usually supported on walls, roof trusses, hip or valley pieces. See Fig. 249.

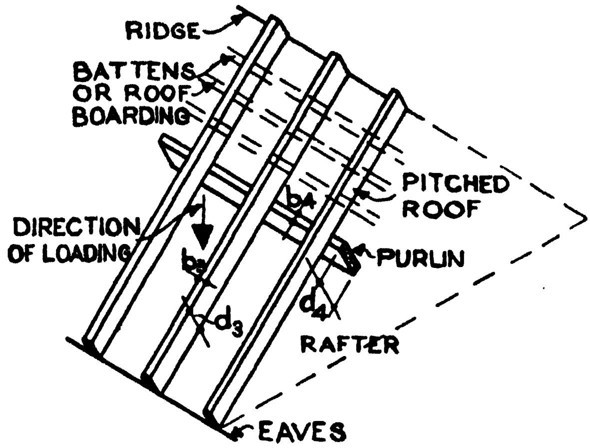


FIG. 249

" Breadth : in relation to a binder, joist, purlin or rafter means the transverse dimension which is at right angles to the direction of the loading." Thus in Figs. 248 and 249 :

b_1 is the breadth of the joist,
b_2 is the breadth of the binder,
b_3 is the breadth of the rafter, and
b_4 is the breadth of the purlin.

" Depth : in relation to a binder, joist, purlin or rafter means the transverse dimension which is at right angles to the breadth." Thus in Figs. 248 and 249 :

d_1 is the depth of the joist,
d_2 is the depth of the binder,
d_3 is the depth of the rafter, and
d_4 is the depth of the purlin.

" Length, in relation to a piece of timber : means the clear dimension between its supports," i.e. l in Fig. 250 below.

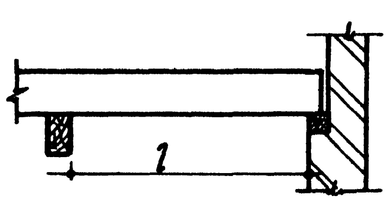


FIG. 250

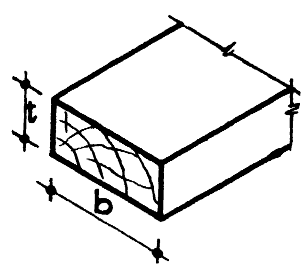


FIG. 251

If t is less than b, then t is to be taken as the thickness.

" Thickness, as applied to a piece of squared timber or a board : means the lesser transverse dimension." See Fig. 251 above.

" Slenderness ratio, in relation to a post or a strut : means the ratio of length to thickness.

" Flat roof : means a roof not having an inclination of more than 20° with the horizontal." See Fig. 252.

" Pitched roof : means a roof having an inclination of more than 20° with the horizontal."

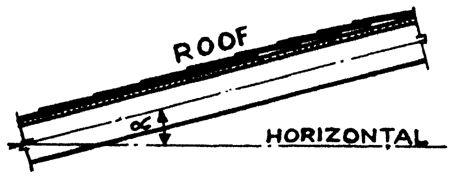


FIG. 252

If α is not greater than 20°, roof is " flat."
If α is greater than 20°, roof is " pitched."

SECTION 1

The requirements of the five Timber By-laws of this section apply to all structural timber coming within the scope of the Timber By-laws generally, irrespective of the Section in which the detail requirements for any particular timber member may lie.

Timber By-law 2. Scope.—" These by-laws shall apply only to timbers as cut and fixed in a building and bearing a load or loads other than those due to their own weight and to wind pressure on their surfaces, but shall not apply to timbers in any building which is exempt from the operation of Parts VI and VII of the London Building Act, 1930."

For a list of the buildings to which these Timber By-laws do not apply, see Section 223 of the Act, p. 224.

Timber By-law 3. Quality of Timber.—" Timber shall be well cut and free from warp, wind or other deformation and from signs of rot, worm and beetle, and shall not contain large, loose or dead knots, checks, splits or other defects to such an extent or so situated in the piece as to render it insufficient in strength or stiffness for its functions in the work, and no timber shall be used which, in the opinion of the district surveyor, is so inferior in quality or conditions as not to be suitable for its purpose. Timber, when used, shall be well seasoned and shall be deemed not to be so if, when tested in the manner described in the Appendix to the British Standard Specification numbered 585—1934, it shows a moisture content exceeding 22 per cent."

Some notes on the relative importance of the different defects named above and on their characters are given in Chapter 35, Volume III.

There are two grades of timber to which the Timber By-laws apply. They are known as " non-graded " and " grade 1,200 lb. *f* " timbers.

To assist in the selection and approval of non-graded timber, the Superintending Architect to the Council has published a short Memorandum prepared by the Department of Scientific and Industrial Research setting out the limits of defects which could be present in non-graded timber. If greater defects occur, the timber should be rejected. The limits are :

" (i) Diagonal grain or spiral grain having a slope not greater than one in eight.

" (ii) End splits having a length or, if there be more than one, having a total length not exceeding the width of the face.

" (iii) Seasoning checks having a depth not exceeding one-quarter the thickness of the piece.

" (iv) Sound knots, when appearing wholly on the face or side of the piece, of a size not greater than one-quarter the width of the face.

" (v) Sound knots on the edge or narrow surface of the piece including round knots, oval knots, corner knots, and splay knots, of a size not greater than five-eighths the width of the edge or one-quarter the width of the face, whichever is the less.

" (vi) Loose knots or knot holes not exceeding in size one-half the size mentioned in (iv) or (v) above respectively.

"(vii) A knot cluster (with or without loose knot holes) where the composite size is not greater than one-and-a-half times the greatest size mentioned in (iv) or (v) above respectively."

Higher stresses than the normal for non-graded timbers are allowed by these Timber By-laws for both Douglas fir and long-leafed pitch pine. These timbers are classed as "Grade 1,200 lb. f" and in addition to complying with the requirements for other timbers, they must each comply with the requirements of the Schedule, which, for convenience, has been brought forward to this position from the end of the Timber By-laws.

Schedule: "Every piece shall be sound and free from defects except as specified in this schedule and shall be of such grain as not to have less than four annual rings to the inch and shall be free from spiral or diagonal grain having an inclination to the direction of the length exceeding 1 in. in 10, except when such spiral or diagonal grain is so disposed as not to impair the strength of the piece.

"The number of annual rings per inch shall be ascertained in the following manner: The measurement shall be made at each end of the piece on a measuring line 3 in. long in the direction of the radius of the rings."

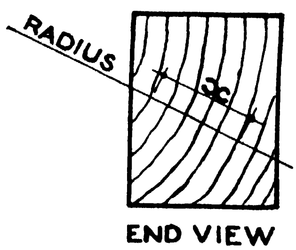


Fig. 253

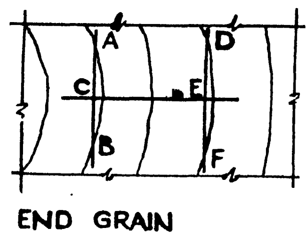


Fig. 254

The x on which the number of rings must be measured must not be less than 3 in.

The radial line on which to measure the number of rings can generally be determined sufficiently accurately by eye. If greater accuracy is required it can be found by the method shown in Fig. 254.

Any line whatever is drawn on the end of the timber to intersect any one ring at A and B. Another line DE is then drawn which must be parallel to the first line and which will intersect the same or any other one ring at points D and E. If the lengths AB and DE are bisected at C and F respectively, then the line CF is one of the radii of the rings.

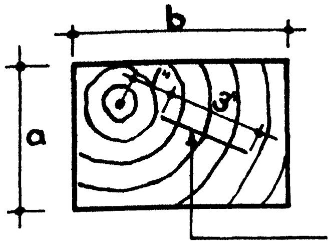
It is important that the number of rings is measured on a section of the timber at right angles to the longitudinal axis of the piece.

If the plane of the section cuts the timber obliquely, although it is still possible to measure the number of rings correctly, a false result may be given, particularly in cases in which the heart has been off centre in the tree from which the timber was cut.

"In the case of a boxed-heart piece, the measuring line shall extend over grain which is representative of a fair average of the section. When in such a piece the least dimension is 6 in. or less, the line shall begin at and extend from a point at a distance of 1 in. from the pith."

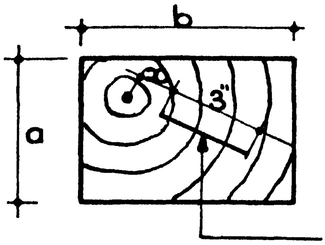
Boxed-heart pieces are pieces which have been cut in such a manner that they contain the heart or pith. See Fig. 255.

"Where in such a piece the least dimension exceeds 6 in., the measuring line shall begin at, and extend outwards from a point at a distance from the pith equal to one-quarter the least dimension of the piece."



In a boxed-heart piece if a is not greater than 6 in., or if b is not greater than 6 in.: measuring length is to be taken on representative grain.

FIG. 255

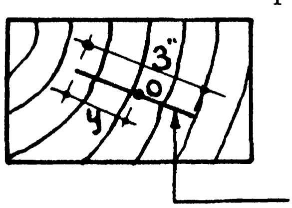


In a boxed-heart piece if a is less than b and if a is greater than 6 in.: measuring length is to be taken on representative grain such that $x=\frac{a}{4}$.

FIG. 256

It may not always be possible to fulfil the conditions for measuring the grain in the manner specified for these cases. For instance, in the last case, if b were 7 in. and a 6·5 in. and if the most representative portion of the grain lay across a line parallel to a and if the pith lay at the centre, then in order to get the full 3 in. for the measuring length x could not be more than $\frac{1}{4}$ in. whereas the requirements are that x should be $1\frac{5}{8}$ in. It is obvious that in such a case the manner of the measurements of the rings should be modified. The choice between a shorter measuring length or a less distance from the pith should then be left to the discretion of the user.

"In the case of a piece without pith, the centre of the measuring line shall be at the centre of the end of the piece."



In a piece without pith if O is the centre of the end: measuring length is to be taken on representative grain such that $y=1\frac{1}{2}$ in.

FIG. 257

"Knots shall be sound and free from rot."

"The width of a knot shall be ascertained in the following manner: The knot shall be measured on that face of the piece in which the area of the knot is greater." The width is to be taken as the average of the greatest and least diameters.

In Fig. 258:

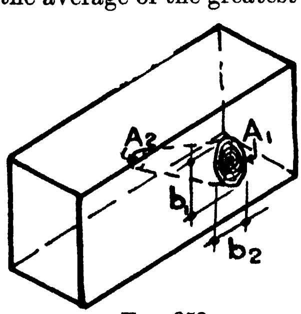


FIG. 258

if the area A_1 is greater than the area A_2 and if b_1 is the greatest diameter of area A_1 and if b_2 is the least diameter of area A_1 and if w is the width required for the purpose of this Schedule, then

$$w = \frac{b_1 + b_2}{2}.$$

When a knot intersects a corner of the timber so that it is exposed on two adjacent faces (or more if the corner is bevelled) the width of the knot is to be taken as the sum of the greatest distances between the corner and the part most remote from it.

In Fig. 259 :

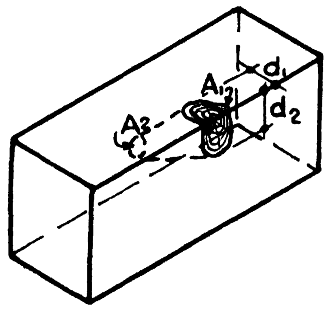


FIG. 259

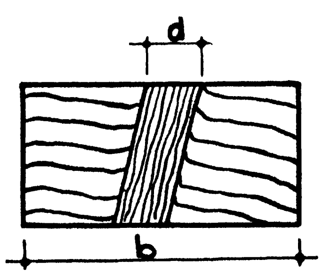
if the area A_1 is greater than the area A_2 and if d_1 is the distance from the corner along one face,
and if d_2 is the distance from the corner along an adjacent face, etc.,
and if w is the width required for the purpose of this Schedule, then

$$w = d_1 + d_2.$$

Comments on the foregoing methods of measuring knots are made in Chapter 35.

Unless the knot is so situated in the timber that the strength will not be reduced beyond a negligible amount, no tight knot is to have a greater diameter than one-fourth of the greater transverse dimension of the section. The wording of this requirement in the actual schedule is ambiguous in two ways. The first is that the diameter is not defined and the second is that it is not stated in which face or faces the knot occurs. It can be assumed, however, that " diameter " is intended to mean the greatest dimension of the knot projected on the plane of the cross-section of the member, and that the knot intersects the wider face or faces.

Thus in Fig. 260 :

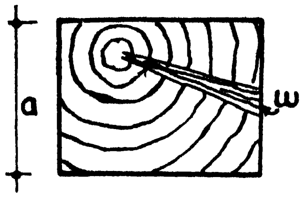


d must not be greater than $\frac{b}{4}$.

FIG. 260

Also if a tight knot is contained within the thickness of a piece of timber, its width as measured above must not be greater than one-third of that thickness.

" A loose knot or knot-hole shall not exceed in width one-half the greatest width permitted in the case of a tight knot, unless so situated as not to impair the strength of the piece.



If w = width of tight knot:

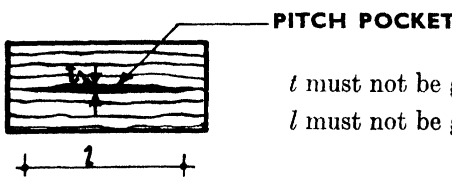
w must not be greater than $\frac{a}{3}$.

FIG. 261

" A knot cluster or a knot-hole cluster shall be measured as a single unit.

" Pitch pockets shall not exceed 8 in. in length nor shall they exceed one-eighth of an inch in width."

The length is to be measured parallel to the grain.



t must not be greater than $\frac{1}{8}$ in.

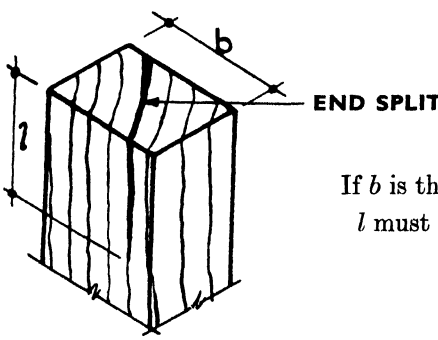
l must not be greater than 8 in.

FIG. 262

" Sapwood shall be not more than slightly discoloured.

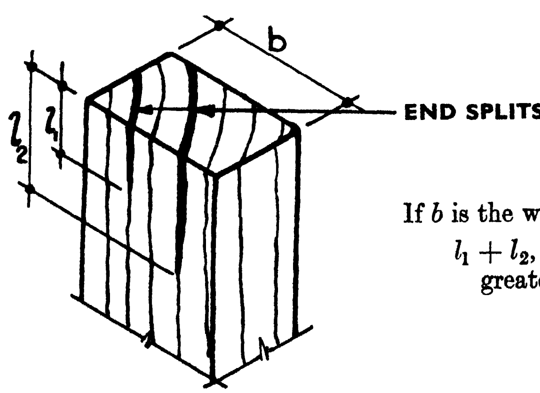
" The depth of torn grain shall not exceed $\frac{1}{16}$ in."

Torn grain is that on the surface which has been disturbed or ripped out in the cutting of the piece to the desired section.



If b is the width of the section:
l must not be greater than b.

FIG. 263



If b is the width of the section:
$l_1 + l_2$, etc., must not be greater than b.

FIG. 264

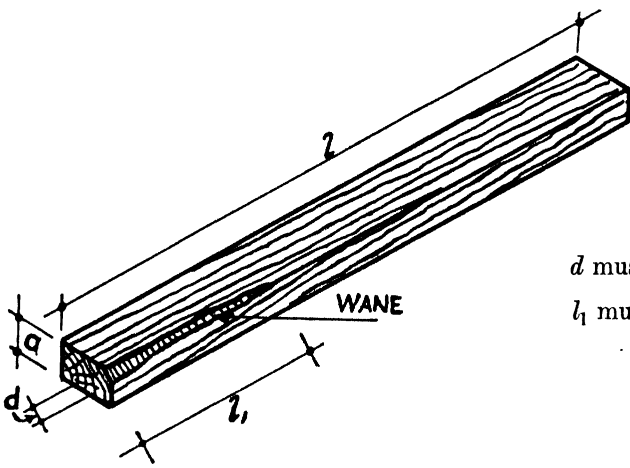
"The length of an end split shall not exceed the width of the piece. If there be more than one split in the same end, then the sum of their lengths shall not exceed the width of the piece." See Figs. 263 and 264, p. 193.

"Checks shall not be such as to impair materially the strength of the piece."

Checks and end splits are discussed in Chapter 35 in relation to the strength of timbers used for different structural purposes.

Wane on the corner of a piece of timber is the inclusion of bark along that corner or the absence of wood. It is measured along the sloping surface of the solid wood which remains.

The wane on one corner of a timber which is not greater than 4 in. in thickness must not be wider than ½ in. nor extend for a greater length than one-third of the total length of that timber.



When a is 4 in. or less and when wane is on one corner only :

d must not be greater than ½ in.

l_1 must not be greater than $\frac{l}{3}$.

FIG. 265

In the above case the area of wane "A" can be taken as $\frac{dl_1}{2}$, with a maximum value of $\frac{l}{12}$ sq. in.

When wane occurs on more than one corner of a timber which is not greater than 4 in. in thickness, the sum of the widths of the individual wanes must not be greater than ½ in. and the sum of the areas of individual wanes must not be greater than "A."

Thus, although in the case of wane on one corner only the length is restricted to one-third of the total length of the member, in this case there is no such restriction and it is therefore possible to use timber with wane for a much greater length and even, in the extreme cases, for the full length.

The wane on one corner of a timber which is greater than 4 in. in thickness must not be wider than one-eighth of the thickness nor extend for a greater length than one-third of the total length of that timber.

Thus in Fig. 265.

When a is greater than 4 in. and wane is on one corner only :

d must not be greater than $\frac{a}{8}$ in.

l_1 ,, ,, ,, ,, $\frac{l}{3}$

In this case (immediately above) the area of wane A_1 can be taken as $\frac{dl_1}{2}$, with a maximum value of $\frac{al}{48}$ sq. in.

When wane occurs on more than one corner of a timber which is greater than 4 in. in thickness the sum of the widths of individual wanes must not be greater than $\frac{a}{8}$ and the sum of the areas of the individual wanes must not be greater than A_1.

Thus, although the case of wane on one corner only the length is restricted to one-third of the total length of the member, in this case there is no such restriction and it is therefore possible to use timber with wane for a much greater length and even, in the extreme cases, for the full length.

Timber By-law 4. Minimum Thickness of Rafters, Purlins, Joists and Binders.—"Every rafter, purlin, joist and binder shall have a breadth in a single piece of not less than $1\frac{3}{4}$ in."

Thus in Figs. 248 and 249:

b_1 must not be less than $1\frac{3}{4}$ in.
b_2 ,, ,, ,, ,, $1\frac{3}{4}$ in.
b_3 ,, ,, ,, ,, $1\frac{3}{4}$ in.
b_4 ,, ,, ,, ,, $1\frac{3}{4}$ in.

The breadth is measured at right angles to both the direction of loading and the length of span. It should not be inferred from this requirement that the breadth of all such members must be provided by a single piece of timber. Two or more pieces may be placed side by side and bolted or otherwise secured to work together as a beam, etc.; but no individual piece used to form composite section may be less than $1\frac{3}{4}$ in. in breadth.

Timber By-law 5. Joints.—"All framing, tenoning, spiking or nailing, bolting, strapping and other jointing shall be so designed as safely to transmit the loads and stresses to which it will be subjected, and the execution of all such jointing shall be to the satisfaction of the district surveyor. Where bolts are used, adequate plates or washers shall be employed to preserve the grain of the timber from crushing."

The design of joints, etc., is discussed in Chapter 35.

Timber By-law 6. Timbers to comply with Section 2 or Section 3.—With the exception of those cases which are given in the next paragraph and which conform to the requirements of Section 3, all timber used structurally in a building, to which these Timber By-laws apply (see Timber By-law 2), must be capable safely of resisting and transmitting all superimposed and dead loads coming on to them, in such a way that the requirements for stresses and other conditions given in Section 2 are satisfied. Roof and floor-boards are covered by these Timber By-laws.

In Section 3 tables and empirical rules are given by which the sizes, etc., of rafters and purlins for pitched roofs, roof and floor-boards, joists and binders can be determined; but these rules may be used only when such timbers comply with the other requirements of that Section (Section 3). If they do not comply with these other conditions of Section 3, then such timbers must be calculated exactly and their sizes, etc., determined by the provisions of Section 2.

Section 2. Timber By-laws 7 to 11

The five Timber By-laws of this section give the limitations of stress and the other conditions which must be satisfied by all structural timber sections which are determined by exact calculations and not by the empirical rules, etc., given in Section 3.

Timber By-law 7. Loads.—The superimposed or live loads which must be taken for the design of structural timber members are given in Table 42, which follows. The dead weights of the various materials used in the building must be taken as those given in the British Standard Specification No. 648—1935. The weights to be taken for the " Grade 1,200 lb. *f* " timbers, Douglas fir and long-leafed pitch pine, are 30 and 41 lb., per cu. ft. respectively. Hardwoods, such as oak and teak, etc., are given at 45 lb. per cu. ft. and softwoods generally at 30 lb. per cu. ft.

Where the dead weight is not given in the British Standard Specification for any particular material which may be used, this weight is to be determined and it must be approved by the district surveyor.

TABLE 42

SUPERIMPOSED LOADINGS

Use of Floor or Roof The index figures refer to the notes which follow this table.		Superimposed Loading in lb. per sq. ft. of Floor or Flat Roof Area		
		Boards to Floors and Flat Roofs	Joists between Binders or Other Supports	Binders or Other Supports
1	Rooms used for residential purposes [2] . .	200	40	40
2	Offices (floors above entrance floor) [2] . .	200	80	50
3	Offices (entrance floor and floors below entrance floor) [2] retail shops,[2] and garages for private cars except those heavier than 2½ tons net.	200	90	80
4	Corridors, stairs and landings [2, 6] . . .	200	100	100
5	Workshops,[2] factories [2] and garages for all motor vehicles except private cars which are not heavier than 2½ tons net.	200	150 [1]	120 [1]
6	Warehouses,[2] book stores,[2] stationery stores [2] and the like.[2]	200	200 [1]	200 [1]
7	Any purpose not given in this table [2] . .	200	[5]	[5]
8	Flat roof (i.e. roof having an inclination with the horizontal of not more than 20°) [2, 3]	200	50	30
9	Ceiling joists	—	25 [4]	—

Note 1.—The exact loading must be determined and it must be approved by the district surveyor ; but in no case must it be less than the value given.

Note 2.—If a superimposed load which can roll or move on wheels is to be applied, the exact weight of that load must be determined and if the weight is then increased by an amount not less than 50 per cent., the result may be considered to a static load for which the member must be designed. The design should allow for such a load to be applied at points where it would produce the greatest bending and shear stresses, etc. Timber By-law 7 does not specifically require it, but it seems clear if the increased moving load at the worst positions on the member would produce effects less than those of the tabulated distributed static loads, that the latter must be taken as the governing design loads.

It should be noted that this requirement for an increase of at least 50 per cent. of the static weight of a moving superimposed load does not apply to loads specified for garages.

Note 3.—The superimposed loads on a sloping roof are to be taken in exactly the same manner as for the corresponding roofs of other buildings and a description of the requirements is given under Note 3 in By-law 4 of the general By-laws—see p. 24.

Note 4.—The calculated deflection of any ceiling joist under this load must not be greater than $\frac{1}{360}$ of its length. If the deflection would be greater than this amount, the span must be reduced, the size of the member increased or the distance between the joists must be reduced. The last is the least economical method of meeting the requirements and it may not even do so. For notes on deflection see Timber By-law 11.

Note 5.—The exact loading must be determined and it must be approved by the district surveyor.

Note 6.—For corridors, stairs and landings contained within the one flat or residence, By-law 4 calls for loads of only 50 lb. per sq. ft. for slabs and 40 lb. per sq. ft. for their supports. Timber By-law 7, however, calls for 100 lb. per sq. ft. for each. If, in certain cases, appreciable savings could be made by the adoption of the requirements of By-law 4, then a waiver of Timber By-law 7 should be sought.

Timber By-law 8. Permissible Stresses in Timber Members.—The stresses in timber must not exceed the values given for each type of loading in Table 43.

TABLE 43

TIMBER STRESSES

	Nature of Stress	Maximum stress, lb. per sq. in.	
		Non-graded Timber	"Grade 1,200 lb. *f*" Timber
1	Compression and tension parallel to the grain, due to bending.*	800	1,200
2	Shear (parallel to the grain)†	90	100
3	Compression perpendicular to the grain . . .	165	325
4	Compression parallel to the grain in posts and struts of slenderness ratios not greater than 10. (For "slenderness ratio" see definitions.)	800	1,000
5	Tension parallel to the grain . . . . . . (no tension should be taken perpendicular to the grain.)	800	1,200

* See Timber By-law 11, p. 202, and Table 46.
† See Chapter 35, Vol. III.

Modulus of Elasticity.—The moduli of elasticity of non-graded and of "Grade 1,200 lb. *f*" timbers are to be taken as 1,200,000 lb. per sq. in. and 1,600,000 lb. per sq. in. respectively.

Grain loaded obliquely.—It will be observed that no values have been specified for the permitted stresses when the grain is loaded obliquely. Values have, however, been given for end and side stresses, referred to as "parallel" and "perpendicular" respectively in Table 43, and from these the stress on a plane making an angle α with the longitudinal direction of the grain can be derived from the formula :

$$s = s_2 + (s_1 - s_2) \sin^2 \alpha \quad . \quad . \quad . \quad . \quad . \quad (21)$$

in which s_1 is the permitted compressive stress parallel to the grain and s_2 is the permitted compressive stress perpendicular to the grain.

For non-graded and "Grade 1,200 lb. *f*" timbers this formula becomes, respectively:

$$s = 165 + 635 \sin^2 \alpha \quad . \quad . \quad . \quad . \quad . \quad (22)$$

and

$$s = 325 + 875 \sin^2 \alpha \quad . \quad . \quad . \quad . \quad . \quad (23)$$

Graphs are given in Chapter 35, from which working stresses can be obtained directly for loads applied at any angle on the grain of "non-graded" and "Grade 1,200 lb. *f*" timbers.

Column Stresses.—The slenderness ratio of a post or strut has been defined as the ratio of the length to the thickness. The Timber By-laws do not indicate how the length is to be derived for the purpose of computing the slenderness ratio; but there does not appear to be any reason why it should not be determined in identically the same manner as for steel or reinforced concrete columns. Under By-laws 85 and 102 and in Chapter 15 this matter has been discussed at some length, and in order to avoid repetition the reader is referred to those notes. The "thickness" is to be taken as the least transverse dimension about which the tendency to buckle will be greatest, and this matter also has been explained under By-laws 85 and 102.

No post or strut must have a slenderness ratio greater than 40. In terms of the radius of gyration this limit is about 138, whereas that permitted for structural steel columns is 150 and that for reinforced concrete 120. These figures have been mentioned just for purposes of comparison.

When the slenderness ratio of a column, post or strut exceeds 10, the stress must not exceed the value given in Table 44 for the appropriate slenderness ratio. The index figures refer to the notes following the table.

TABLE 44

Slenderness Ratio		Maximum Compression Stress, lb. per sq. in.	
Exceeding	but not exceeding	Non-graded Timber	"Grade 1,200 lb. *f*" Timber[1]
10	12	785	985
12	14	775	970
14	16	755	950
16	18	725	920
18	20	690	875
20	22	635	820
22	24	565	745
24	26	485	650
26	28	420	565 [2]
28	30	365	485
30	32	320	430
32	34	285	380
34	36	255	340
36	38	225	300
38	40	205	275

Note 1.—It is not clear how these values have been derived because, although at the upper and lower limits of the slenderness ratio they bear some reasonable relationship to the permitted compressive stresses in the non-graded timbers, they yet vary quite considerably from the proportion which they should have to the stresses in non-graded timbers for slenderness ratios about 25 to 30. Nevertheless although this discrepancy is marked and attention is drawn, formally, to it, it is not considered that any difficulties should derive from it in practice.

Note 2.—This value has been introduced by the authors in place of that of 600 lb. per sq. in. which is given in the actual Timber By-laws. A misprint has occurred in the official publications, and therefore the amended value has been substituted.

The graphs given as Fig. 266 have been plotted from the values given in Table 44 and have been constructed as a " fair curves."

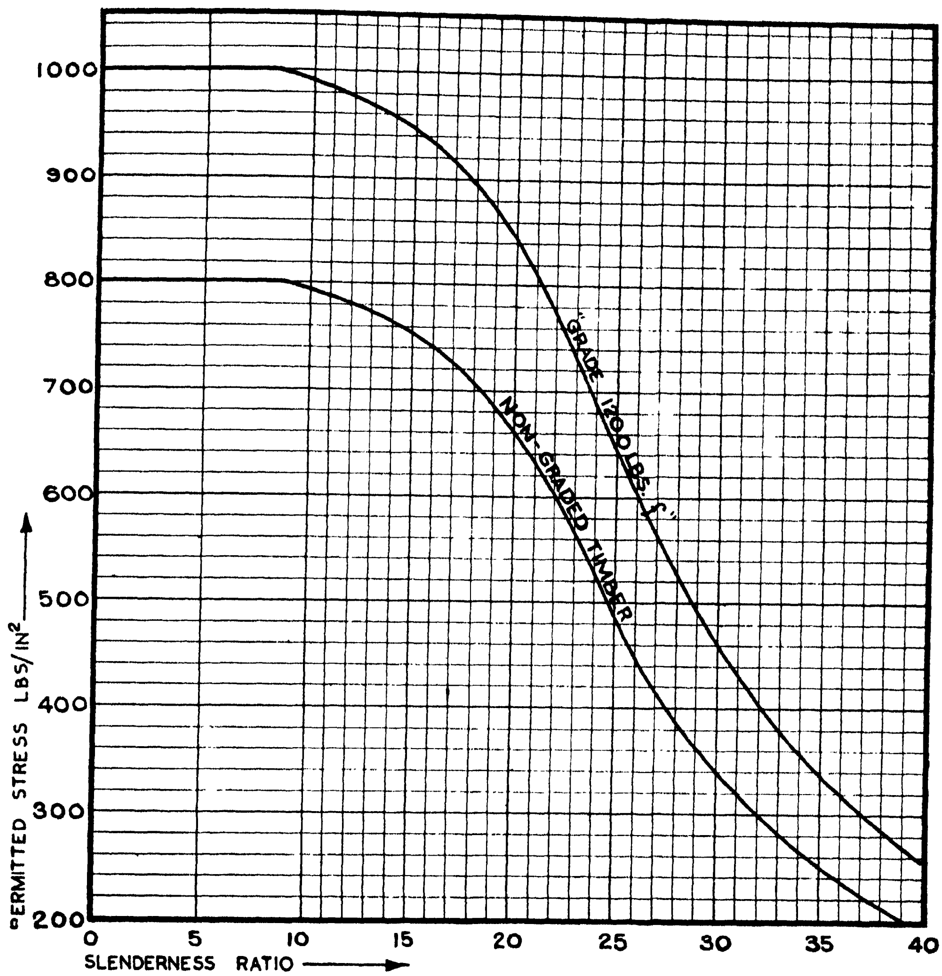


FIG. 266. Permitted column stresses.

Timber By-law 9. Maximum Slenderness Ratio of Posts.—" The slenderness ratio of any post or strut shall not exceed 40."

This requirement has been discussed under Timber By-law 8, p. 198.

Timber By-law 10. Combined Direct Loads and Bending on Posts.—" When a post or strut is subject to bending action in addition to axial loading, such post or strut shall be of a section sufficient to withstand such bending action in addition to the axial load."

The question at once arises : what permitted stress is to be adopted in a post subjected to both direct and bending stresses ? There is no indication given in the by-law. For such conditions in structural steel By-law 87 gives a formula from which the permitted stress may be derived, the values of the calculated individual direct and bending stresses and the normal permitted stress as determined by the slenderness ratio all being used in this formula.

Exactly the same method may be applied to timber posts, and formulæ have been derived by the authors giving the same relationships of stresses as in the case of steel columns.

The formulæ are:

Non-graded timber, $F_2 = fc + 908\left(1 - \frac{fc}{F_1}\right)\left(1 - \cdot 0123\frac{L}{b}\right)$. (24)

and "Grade 1,200 lb. f" timber:

$$F_2 = fc + 1123\left(1 - \frac{fc}{F_1}\right)\left(1 - \cdot 0112\frac{L}{b}\right) \quad (25)$$

in which F_2 = the required permitted combined direct and bending stress in a timber post.

fc = the total longitudinal load in pounds divided by the gross cross-sectional area of the post.

F_1 = the permitted stress for an axial load in a post or strut having a slenderness ratio $\frac{L}{b}$. These stresses are given in Table 44 or the Graph, Fig. 266.

and $\frac{L}{b}$ = the slenderness ratio of the post. It is assumed that the Council would accept this method of dealing with such conditions in timber posts.

From the formulæ, the graphs given in Figs. 267 and 268 have been prepared to facilitate the calculating of such cases. For the calculated value of fc, in any case, one must follow up the vertical line from that value until the appropriate slenderness ratio line is met and then from the point of intersection the horizontal line must be followed to the left where the permitted combined stress F_2 can be read off.

NON-GRADED TIMBER

Direct and bending stresses in columns.

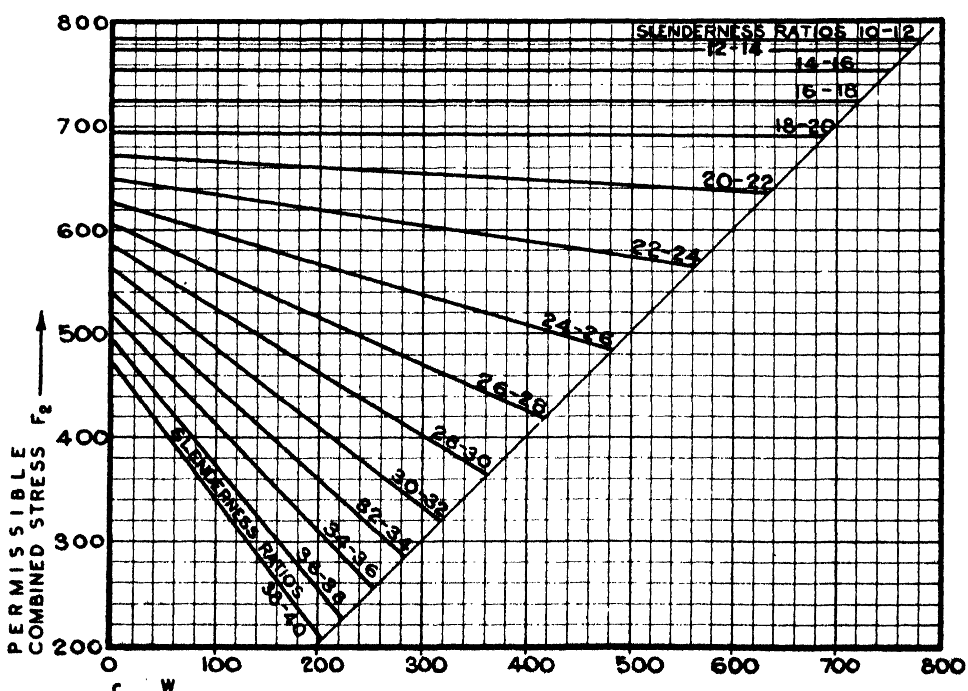


FIG. 267

"GRADE 1,200 LBS. *f*" TIMBER

Direct and bending stresses in columns.

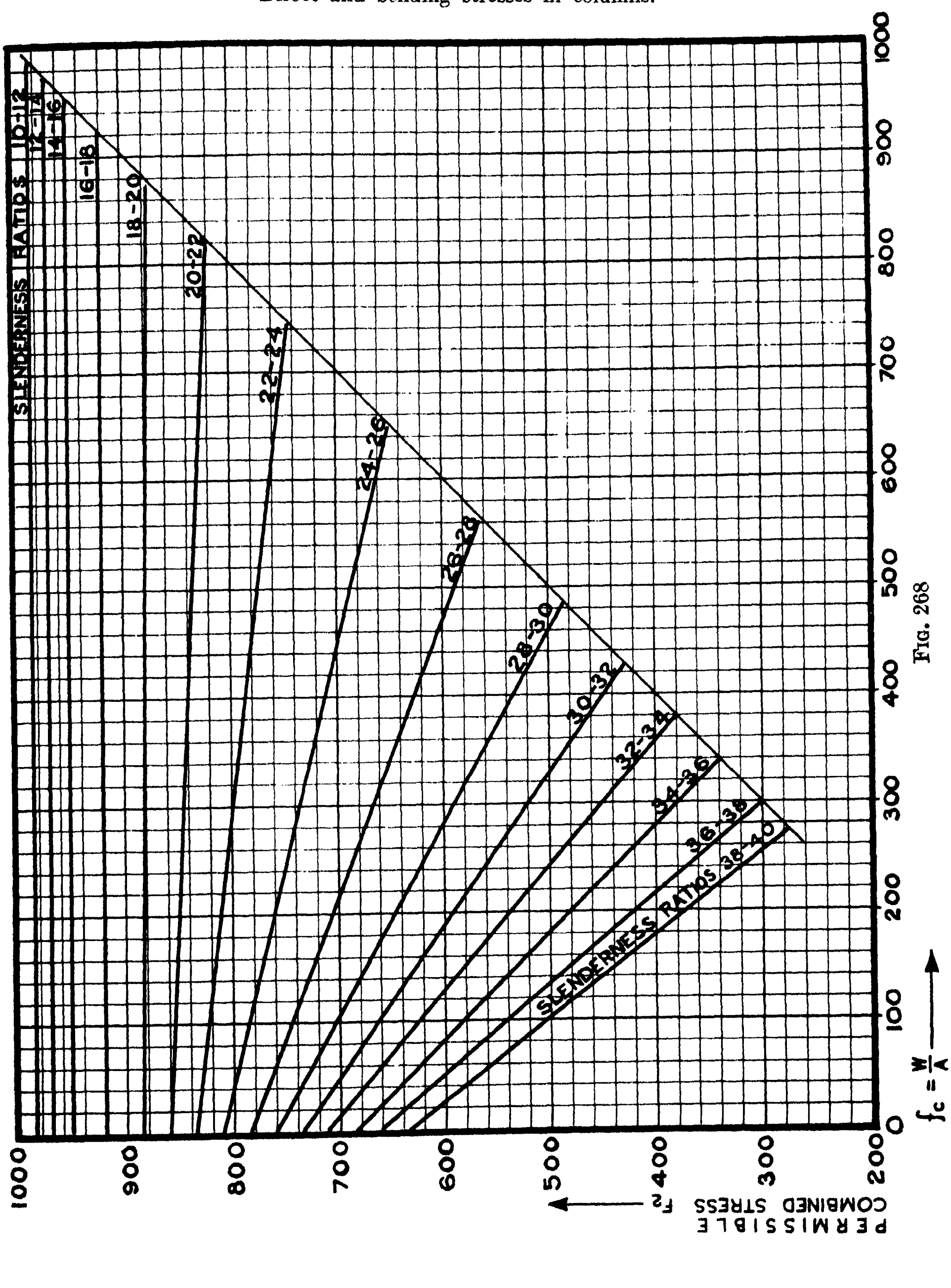


FIG. 268

The sum of the calculated bending stress (found by dividing the bending moment by the section modulus of the post) and the direct stress fc must in no case be greater than F_2.

Timber By-law 11. Maximum permitted Deflections of Beams, etc.—" A beam or cantilever (which for the purpose of this by-law shall be deemed to include a binder, joist, purlin, rafter, floor-board and boarding to flat roofs) shall be of such dimensions that the calculated deflection of such beam under the proper loading shall not exceed $\frac{1}{360}$ of its length."

It is impracticable to give criteria for all conditions of span and loading ; but the data in Table 45 will probably be of assistance to the designer, as they apply to the commonest cases.

In Table 45, l represents the span of the beam, etc., or the length of the cantilever, and α represents the ratio of the actual stress to the full permitted stress.

TABLE 45

Span or length l	Load	The Depth must not be less than :	
		Non-graded Timber	" Grade 1,200 lb. f " timber
Single span,on simple supports.	Uniformly distributed on whole span.	$\frac{l\alpha}{20}$	$\frac{l\alpha}{17 \cdot 8}$
Single span on simple supports.	Point load at centre of span.	$\frac{l\alpha}{25}$	$\frac{l\alpha}{22 \cdot 2}$
Cantilever.	Uniformly distributed on whole length.	$\frac{l\alpha}{8 \cdot 3}$	$\frac{l\alpha}{7 \cdot 4}$
Cantilever.	Point load at free end of cantilever.	$\frac{l\alpha}{6 \cdot 25}$	$\frac{l\alpha}{5 \cdot 6}$

These rules apply only to the conditions specified, and in the case of cantilevers it has been assumed that the fixed ends are restrained in a line at right angles to the line of action of the load—that is to say, there must be no initial slope in a cantilever at its support. In most cases a cantilever will be the projecting portion of a beam or other timber on an adjacent span and its slope at the support will be dependent on the length, loading and supports of that adjacent span, and, in consequence, it may have an initial slope. Such cases must be investigated separately for their individual peculiarities and treated on their merits.

Whereas in a beam the area under compression will usually be braced laterally by other structural members, most commonly by boarding or by the timber joists which it supports, there frequently occur in practice cases in which timber beams are not thus supported laterally.

There is nothing in the Timber By-laws restricting the length of an unsupported compression side of a beam ; but it is very unsound engineering to use long and narrow unrestrained beams, and the authors therefore strongly recommend the limiting of the ratio of the length to the width of the beam to a maximum value of 50. Also, above a ratio of 20 it is considered that the permissible stress in the timber should be reduced in the same manner as for steel beams.

Formulæ, parallel to that given in By-law 81, are :
Non-graded timber :

$$f_b = 1100 - 15\frac{L}{b} \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

and " Grade 1,200 lb. f " timber :

$$f_b = 1650 - 22{\cdot}5\frac{L}{b} \quad . \quad . \quad . \quad . \quad . \quad (27)$$

in which

f_b = recommended stress for bending in a beam, where

$\frac{L}{b}$ = ratio of unsupported span or length to width, lying between 20 and 50.

Table 46 gives the suggested stresses based on these formulæ.

TABLE 46

$\frac{L}{b}$	Recommended Beam Stress	
	Non-graded timber	" Grade 1,200 lb. f " Timber
20	800	1200
21	785	1177
22	770	1155
23	755	1132
24	740	1110
25	725	1087
26	710	1065
27	695	1042
28	680	1020
29	665	997
30	650	975
31	635	952
32	620	930
33	605	907
34	590	885
35	575	862
36	560	840
37	545	817
38	530	795
39	515	772
40	500	750
41	485	727
42	470	705
43	455	682
44	440	660
45	425	637
46	410	615
47	395	592
48	380	570
49	365	547
50	350	525

SECTION 3

This section contains Timber By-laws 12 to 19, and the requirements apply to all those rafters and purlins in pitched roofs, roof and floor-boards, joists and binders that have not been determined by calculations under the conditions of Section 2.

Under Section 3 the sizes of such members can be determined more or less empirically by rules, which, however, may only be applied when the general conditions contained in Timber By-law 12 and the particular conditions given in other by-laws of the Section are satisfied.

Timber By-law 12. Scope of Section 3. General Conditions.—" The provisions of this section shall be subject to the following requirements :

" (*a*) Pitched roofs shall be covered with ordinary slates, tiles, asbestos-cement, or other material of not greater weight.

" (*b*) Flat roofs shall be covered with lead, copper, zinc, or not more than 1 in. thickness of asphalte, or other materials of not greater weight.

" (*c*) Floors and flat roofs shall be constructed of boarding on joists or firring and shall not be pugged with concrete or otherwise subjected to abnormal dead loading ; but this requirement shall not preclude the use of plastering, parquet or other light ceiling and floor covering."

In those cases in which pugging is necessary, as for instance under By-law 141 (3), where a habitable room may be over a garage or stable, the joists and other members carrying the floor must then be determined by the calculations under Section 2 of these Timber By-laws.

In addition to the foregoing requirements the actual loads on members determined under Section 3 must not exceed those specified in Table 42, p. 196, and a moving load is to be treated as indicated in Note 2 to Timber By-law 7.

Timber By-law 13. Sizes of Members.—" For the purposes of this section in these by-laws the dimensions of a piece of timber shall be the actual measurement and shall not be taken as any ' nominal ' or ' scant ' size."

It commonly happens that the actual dimensions of a piece of timber are as much as half an inch shorter than the nominal dimensions. The rules given in this Section must not be applied, therefore, to the nominal sizes of timbers, but to the actual sizes.

" Where a piece of timber is morticed or notched or its sectional area is in any way reduced in such a manner as materially to impair its strength, the required size shall be so increased as to compensate for any such reduction in sectional area to the satisfaction of the district surveyor."

Timber By-law 14. Timbers to be Rectangular and in One Length between Supports.—" Every rafter, purlin, joist and binder shall be rectangular in section and shall extend in one piece between its supports, and, except as provided in By-law 19 shall be supported at both ends."

This requirement does not mean that two or more rectangular sections cannot be used side by side if they are bolted or secured so that they will work together as one wide member. In this case, of course, every piece would have to be at least $1\frac{3}{4}$ in. thick, which is the minimum thickness required by Timber By-law 4 for every such joist, etc.

The intention of this by-law is to prohibit splices in the lengths of joists, etc., at points between the supports. If suitable lengths cannot always be obtained, intermediate supports over which splices can be made must be introduced.

Timber By-law 15. Measurement of Spacing of Joists, etc.—" The spacing of rafters, purlins, joists and binders shall be measured in the clear. Where such spacing on one side of any such timber is not equal to that on the other side, the spacing for the purpose of this section of these by-laws shall be taken as an average of the two spacings."

Thus in Figs. 269 and 270:

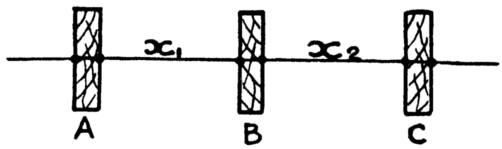


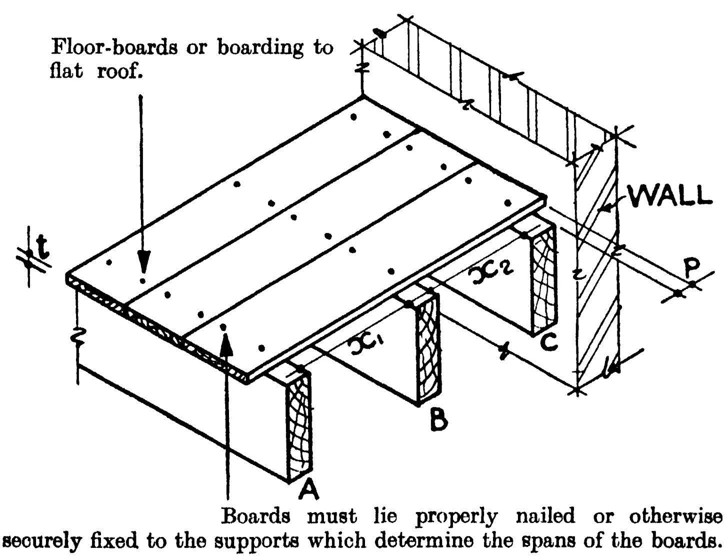
FIG. 269

x_1 = spacing between members A and B,
x_2 = ,, ,, ,, B ,, C,

If x_1 and x_2 are unequal the average spacing $\left(x = \frac{x_1 + x_2}{2}\right)$ must be taken as that to be used under this Section, and it must not be greater than the determined spacing.

It should be observed, although this Timber By-law does not say so directly, that this average spacing is to be maintained between every three consecutive members. Thus a floor, for example, with joists closely spaced at one part and widely at another would, rightly, not be permitted even though the average spacing of joists for the whole floor conformed to the determined spacing. The average spacing between any three members must be within the specified value.

Timber By-law 16. Fixings of Boards and Firring.—" Floor-boards and boarding to flat roofs shall have a thickness of not less than ⅝ in. and shall be properly nailed or otherwise securely attached to every joist, firring or other fixing which

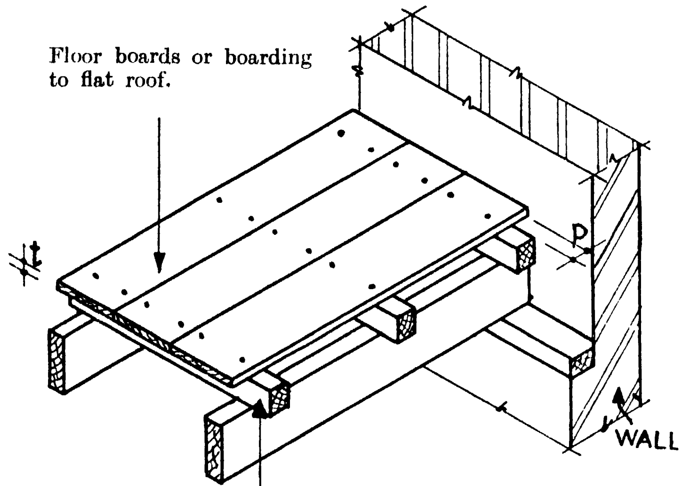


t must not be less than ⅝ in.
t must not be less than $\frac{x_1}{24}$ or $\frac{x_2}{24}$ } whichever is the greater.
(Timber By-law 17)
p must not be greater than $3t$.

If there is no wall at the ends of the boards there must not be any overhang—that is to say, there must be a joist or other support directly under the ends.

FIG. 270

determines the span of such boards, and they shall be similarly attached at the ends to such support except in the case of the joists or firrings next a wall, in which case the ends of the boards may project beyond the joists or firrings to an extent not exceeding three times the thickness of the boards. Where boarding is attached to firring, the firring shall be nailed or otherwise securely attached to the joists."



t must not be less than $\frac{5}{8}$ in.
t must not be less than $\frac{x_1}{24}$ or $\frac{x_2}{24}$ } whichever is the greater.
(Timber By-law 17.)
p must not be greater than $3t$.

If there is no wall at the ends of the boards there must not be any overhang—that is to say, firring or other support must be provided directly under the ends.

FIG. 271

Timber By-law 17. Thickness of Boards to Floors and Flat Roofs.—"The span of the boarding of flat roofs and floors shall be taken as the clear dimensions between the joists or other supports and shall not exceed 24 times the thickness of the boarding."
See Figs. 270 and 271.

Timber By-law 18. Sizes and Spacing of Timbers.—Tables 47 and 48 are to be used to determine the dimensions of certain sections. There are eleven classes of members which may thus be determined. They are:

A. Rafters, purlins and ceiling joists.
B. Joists to flat roofs.
C. Binders to flat roofs and joists and binders to residential floors.
D. Joists to office floors above the entrance floor.
E. Binders to office floors above the entrance floor.
F. Joists to offices on and below entrance floor and to retail shops and the like, and garages for private cars of not more than $2\frac{1}{4}$ tons net weight.

TABLE 47

NON-GRADED TIMBER

The values in the body of the table represent the spacing divided by the breadth.

l/d	A	B	C	D	E	F	G	H	I	J	K
5	—	—	—	—	—	—	—	—	—	—	15
6	—	—	—	—	—	—	—	24½	16	20	12
7	—	—	45	—	38½	—	25	21	13½	17	10
8	64	28	39½	22	33½	20	22	18	12	14½	9
9	56	24½	34½	19	29½	17½	19	15½	10½	12½	7½
10	45	19½	27½	15	23½	14	15	12½	8	10	6
11	37	16	22½	12½	19	11	12½	10	6½	8	4½
12	31	13½	19	10	16	9	10	8½	5	6½	3½
13	26	11	16	8½	13½	7½	8½	7	4	5½	—
14	22½	9½	13½	7	11½	6½	7	6	3½	—	—
15	19½	8	11½	6	10	5½	—	5	—	—	—
16	17	7	10	5	8½	4½	—	4	—	—	—
17	15	6	9	4½	7½	4	—	—	—	—	—
18	13	5	7½	—	6½	—	—	—	—	—	—
19	11½	4½	6½	—	—	—	—	—	—	—	—
20	10½	4	6	—	—	—	—	—	—	—	—
21	9	—	5	—	—	—	—	—	—	—	—
22	7½	—	4	—	—	—	—	—	—	—	—
23	6½	—	—	—	—	—	—	—	—	—	—
24	5½	—	—	—	—	—	—	—	—	—	—
25	4½	—	—	—	—	—	—	—	—	—	—
26	4	—	—	—	—	—	—	—	—	—	—

TABLE 48

"GRADE 1,200 LB. f" TIMBER

The values in the body of the table represent the spacing divided by the breadth.

l/d	A	B	C	D	E	F	G	H	I	J	K
5	—	—	—	—	—	—	—	—	—	—	16½
6	—	—	—	—	—	—	—	27½	19	23½	13½
7	—	—	50	—	43	—	—	23	16	20	11½
8	71½	31½	44	24½	37½	22½	24½	20	14	17½	10
9	63	28	39	21½	33	19½	21½	18	12	15	8½
10	57	25	35	19½	30	17½	19½	16	11	13½	7½
11	51½	22½	31½	17½	27	16	17½	14½	10	12½	7
12	47	20½	29	16	24½	14½	16	13	9	11	6
13	40	17½	24½	13½	21	12	13½	11	7½	9½	5
14	34½	15	21	11½	18	10½	11½	9½	6	8	4
15	29½	12½	18	9½	15½	9	9½	8	5½	6½	3½
16	26	11	15½	8½	13½	7½	8½	7	4½	—	—
17	23	9½	14	7½	11½	6½	7½	6	4	—	—
18	20	8½	12	6½	10	5½	—	5	—	—	—
19	16½	7	10	5	8½	4½	—	4	—	—	—
20	14½	6	8½	4½	7	4	—	—	—	—	—
21	12	5	7	—	6	—	—	—	—	—	—
22	10½	4	6	—	—	—	—	—	—	—	—
23	9	—	5	—	—	—	—	—	—	—	—
24	7½	—	4½	—	—	—	—	—	—	—	—
25	6½	—	3½	—	—	—	—	—	—	—	—
26	6	—	—	—	—	—	—	—	—	—	—
27	5	—	—	—	—	—	—	—	—	—	—
28	4½	—	—	—	—	—	—	—	—	—	—
29	4	—	—	—	—	—	—	—	—	—	—

G. *Binders to offices on and below entrance floor and to retail shops and the like, and garages for private cars of not more than $2\frac{1}{4}$ tons net weight.*

H. *Joists and binders to corridors and landings.*

I. *Joists to workshops and factories and the like and garages for motor vehicles other than private cars of not more than $2\frac{1}{4}$ tons net weight.*

J. Binders to workshops and factories and the like, and garages for motor vehicles other than private cars of not more than $2\frac{1}{4}$ tons net weight.

K. Joists and binders to warehouses, book stores stationery stores and the like.

These different classes have been indicated in Tables 47 and 48 (which refer to non-graded and "Grade 1,200 lb. *f*" timbers respectively) only by the index letters given above.

Case 1.—When the (i) spacing, (ii) width, and (iii) span have previously been fixed, the depth of each timber will be found by the following steps :

(*a*) by dividing the spacing by the width ;

(*b*) by finding in the appropriate column for the correct class of member, the number having this value or the next higher number if the exact number is not given ; and then

(*c*) by going horizontally to the left at this level to read off in the margin a value which is the ratio of the span to the depth ;

(*d*) divide the span by this value and the depth will be given.

Example.—Non-graded timber, $2\frac{1}{2}$ in. wide, is to be used as binders at 4 ft. spacing for a corridor floor over a 6-ft. span.

Then $\frac{48}{2\cdot5} = 19\cdot2$. In column H of Table 47 this value is not given, but the adjacent higher value is 21, and this gives on the left a value of 7. Therefore the depth of the binders must be $\frac{6 \times 12}{7} = 10\frac{1}{2}$ in.

Case 2.—When the (i) spacing, (ii) depth, and (iii) span have previously been fixed, the width of each timber will be found by the following steps :

(*a*) by dividing the span by the depth.

(*b*) by finding under the left-hand column the number having this value, or the next higher number if the exact number is not given, and then

(*c*) by going horizontally to the right at this level to read off, in the appropriate column for the correct class of member, a value which is the ratio of the spacing to the width ;

(*d*) divide the spacing by this value and the width will be given.

Example.—"Grade 1,200 lb. *f*" timber, $11\frac{1}{2}$ in. deep, is to be used for the floor joists, at 15-in. spacings, in a workshop over a span of 14 ft. 3 in.

Then $\frac{14\cdot25 \times 12}{11\cdot5} = 14\cdot9$. From the value of 15 in the left-hand column of Table 48 go to the right to find at the same level in Column I the value $5\frac{1}{2}$. Therefore the widths of the joists must be :

$$\frac{15}{5\cdot5} = 2\cdot72 \text{ in., say } 2\tfrac{3}{4} \text{ in. or 3 in. nominal.}$$

Case 3.—When the (i) depth, (ii) width, and (iii) span have previously been fixed, the spacing of the timbers will be found by the following steps :

(*a*) by dividing the span by the depth ;

(*b*) by finding under the left-hand column the number having this value, or the next higher number if the exact number is not given ; and then

(*c*) by going horizontally to the right at this level to read off, in the appropriate column for the correct class of member, a value which is the ratio of the spacing to the width ;
(*d*) multiply the width by this value and the spacing will be given.

Example.—Non-graded timbers, each 2 in. wide and 9 in. deep, are to be used as joists for a flat roof of 11-ft. span.

Then $\frac{11 \times 12}{9} = 14{\cdot}7$. From the value of 15 in the left-hand column of Table 47 go to the right to find at the same level in column B the value 8. Therefore the spacing of the joists will be $8 \times 2 = 16$ in.

Case 4.—When the (i) depth, (ii) width, and (iii) spacing have previously been fixed, the span or the distance between the supports of such timbers will be found :

(*a*) by dividing the spacing by the width, then
(*b*) by finding the appropriate column for the correct class of member the number having this value, or the next higher number if the exact value is not given, and then
(*c*) by going horizontally to the left at this level to read off in the margin a value which is the ratio of the span to the depth.
(*d*) Multiply the depth by this value and the span will be given.

Example.—" Grade 1,200 lb. *f* " binders, 5 in. wide and 7 in. deep, are to be used in an upper floor of an office at a spacing of 8 ft.

Then $\frac{8 \times 12}{5} = 19{\cdot}2$. In Column E of Table 48 this value is not given, but the adjacent higher value is 21, and this gives in the left-hand column a value of 13. Therefore the span of the binders can be up to $\frac{13 \times 7}{12} = 7$ ft. 7 in.

It may so happen that the dimensions and spacings of timber members determined under this by-law, whilst satisfying all the requirements of these rules and being, therefore, in accordance with the Timber By-laws, may yet be quite unsuitable for other reasons. For example, $2\frac{3}{4}$-in. × 8-in. " Grade 1,200 lb. *f* " joists at 12-in. spacing in a residential floor (i.e. at 15-in. centres) would be permitted on a span up to 16 ft. It is obvious that such a shallow floor would have excessive deflection and be subject to very objectionable vibration, set up even by the movement of people on it.

Therefore these empirical rules are to be employed with discretion ; and, in such cases as that of the example just given, it is suggested that the conditions of Timber By-law 11 should be observed. This is not a legal requirement ; but it is just one of the simple precautions which it should be the duty or aim of every designer to take. This does not mean that complicated calculations for deflection need be made, as the criteria given in Table 45, p. 202, will serve as the necessary check in the majority of cases.

Timber By-law 19. Cantilevers.—When a timber rafter, purlin joist or binder is projected beyond a support so as to form a cantilever, then

(*a*) the timber must continue for the full length of the cantilevered portion without change of section ;
(*b*) the cantilevered portion together with all the superimposed loads must be counterbalanced by the dead load of the other portion—(see Note 1 p. 210) ;

and

(*c*) the length of the cantilevered portion (which is to be taken as the clear distance between the face of the support and the end of the cantilever) must not be greater than one-quarter of the length which would have been permitted by

Timber By-law 18 for a similar member on a simply supported span (see example given in Note 2 below).

Note 1 (see clause (*b*)p. 209).—In Fig. 272 :

l_2 = the length of the cantilever (see clause (*c*) p. 209).

If WD_2 = the resultant of all the dead loads of the cantilever,
and if b = the distance of WD_2 from the centre of the support of the cantilever,
and if WL_1 = the resultant of all the live loads on the cantilever,
and if c = the distance of WL_1 from the centre of the support of the cantilever,
and if WD_1 = the resultant of all the dead loads of the adjacent span,
and if a = the distance of WD_1 from the centre of the support of the cantilever,
then :

$$\propto (WD_2 \times b) + (WL_1 \times c) \text{ must not be greater than } (WD_1 \times a)^*$$

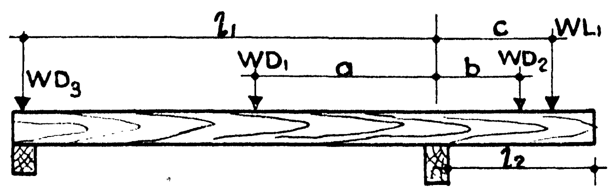


FIG. 272

If WD_1 itself is insufficient to resist the overturning of the cantilever, but if the support A of the adjacent span is capable of resisting an uplift (by suitable anchorage or by the imposed local dead weight of other portions of the structure) equal to a force WD_3, then WD_3 may be allowed to assist in the counter-balancing such that :

$$(WD_1 \times a) + (WD_3 \times l_1) \text{ must not be less than } \propto (WD_2 \times b) + (WL_1 \times c)^*$$

Note 2 (see clause (*c*) p. 209).—

Example.—3-in. × 9-in. non-graded timber joists, at 15-in. spacing, carrying a flat roof, are to be cantilevered over a support afforded by a brick wall.

Such joists at the same spacing, on a simply supported span, would be permitted on a span up to 13 ft. 6 in. in length (see Case 4, Timber By-law 18, p. 209). Therefore the length of the cantilevered portion could be up to 3 ft. 4½ in.

It is of interest to compare this value with that which would be permitted under Section 2 of these Timber By-laws, that is to say the value permitted by exact calculations. Assuming a total dead weight of 25 lb. per sq. ft., which is a reasonable amount, and the same joists at the same distances, i.e. 18-in. centres, and taking the superimposed load, 50 lb. per sq. ft. as specified in Timber By-law 7, and the permitted stress 800 lb. per sq. in. in Timber By-law 8, it is found that the length of the cantilever from the centre of the support can be 6 ft. 11 in., or say 6 ft. 6 in. from the face. This value exceeds that which is required by Timber By-law 11, namely, 8·3 × 9 in. or 6 ft. 2¼ in. from the centre of the support or say 5 ft. 10 in. from the face. However, even this latter value is 73 per cent. greater than that determined under Timber By-law 19.

Thus, again, it becomes clear that the empirical rules given in Timber By-laws 18 and 19 are to be used with the greatest discretion. The essential demands for economy and sound construction both point to the wisdom of adopting the requirements of Section 2 (exact calculations) in structural timber design, rather than the empirical rules of Section 3, which latter are so inconsistent in their results as to allow a construction hopelessly flimsy in one case and equally conservative and uneconomical in another.

* $\propto$ is the factor of safety which should not be less than 1·3.

Section 4. General

Timber By-law 20.—Conversion of Buildings.—" It shall not be lawful to convert a building or any part thereof in such manner that such building or part of a building when so converted will not be in conformity with these by-laws."

Timber By-law 21. Notice to District Surveyor.—" Before any building or part of a building is converted, notice in writing of such proposed conversion shall be given to the district surveyor by the owner or occupier of the building or part of the building in question."

Timber By-law 22. Duties of District Surveyor.—" For the purpose of securing the due observance of the provisions of these by-laws, it shall be the duty of the district surveyor to survey any building or work affected thereby, and he may require the builder to supply him with all such plans, calculations or other particulars and samples of timber as he may reasonably require."

Timber By-law 23. Workmanship.—" All work affected by any of the provisions of these by-laws shall be carried out to the satisfaction of the district surveyor in a proper and workmanlike manner."

Timber By-law 24. Penalties.—" Every person who contravenes or fails to comply with any of the provisions of these by-laws or of any requirement made thereunder or in pursuance thereof shall be deemed to have committed an offence against these by-laws and shall be liable on summary conviction to a penalty not exceeding the sum of £50 and a daily penalty not exceeding £10."

CHAPTER 14

LONDON BUILDING ACT SECTIONS

In the following pages extracts are given from the London Building Act, 1930, and from other enactments to the requirements of which it may be necessary for a building or any part of it to conform. In the case of Section 79 of the Act, the authors have introduced some explanatory sketches.

These extracts contain all sections of Parts VI and VII of the Act, which have not been replaced by the by-laws (see By-law 158, p. 185, and also Chapter 1, p. 3) and they contain also sections from other parts of the Act which relate to Parts VI and VII, i.e. to the construction of buildings and to other technical requirements of buildings.

Part I. Section 5. Definition.—" In this Act, save as is otherwise expressly provided therein and unless the context otherwise requires, the following expressions have the meanings hereby respectively assigned to them (that is to say):

" basement storey: means any storey of a building which is under the ground storey;

" bressummer: means a wooden beam or a metallic girder which carries a wall;

" builder: means the person who is employed to build or to execute work on a building or structure, or where no person is so employed the owner of the building or structure;

" building of the warehouse class: means a warehouse, manufactory, brewery, or distillery or any other building exceeding in cubical extent 150,000 cu. ft., which is neither a public building nor a domestic building;

" building owner: means such one of the owners of adjoining land as is desirous of building, or such one of the owners of buildings, storeys, or rooms separated from one another by a party wall or party structure as does or is desirous of doing a work affecting that party wall or party structure;

" city: means all parts within the jurisdiction of the Common Council as the successors of the Commissioners of Sewers of the city;

" cross wall: means a wall used or constructed to be used, in any part of its height, as an inner wall of a building for separation of one part from another part of the building: that building being or being constructed or adapted to be wholly in one occupation;

" cubical extent: in relation to the measurement of a building: means the space contained within the external surfaces of its walls and roof and the upper surface of the floor of its lowest storey;

" dangerous business: means the business of the manufacture of matches, or of other substances liable to sudden explosion, inflammation or ignition, or of turpentine, naphtha, varnish, tar, resin or Brunswick black, or any other manufacture dangerous on account of the liability of the substances employed therein to cause sudden fire or explosion;

" District surveyor: means any district surveyor appointed under this Act, and includes any person appointed under this Act to act as the deputy of or to assist any district surveyor so appointed;

"domestic building: includes a dwelling-house, and any other building not being either a public building or a building of the warehouse class;

"dwelling house: means a building used or constructed or adapted to be used wholly or principally for human habitation;

"External wall: means an outer wall or vertical enclosure of any building, not being a party wall;

"fire-resisting materials: means any of the materials described or referred to in the First Schedule to this Act;

"first storey: means that storey of a building which is next above the ground storey; the successive storeys above the first storey being the second storey, the third storey and so on to the topmost storey;

"foundation, in relation to a wall having footings: means the solid ground or artificially formed support on which the footings of the wall rest, but in the case of a wall carried by a bressummer means such bressummer;

"girder: means a metal girder or joist;

"ground storey: means that storey of a building to which there is an entrance from the outside on or near the level of the ground; and, where there are two such storeys, then the lower of the two; but does not include any storey of which the upper surface of the floor is more than 4 ft. below the level of the adjoining pavement;

"Guildhall: means the lands, offices, courts and buildings commonly called the Guildhall, and the offices, courts and buildings adjoining or appurtenant thereto, which now are used by or may hereafter be erected for the use of the Corporation, or of any committee, commission, or society appointed by them;

"habitable, in relation to any room: means constructed or adapted to be inhabited;

"height, in relation to any building: means the measurement taken from the level of the footway (if any) immediately in front of the centre of the face of the building, or (where there is no such footway) from the level of the ground before excavation, to the level of the top of the parapet or, where there is no parapet, to the level of the top of the external wall or (in the case of gabled buildings) to the base of the gable;

"high building: means any building of which any storey is an upper storey;

"inhabited, in relation to any room: means that the room is one in which some person passes the night, or which is used as a living-room, or with respect to which there is a probable presumption (until the contrary is shown) that some person passes the night therein or that it is used as a living-room;

"local authority, in relation to any building, structure, place, land or thing, or any proposed building, structure or thing: means the council of the metropolitan borough within which the building, structure, place, land or thing is, or the proposed building, structure or thing will be, or as respects the City the Common Council;

"London: means the administrative county of London;

"Metropolis Management Acts: means the Metropolis Management Acts, 1855 to 1893, as amended by any subsequent Act;

"noxious business: means the business of a blood boiler or bone boiler, or any other like business which is offensive or noxious; but does not include the business of a soap boiler, tallow melter, knacker, fellmonger, tripe boiler, or slaughterer of cattle or horses;

"occupier: does not include a lodger; and the expression 'occupy' and 'occupation' shall be construed accordingly;

"party arch: means an arch separating adjoining buildings, storeys or rooms belonging to different owners, or occupied or constructed or adapted to be occupied by different persons, or separating a building from a public way or a private way leading to premises in other occupations ;

" party fence wall : means a wall used or constructed to be used as a separation of adjoining lands of different owners, and standing on lands of different owners and not being part of a building, but does not include a wall constructed on the land of one owner the footings of which project into the land of another owner ;

" party structure : means a party wall, and a partition, floor, or other structure, separating vertically or horizontally, buildings, storeys or rooms approached by distinct staircases or separate entrances from without ;

" party wall : means—

(*a*) a wall forming part of a building and used, or constructed to be used, for separation of adjoining buildings belonging to different owners, or occupied or constructed or adapted to be occupied by different persons ; or

(*b*) a wall forming part of a building and standing to a greater extent than the projection of the footings on lands of different owners ;

" superintending architect : means the superintending architect of metropolitan buildings for the time being ;

" the Council : means the London County Council ;

" topmost storey : means the uppermost storey in a building, whether constructed wholly or partly in the roof or not ;

" Tribunal of Appeal : means the Tribunal of Appeal constituted in accordance with the provisions of this Act ;

" upper storey : means any storey whereof the level of the upper surface of the floor is at a greater height than 50 ft. above the level of the footway immediately in front of the centre of the face of the building in which the storey is situate, or if there is no such footway above the level of the ground before excavation."

PART VI

Section 57. Open Sheds.—" (2) Notwithstanding anything in this Act, open sheds not exceeding 16 ft. in height and not exceeding 4 squares in area may be constructed of any such materials and in any such manner as may be approved by the district surveyor."

Section 60. Openings in Party Walls.—" (4) An opening shall not be made in any party wall except in accordance with the provisions of this Act in relation thereto." See also p. 231.

Section 64. Walls deemed to be Party Walls.—" In either of the following cases :

" (*a*) where a wall has after the thirty-first day of December eighteen hundred and ninety-four been built as a party wall in any part ; or

" (*b*) where a wall has after that date become a party wall in any part ;

the wall shall be deemed a party wall to the extent of such part of its length as is used as such a wall."

Section 69. Restriction on Cutting Away of Chimney-breast or Shaft.—" (19) A chimney-breast or shaft built in or with any party wall shall not be cut away, unless the district surveyor certifies that it can be done without injuriously affecting the stability of any building."

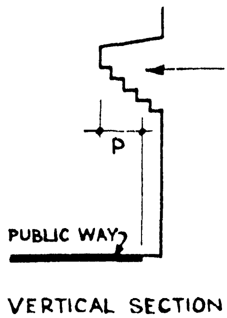
Section 73. Fire-resisting Corridors and Stairs.—" In every public building the floors of the lobbies, corridors, passages and landings, and the flights of stairs shall be of fire-resisting materials, and carried by supports of fire-resisting materials."

Section 74. Ventilation of Staircases.—" (1) In every building constructed or adapted to be occupied in separate tenements by more than two families, the principal staircase used by the several families in common shall be ventilated upon every storey above the ground storey by means of windows or skylights opening directly into the external air, or shall be otherwise adequately ventilated.

" (2) The principal staircase in every building being a dwelling-house, and not subject to the provisions of subsection (1) of this section, shall be ventilated by means of a window or skylight opening directly into the external air."

Section 79. Projection from Buildings.—" Unless the Council otherwise permit, the following provisions shall apply to projection from buildings :

" (2) No cornice shall exceed in projection 2 ft. 6 in. over the public way.



p must not be greater than 2 ft. 6 in.

FIG. 273

" (3) In a street or way, of a width not greater than 30 ft., any shop front may project beyond the external wall of the building to which it belongs to any extent not exceeding 5 in. and any cornice of any such shop front may project to any extent not exceeding 13 in. ; and in any street or way, of a width greater than 30 ft., any shop front may project to any extent not exceeding 10 in., and any cornice of any such shop front may project to any extent not exceeding 18 in. beyond the external wall of the building to which it belongs over the ground of the owner of the building."

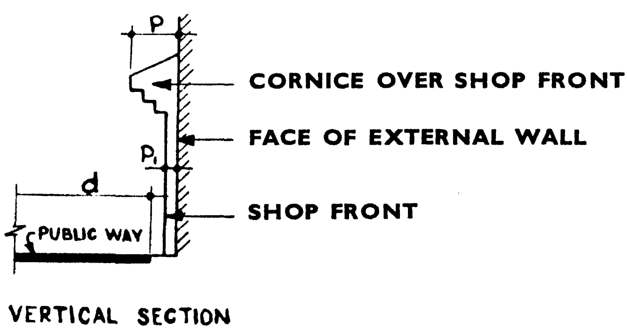


FIG. 274

d = width of street or public way. If d is not greater than 30 ft.:

p must not be greater than 13 in.
p_1 ,, ,, ,, 5 in.

If d is greater than 30 ft. :

p must not be greater than 18 in.
p_1 ,, ,, ,, 10 in.

"Provided that nothing in this paragraph shall authorise in any such street the projection of any part of any such shop front, other than the cornice, on or over the public way, or any land to be given up to the public way.

"(5) In any dwelling-house being, either situated in a street not less than 40 ft. wide or so situated in any street that the front wall of the dwelling-house is distant not less than 40 ft. from the opposite boundary of the street, bay windows may, notwithstanding the provisions of this Act relating to buildings beyond the general line of buildings in streets, be constructed on land belonging to the owner of the dwelling-house. Provided that no such bay windows shall:

"(*a*) exceed three storeys in height above the level of the footway of the street; or
"(*b*) project more than 3 ft. from the main wall of the dwelling-house; or
"(*c*) project in any part nearer to the centre of the roadway of the street than the prescribed distance; or
"(*d*) be in any part nearer to the centre of the nearest party wall than the extreme extent of their projection from the main wall of the dwelling-house; or
"(*e*) in the aggregate exceed in width three-fifths of the frontage of the building towards the street to which the bay windows face; or
"(*f*) be construed upon any part of the public way or upon any land agreed to be given up to the public way; or
"(*g*) be used for purposes of trade.

"Bay windows to which the foregoing provisions of this paragraph do not apply shall not be constructed unless the Council after consultation with the local authority consent thereto."

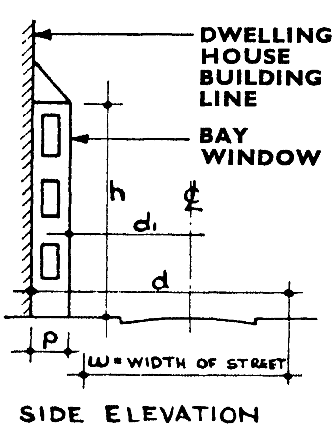


FIG. 275

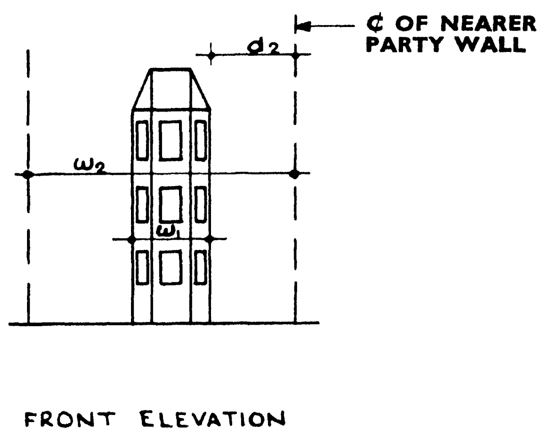


FIG. 276

If w is not less than 40 ft. or if w is less than 40 ft., but if d is not less than 40 ft. a bay window may be constructed such that:

(*a*) h is not greater than three storey heights,
and (*b*) p is not greater than 3 ft.,
and (*c*) d_1 is not greater than the distances prescribed by the Act,
and (*d*) d_2 is not less than p,
and (*e*) w_1 is not greater than $\frac{3}{5}w_2$,
and (*f*) p does not project on any public way or land,
and (*g*) window is not to be used for trade purposes.

Other bay windows may be constructed by agreement with the council.

"(6) In any building being either situate in a street not less than 40 ft. wide or so situate in any street that the front wall of the building is distant not less than 40 ft. from the opposite boundary of the street projecting oriel windows or turrets may be constructed, so, however, that:

"(*a*) no part of any such projection shall extend more than 3 ft. from the face of the front wall of the building or more than 12 in. over the public way; and

"(*b*) no part of any such projection shall be less than 10 ft. above the level of the footway of the street; and

"(*c*) no part of any such projection (where it overhangs the public way) shall be within a distance of 4 ft. of the centre of the nearest party wall; and

"(*d*) on no floor shall the widths of any such projections in the aggregate exceed three-fifths of the length of the wall of the building on the level of that floor; and

"(*e*) every such projection shall be constructed to the satisfaction of the district surveyor, or in the event of disagreement to the satisfaction of the superintending architect, whose determination shall be final.

"Oriel windows or turrets to which the foregoing provisions of this paragraph do not apply shall not be constructed unless the Council after consultation with the local authority consent thereto."

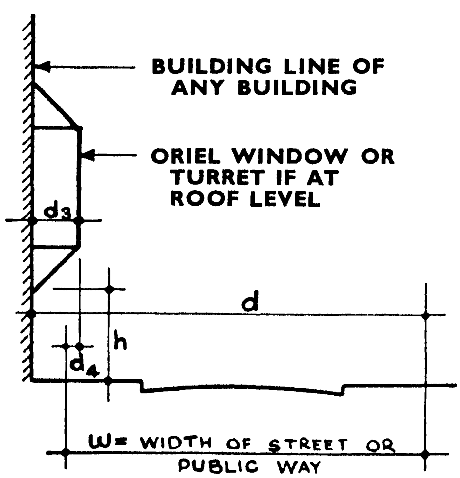


FIG. 277

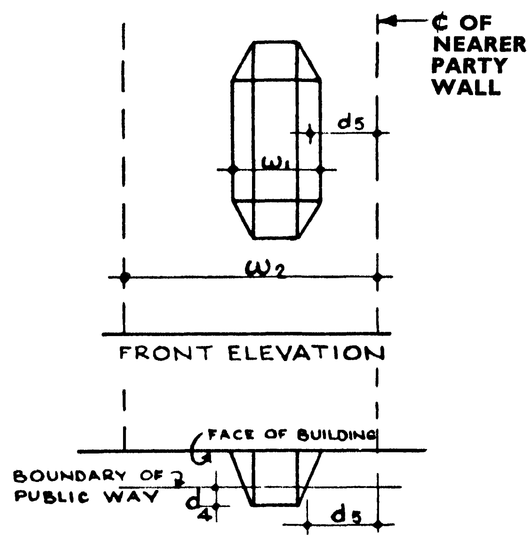


FIG. 278

If w is not less than 40 ft. or if w is less than 40 ft., but if d is not less than 40 ft., oriel windows or turrets may be constructed such that:

(*a*) d_3 is not greater than 3 ft. and d_4 is not greater than 12 in.
and (*b*) h is not less than 10 ft.
and (*c*) d_5 is not less than 4 ft.
and (*d*) w_1 is not greater than $\frac{3}{5}w_2$,
and (*e*) each projection will be constructed to the satisfaction of the district surveyor.

Other oriel windows or turrets may be constructed with the agreement of the Council.

"(7) The roof, flat, or gutter of every building, and every balcony, verandah, shop front or other similar projection, or projecting window shall be so arranged and constructed and so supplied with gutters and pipes as to prevent the water therefrom from dropping upon or running over any public way.

"(8) Except in so far as is permitted by this section in the case of shop fronts and projecting windows and with the exception of water pipes and their appurtenances, copings, string-courses, cornices, fascias, window dressings and other like architectural decorations, no projection from any building shall extend beyond the general line of buildings in any street unless the Council after consultation with the local authority consent."

Section 80. District Surveyor's Fees for Separate Chambers.—"Nothing in this subsection shall entitle the district surveyor to charge for the inspection of each set of chambers as a separate building."

Section 81. Cubical Extent of Buildings. (See also p. 228).—"(1) Subject to the provisions of this section no building of the warehouse class, and no building or part of a building, used for purposes of trade or manufacture, shall be of a cubical extent exceeding 250,000 cu. ft., unless it is divided by division walls in such manner that no division of the building or part of a building (as the case may be) is of a cubical extent exceeding 250,000 cu. ft., and no addition shall be made to any such building part of a building or division so as to make it contravene the provisions of this subsection.

"(2) Where the Council are satisfied on the report of the superintending architect and of the chief officer of the London Fire Brigade that additional cubical extent is necessary for any such building, part of building or division as aforesaid, and are satisfied that proper arrangements have been, or will be, made and maintained for lessening so far as reasonably practicable danger from fire, the Council may consent to such building, part of a building, or division containing additional cubical extent; but such consent shall continue in force only while such building, part of a building or division is actually used for the purposes of the trade or manufacture (if any) in respect of which the consent was granted.

"(3) The Council may, in any case in which they think fit so to do, consent to any such building or part of a building as aforesaid being divided (wholly or in part) horizontally by floors to be constructed in such manner and of such materials, and in all other respects as the Council may require or approve, and in such case such floors shall for the purposes of this section be deemed to be division walls.

"(4) The provisions of this section shall not apply to any building which, being at a greater distance than 2 miles from Saint Paul's Cathedral, is used wholly for the manufacture of the machinery and boilers of steam vessels or for a retort house, or for the manufacture of gas, or for generating electricity, and which consists of one floor only, and is constructed of brick, stone, iron or other incombustible materials throughout, and is not used for any purpose other than such purposes as are specified in this subsection.

"Every such building shall for the purposes of Part VII of this Act be deemed a building to which the general provisions of this Part of this Act are inapplicable."

Section 82. Uniting of Buildings.—"(1) Buildings shall not, without the consent of the Council, be united unless they are wholly in one occupation, and, when so united and considered as one building, would be in conformity with this Act.

"(2) An opening shall not be made in any division wall separating divisions of a building of the warehouse class, or used for purposes of trade or manufacture, or in any party wall, or in two external walls separating buildings, in any case in which such divisions or buildings (as the case may be) if taken together would extend to more than 250,000 cu. ft., except under the following conditions:

" (a) the opening shall have the floor, jambs, and head formed of brick, stone, iron or other incombustible materials, and be closed by two wrought-iron doors, sliding doors, or shutters each not less than ¼ in. thick in the panel, at a distance from each other of the full thickness of the wall, fitted to grooved or rebated iron frames, without woodwork of any kind;

and all such doors, sliding doors, and shutters shall be fitted with sufficient and proper bolts or other fastenings, and be capable of being opened from either side, and shall have on each face thereof styles and rails at least 4 in. wide and ¼ in. thick, and shall be constructed, fitted and maintained in an efficient condition.

" Provided that, in lieu of being constructed and fitted as aforesaid, such doors, sliding doors and shutters may be constructed of any such fire-resisting materials and be fitted in any such manner as may be approved by the Council.

" (b) The opening shall not exceed in width 7 ft. or in height 8 ft., and the width of any such opening in any wall of a storey (or if there is more than one such opening in any such wall, the widths of all such openings taken together) shall not exceed one-half of the length of such wall:

Provided that:—

" (i) any such opening may be 9 ft. 6 in. in height in a wall of which the thickness is not less than 24 in., or if the doors, sliding doors or shutters closing such openings are placed at a distance of not less than 24 in. from each other; and

" (ii) the Council may consent to any such opening being of such greater height or width as they may think fit.

" (3) Whenever any buildings which have been united cease to be in one occupation, the owner thereof, or, if the buildings are the property of different owners, then each such owners, shall forthwith give notice of such change of occupation to the district surveyor, and shall cause all openings uniting the same in any party wall or in any external wall to be stopped up (unless the Council consent to such openings or any of them being retained) with brick or stone work not less than 13 in. in thickness (except in the case of a wall 8½ in in thickness, in which case 8½ in. shall be sufficient), and properly bonded with such wall; and any timber placed in the wall and not in conformity with this Act shall be removed.

" (4) Buildings shall be deemed to be united when any opening is made in the party wall or the external walls separating such buildings, or when such buildings are so connected that there is access from one building to the other without passing into the open air.

" (5) The provisions of this section shall apply:

" (a) to all openings at any time after the thirty-first day of July nineteen hundred and eight made, or proposed to be made, in any party wall or two external walls or in any division wall, notwithstanding the existence in any such wall of an opening uniting buildings or affording communication between divisions of a building (as the case may be), and

" (b) to such buildings as if they had not been previously united."

Section 83.—Division Walls to be regarded as Party Walls.—" The provisions of this Act with respect to party walls shall apply to such division walls as are referred to in this Part of this Act.

"Provided that, in the case of any such division wall, the Council, may, if they think fit, consent to such departure as they may consider expedient from such of the said provisions as are contained in this Part of this Act and in the Second Schedule to this Act."

Section 84. Approved Plans to be sent by Council to District Surveyor.—" A copy of any plans and particulars approved by the Council under this Part of this Act shall be furnished by the Council to the district surveyor within whose district the building to which such plans and particulars relate is situate."

Section 85. Public Buildings.—" (1) Notwithstanding anything in this Act, every public building, including the walls, roofs, floors, galleries and staircases, and every structure and work constructed or done in connection with, or for the purposes of, the same, shall be constructed in such manner as may be approved by the district surveyor ; or in the event of disagreement may be determined by the Tribunal of Appeal ; and, save so far as respects the rules of construction, every public building shall throughout this Act be deemed to be included in the expression 'building' and be subject to all the provisions of this Act in the same manner as if it were a building other than a public building.

" (2) No public building shall be used as such until the district surveyor or the Tribunal of Appeal has or have declared his or their approval of the construction thereof.

" (3) After the district surveyor has so declared his approval of the construction or has certified that it complies with the directions of the Tribunal of Appeal, any work affecting or likely to affect the building shall not be done to, in or on the building without the approval of the district surveyor, or such certificate as aforesaid."

Section 86. Conversions into Public Buildings.—" Where it is proposed to convert any building into a public building, such conversion shall be carried into effect, and the public building thereby formed, including the walls, roofs, floors, galleries and staircases thereof, shall be constructed in such manner as may be approved by the district surveyor ; or in the event of disagreement, may be determined by the Tribunal of Appeal ; and the provision of this Act shall apply to such conversion as though it were the construction of a public building."

Section 87. Staircases in Public Buildings.—" The following rules shall be observed with respect to new churches, chapels, meeting-houses, public halls, public lecture-rooms, public exhibition rooms, and public places of assembly, and with respect to additions or alterations to any existing church, chapel, meeting-house, public hall, public lecture-room, public exhibition room or public place of assembly being additions or alterations whereby increased accommodation is to be provided :

" (*a*) Every staircase for the use of the public shall be supported and enclosed by brick walls not less than 9 in. thick, and the treads of each flight of stairs shall be of uniform width.

" (*b*) No staircase, internal corridor, or passageway for the use of the public shall be less than 4 ft. 6 in. wide :

" Provided that, where not more than 200 persons are to be accommodated in such church, chapel, meeting-house, hall, lecture room, exhibition room or place of assembly, such staircase, internal corridor, or passageway may be of the width of 3 ft. 6 in.

" (*c*) Every staircase, corridor or passageway for the use of the public, which communicates with any portion of the building intended for the accommodation of a larger number of the public than 400, shall be increased in width by 6 in. for every additional 100 persons until a maximum width of 9 ft. is obtained.

"Provided that:

"(i) every such staircase being 6 or more feet wide shall be divided by a hand rail, and

"(ii) in lieu of a single staircase, corridor or passageway of the width prescribed by this section, it shall be lawful to substitute two staircases, corridors or passageways each being of a width at least equal to two-thirds of the width so prescribed for the single staircase, corridor, or passageway; but so that neither of such two substituted corridors, staircases or passageways shall be less than 3 ft. 6 in. wide.

"(*d*) In all cases where a portion of the public is to be accommodated over or at a higher level than others of the public, a separate means of exit of the width so prescribed for staircases, internal corridors or passageways, and communicating directly with a street or open space, shall be provided from each floor or level.

"(*e*) All doors and barriers shall be made to open outwards, and no outside locks or bolts shall be affixed thereto."

Section 88. Buildings under Railway Arches.—"Where a building erected after the thirty-first day of December eighteen hundred and ninety-four under or in or by inclosure of a railway arch, or abutting thereon, is constructed or adapted to be used for human habitation, this Act shall apply to the building, and to every work done to, in or on the same, in like manner and to the like extent, as far as may be, as if the building were built in any other position."

Part VII

Section 89. Buildings to which Rules of Act do not apply.—"(1) Before any builder erects an iron building or structure, or any other building or structure to which the general provisions of Part VI of this Act are inapplicable, or in the opinion of the Council inappropriate, having regard to the special purpose for which the building or structure is designed and intended to be used, he shall make an application to the Council, accompanied by a plan of the proposed building or structure, with such particulars as to the construction thereof as may be required by the Council.

"(2) The Council, if satisfied with such plan and particulars, shall signify their approval thereof in writing, and thereupon the building or structure may be constructed according to such plan and particulars; but the Council shall not authorise any building of the warehouse class to be erected of greater cubical extent than 250,000 cu. ft., except in accordance with the foregoing provisions of this Act.

"(3) The Council may, for the purpose of regulating the procedure in relation to such applications, issue such general rules as they think fit as to the time and manner of making applications and as to the plans to be presented, the expenses to be incurred and any other matter connected therewith.

"(4) All expenses incurred in and about obtaining the approval of the Council shall be paid by the builder to the superintending architect, or to such other person as the Council may appoint, and in default of payment may be recovered in a summary manner.

"(5) A copy of any plans and particulars approved by the Council shall be furnished to the district surveyor within whose district the building or structure to which such plans and particulars relate is situate; and it shall be his duty to ascertain that the same is built in accordance with the said plans and particulars."

Section 90. Certain Temporary Buildings.—" (1) Where an application is made to the Council under the last foregoing section with respect to a building or structure of a temporary character, the Council may, if they approve of the plan and particulars of the building or structure, limit the period during which it shall be allowed to remain, and may make their approval subject to such conditions as to the removal of the building or structure or otherwise as they think fit.

" (2) If at the expiration of that period the building or structure is not removed in accordance with those conditions, the Council may serve a notice on the occupier or owner of the building or structure requiring him to remove it within a reasonable time specified in the notice, and if the occupier or owner fails to remove the building or structure within the time named, the Council may, notwithstanding the imposition and recovery of any penalty, cause complaint thereof to be made before a Petty Sessional Court, who shall thereupon issue a summons requiring the occupier or owner to appear to answer the complaint.

" (3) If the complaint is proved to the satisfaction of the Court, the Court may make an order in writing authorising the Council to enter upon the land upon which the building or structure is situate and to remove or take down the building or structure, and to do whatever may be necessary for that purpose, and to remove the materials of which the building or structure is composed to a convenient place, and (unless the expenses of the Council are paid to them within 14 days after such removal) to sell the materials as they think proper.

Section 91. Wooden Structures.—" (1) No person shall set up in any place any wooden structure (unless it is exempt from the operation of this Part of this Act), except hoardings enclosing vacant land and not exceeding in any part 12 ft. in height, without having first obtained for that purpose a licence from the local authority, and the licence may contain such conditions with respect to the structure and the time for which it is to be permitted to continue in the said place as the local authority think expedient.

" Provided that in the case of any wooden structure of a movable or temporary character, erected by a builder for his use during the construction, alteration or repair of any building, a licence shall only be required if the structure is not taken down or removed immediately after such construction, alteration or repair.

" (2) Any proceedings for any default in obtaining or observing the conditions of any licence granted under this section may be taken by the local authority as respects their area.

" (3) This section shall not extend to the City or apply to any hoarding duly licensed by the local authority under any other Act."

Section 92. Power of Court to make Orders.—" Where any person has :

" (*a*) failed to comply with any notice duly served on him under this Part of this Act requiring him to remove a building or structure ; or

" (*b*) set up, erected, retained or adapted any building or structure to which this Part of this Act applies, without having obtained the approval or licence required by this Part of this Act ; or

" (*c*) contravened or failed to comply with any condition contained in such an approval or licence,

the Court before whom proceedings in respect of the contravention or non-compliance are taken by the Council, or the local authority, as the case may be, may, in addition to imposing any penalty, and without prejudice to any other powers conferred on them by this Act, make an order in writing directing the said person to demolish the building or structure or any part thereof, or to comply with the condition."

Section 93. Timber Stacks.—" This Part of this Act shall not apply to a pile, stack or store of timber not being a structure affixed or fastened to the ground."

Section 94. Structure of Railway Companies.—" This Part of this Act shall not apply to structures or erections set up or erected upon the premises of any railway company, and used for the purposes of or in connection with the traffic of such railway company.

Section 98. Projecting Shop.—" (1) Where any part of any building which is used or adapted to be used as a shop projects for a distance of 7 ft. or more beyond the main front of any building of which it forms part, and in which any persons are employed or sleep, the projecting portion of such shop shall be provided, by the owner, with a roof constructed of fire-resisting materials not less than 5 in. thick.

" (2) It shall be lawful to construct or place in or upon the roof of the portion of any shop so projecting beyond the main front of a building as aforesaid lantern-lights or ventilating cowls :

" Provided that :

" (i) no such lantern-light or ventilating cowl shall be constructed or placed so that any part thereof will be at a less distance than 6 ft. from the main front of the building from which the shop projects, or within such distance as is reasonable in the circumstances of the case from any other external or party wall ; and

" (ii) the sides of such lantern-light or ventilating cowl (except the side facing away from the main building) shall be carried up in fire-resisting materials for 2 ft. above the roof in or upon which it is constructed or placed ; and

" (iii) no part of any such lantern-light or ventilating cowl shall project above the roof in or upon which the same is constructed or placed to a greater extent than 5 ft.

" (3) The Council, or (in the event of an appeal) the Tribunal of Appeal, may, in any case where it is reasonable so to do, sanction, subject to such conditions (if any) as the Council or the Tribunal of Appeal may impose, the exemption of any building from all or any of the provisions of this section."

PART XIV

Section 154. Supervision by District Surveyor.—" Subject to the provisions of this Act every building or structure and every work done to in or upon any building or structure and all matters relating to the width and direction of streets, the general line of buildings in streets, the provision of open spaces about buildings and the height of buildings shall be subject to the supervision of the district surveyor appointed to the district in which the building, structure or street is situate."

Section 161. Notices to District Surveyor.—" (1) In the following cases and at the following times (that is to say) :

" (*a*) where a building or structure or work is about to be begun, then two clear days before it is begun ; and

" (*b*) where a building or structure or work is after the beginning thereof suspended for any period exceeding three months, then two clear days before it is resumed ; and

" (*c*) where during the progress of a building or structure or work the builder employed thereon is changed, then two clear days before a new builder enters upon the continuance thereof :

the builder shall serve on the district surveyor a notice (in this Act referred to as a 'building notice') respecting the building or structure or work. All works in progress at the same time to in or on the same building or structure may be included in one building notice.

"(2) Every building notice shall state the situation, area, height, number of storeys and intended use of the building or structure and the number of buildings or structures, if more than one, and the particulars of the proposed work and the name and address of the person giving the notice and those of the owner then in possession of and the occupier of the building or structure or of its site or intended site."

PART XVII

Section 221. Obstruction of Streets.—"(1) No person not being lawfully authorised, shall erect or place, or cause to be erected or placed, any post, rail, fence, bar, obstruction or encroachment whatsoever in upon over or under any street, and no person, not being lawfully authorised, shall alter or interfere with any street in such a manner as to impede or hinder the traffic for which the street was formed or laid out from passing over the street.

"(2) The local authority may, at the expiration of two days, after giving notice in writing to any such person requiring him to demolish or remove the post, rail, fence, bar, obstruction or encroachment, or to reinstate or restore the street to its former condition (as the case may be), demolish or remove the post, rail, fence, bar, obstruction or encroachment and reinstate or restore the street to its former condition, and recover the expenses thereof from such person in a summary manner.

"(3) This section shall not extend to the City."

Section 223. Buildings exempt from Parts VI and VII of the Act.—"The following buildings, structures and works shall be exempt from the operation of Parts VI and VII of the Act.

"(1) Bridges, piers, jetties, embankment walls, retaining walls and wharf or quay walls.

"(2) The Mansion House, Guildhall and Royal Exchange of the City.

"(3) The offices and buildings of the Bank of England within the City.

"(4) All buildings erected by or with the sanction of the Commissioners for the Exhibition of eighteen hundred and fifty-one on any lands belonging to them, and purchased in pursuance of any power vested in them by charter or Act of Parliament, except streets or blocks of buildings erected by them, or with their sanction, as private dwelling-houses.

"(5) The Sessions House of the Central Criminal Court, and all other sessions houses or other public buildings belonging to, or occupied for public purposes by, the justices of the peace of the counties of Middlesex and London, and of the City of London respectively, or by the County Councils of London and Middlesex respectively.

"(6) The erections and buildings authorised by an Act passed in the ninth year of the reign of His late Majesty King George the Fourth for the purposes of a market in Covent Garden.

"(7) The buildings of the Metropolitan Cattle Market and any building within the market premises inhabited or adapted to be inhabited by any official or servant of the Corporation for the purposes of that market.

"(8) (*a*) Subject as hereinafter provided, any building or part of a building belonging to the Port of London Authority in connection with their Surrey Canal undertaking, or to a canal company and used exclusively for the purposes of canal works under any Act of Parliament;

" (*b*) any building or structure situate upon a railway or within the railway or station premises, and used for the purposes of, or in connection with the traffic of, a railway company ;

" (*c*) any building or part of a building belonging to a gas company and used exclusively for gasworks ;

" (*d*) any building or part of a building belonging to the Port of London Authority in their capacity as successors of the Conservators of the River Thames, and used by them as a workshop or store ;

" (*e*) the foundations and walls of buildings belonging to a railway company situate over any station or works of a railway company, or immediately adjoining any railway or works of a railway company, and upon land acquired under the powers of an Act of Parliament ;

" (*f*) any building within the station premises of any railway company inhabited, or adapted to be inhabited in whole or in part by any official or servant of the railway company ;

" Provided that, save as otherwise expressly provided therein, nothing in this Paragraph shall exempt any buildings used for the purpose of human habitation so far as they are so used.

" (9) Any building or structure or part of a building or structure belonging to the Port of London Authority, or to a dock company constituted by Act of Parliament, and situate within the dock premises of the authority or company.

" (10) Any building, not exceeding in area 30 sq. ft., and not exceeding in height 5 ft. in any part, measured from the level of the ground to the underside of the eaves or roof plate, and distant at least 5 ft. from any other building, and from any street, and not having therein any stove, flue, fireplace, hot-air pipe, hot-water pipe, or other apparatus for warming or ventilating the first-mentioned building if no portion thereof extends beyond the General Line of Buildings in any street.

" (11) All buildings and structures (not exceeding in height 30 ft. as measured from the footings of the walls and not exceeding in extent 125,000 cu. ft. and not being public buildings) wholly in one occupation and distant at least 8 ft. from the nearest street or way and at least 30 ft. from the nearest buildings and from the land of any adjoining owner.

" Provided that a detached dwelling-house shall not be excluded from this exemption solely by reason of its being within 30 ft. of another detached building constructed as stables or offices to be used in connection with the dwelling-house.

" (12) All buildings not exceeding in extent 250,000 cu. ft., and not being public buildings, and distant at least 30 ft. from the nearest street or way, and at least 60 ft. from the nearest building and from the land of an adjoining owner.

" Provided that a detached dwelling-house shall not be excluded from this exemption solely by reason of its being within 60 ft. of another detached building constructed as stables or offices to be used in connection with the dwelling-house.

" (13) All party fence walls not exceeding in height 7 ft., measured from the top of the footings of the wall.

" (14) Greenhouses not being attached to other buildings.

" (15) The necessary woodwork of the sashes, doors and frames of greenhouses attached to other buildings.

" (16) Cases of metal and glass, used solely for holding plants, fastened to the woodwork of the sill and lower sash of a window, if no portion projects over the public way or more than 12 in. beyond the external face of the wall of the building.

"(17) Openings made in walls or flues for the purpose of inserting therein ventilating valves of a superficial extent not greater than 40 sq. in., if the valves are not nearer than 12 in. to any timber or other combustible material.

"If any addition is made to any building or structure specified in paragraph (10), paragraph (11) or paragraph (12) of this section, whereby any increase is caused in the area height or extent of the building or structure beyond the area, height or extent respectively mentioned in those paragraphs, the Council may give notice to the owner or occupier of the building or structure, either to remove the addition or to make the building so increased in height or extent conform to all or any of the provisions of this Act and with any by-laws under this Act relating to the construction of buildings; and upon his failing to do so, within 14 days from the service upon him of such a notice the Council may remove the addition to the building or structure, and may recover the expenses of the removal from the owner or occupier in a summary manner."

Section 226. Crown Exemption.—"(1) There shall be exempted from so much of this Act as relates to buildings and structures:

"every building, structure, or work vested in and in the occupation of His Majesty either beneficially, or as part of the hereditary revenues of the Crown, or in trust for the public service or for public services; and

"every building, structure, or work vested in and in the occupation of any department of His Majesty's Government, or of the Metropolitan Police, or of the trustees of the British Museum for public purposes or for the public service; and

"every building, structure or work vested in and occupied for the service of the Duke of Cornwall for the time being."

Section 227.—"Where a local authority, or a company, has statutory powers for the supply of electricity in any district in London, the buildings of that local authority or company, used as a generating station or for works, shall be deemed to be special buildings to which the general provisions of Parts V, VI and VII of this Act and the First and Second Schedules thereof do not apply; and plans thereof shall be submitted to the Council for their approval; and the Council shall have power to authorise the buildings to be erected of greater dimensions than 250,000 cu. ft., and in other respects to exempt the buildings from any of the provisions of this Act if they think fit."

Section 228.—Exempted Lands, Buildings, etc., of Inns of Court.—"The lands, buildings and property of:—

"(1) the Honourable Society of the Inner Temple;

"(2) the Honourable Society of the Middle Temple;

"(3) the Honourable Society of Lincoln's Inn;

"(4) the Honourable Society of Gray's Inn, shall be exempt from the provisions of this Act.

"Provided that any such land, building or property which abuts upon any public street, public place or public way shall be subject to the provisions of Part III of this Act."

Section 229. Duration of Exemptions.—"Any building, structure or work, in any respect exempt from the operation of the provisions of this Act, or in any manner privileged in respect of any of the said provisions, shall remain so exempt or privileged, so long only as it is used for the purpose, or retains the character by reason whereof it is so exempt or privileged."

Section 230. Alterations to Conform to the Act.—"It shall not be lawful (except with the consent of the Council) to make any alteration of any building in such manner that, when so altered, it will, by reason of the alteration, not comply with the provisions of this Act."

Section 231. Partial Removal of Party Walls, etc.—" Unless in any case the Council otherwise allow where a party wall or external wall which does not comply with this Act has been taken down, burnt or destroyed to the extent of one-half thereof (measured in superficial feet) every remaining portion of the old wall which does not comply with this Act shall either be made to comply therewith or be taken down before the rebuilding thereof."

Section 232. Alterations and Additions to Conform to the Act.—" Every addition to or alteration of a building and any other work made or done for any purpose in to or upon a building (except that of necessary repair not affecting the construction of any external or party wall) shall be subject to the provisions of this Act and of by-laws thereunder relating to buildings."

Section 233. Building erected before Operation of the Act.—" A building, structure or work erected or constructed before the commencement of this Act to which no objection could have been taken under any law then in force shall (subject to the provisions of this Act as to the alteration of buildings) be deemed to be erected or constructed in compliance with the provisions of this Act."

Section 234. Conversion of Buildings.—" Unless in any case the Council otherwise allow, no person shall :

" (1) convert into or use as a dwelling-house any building or part of a building not originally constructed for human habitation ;

" (2) convert into one dwelling-house two or more dwelling-houses constructed originally as separate dwelling-houses ;

" (3) convert into or use as two or more dwelling-houses any building constructed originally as one dwelling-house ;

" (4) convert a building which when originally erected was legally exempt from the operation of any building enactments or bye-laws in force within London into a building which had it been originally erected in its converted form would have been within the operation of those enactments or bye-laws ;

" (5) reconvert into or use as a dwelling-house any building which has been discontinued as or appropriated for any purpose other than a dwelling-house ;

" (6) convert into or use as a dwelling-room or part of a dwelling-room any room or part of a room used as a shop ; or

" (7) convert a dwelling-house or any part of a dwelling-house into a shop ; in such manner that the building or part of a building will after being so converted not comply with the provisions of this Act relating to the class of buildings to which the building or part will after being so converted belong."

First Schedule to the Act. (*Fire-resisting Materials.*)—The following materials shall, for the purposes of this Act, be deemed to be fire-resisting materials :

" (I) For general purposes :

" (1) Brickwork constructed of good bricks, well burnt, hard and sound, properly bonded and solidly put together :

" (*a*) with good mortar compounded of good lime and sharp, clean sand, hard, clean broken brick, broken flint, grit or slag ; or

" (*b*) with good cement ; or

" (*c*) with cement mixed with sharp, clean sand, hard, clean broken brick, broken flint, grit or slag ;

" (2) Granite and other stone suitable for building purposes by reason of its solidity and durability.

" (3) Iron, steel and copper.

" (4) Slate, tiles, brick and terra-cotta, when used for coverings or corbels.

"(5) Flagstones when used for floors over arches, if the flagstones are not exposed on the underside and not supported at the ends only.

"(6) Concrete composed of broken brick, tile, stone chippings, ballast, pumice, or coke breeze and lime, cement, or calcined gypsum.

"(7) Any combination of concrete and steel or iron.

"(II) For special purposes:

"(1) In the case of doors and shutters, and their frames, oak, teak, jarrah, karri, or other hard timber not less than $1\frac{3}{4}$ in. finished thickness, the frames being bedded solid to the walls or partitions.

"(2) In the case of staircases and landings, oak, teak, jarrah, karri or other hard timber, the treads, risers, strings and bearers being not less than $1\frac{3}{4}$ in. finished thickness, and the ceilings and soffits (if any) being of plaster or cement.

"(3) Oak, teak, jarrah, karri or other hard timber when used for beams or posts or in combination with iron, the timber and the iron (if any) being protected by plastering or other incombustible or non-conducting external coating not less than 2 in. in thickness, or in the case of timber not less than 1 in. in thickness on iron lathing.

"(4) (*a*) In the case of floors and roofs: brick, tile, terra-cotta, or concrete composed as described in sub-paragraph (6) of paragraph (I) of this schedule not less than 5 in. thick, in combination with iron or steel.

"(*b*) In the case of floors and of the roofs of projecting shops: pugging of concrete, composed as described in the said sub-paragraph (6) of paragraph (I), not less than 5 in. thick, between wood joists, if a fillet 1 in. square is secured to the sides of the joists, and placed so as to be in a central position in the depth of the concrete, or concrete blocks not less than 5 in. thick, laid between wood joists, or fire-resisting bearers secured to the sides of joists.

"(5) In the case of verandahs, balustrades, outside landings, the treads, strings, and risers of outside stairs, outside steps, porticoes, and porches, oak, teak, jarrah, karri, or other hard timber, not less than $1\frac{3}{4}$ in. finished thickness.

"(6) In the case of internal partitions enclosing staircases and passages, terra-cotta, brickwork, concrete, or other incombustible material not less than 3 in. thick.

"(7) In the case of glazing for windows, doors, and borrowed lights, lantern or skylights, glass not less than $\frac{1}{4}$ in. in thickness, in direct combination with metal the melting point of which is not lower than 1,800° F. in squares not exceeding 16 sq. in., and in panels not exceeding 2 ft. across either way, the panels to be secured with fire-resisting materials in fire-resisting frames of hard wood not less than $1\frac{3}{4}$ in. finished thickness or of iron.

"(III) Any other material from time to time approved by the Council as fire-resisting."

Regulations with regard to Applications Under Part III of the London County Council (General Powers) Act, 1908.

Additional Cubical Extent. (See also Section 82 of the Act.)—"(1) No application for consent to the provision of additional cubical extent shall be entertained in respect of a building which is or is to be in more than one occupation.

"(2) All consents shall be subject to such fire-extinguishing appliances being provided as may in the opinion of the Council be necessary, and to direct telephonic communication being established with a London Fire Brigade station, if required by the Council.

"(3) No heating, lighting, electrical or ventilating arrangements shall be installed unless and until a detailed specification of the proposals has been submitted to and approved by the Council, and except in accordance with such approved specification.

"(4) Arrangements for lessening the danger from fire in premises in respect of which consent is given shall be properly maintained. Premises will be liable to inspection by the chief officer of the London Fire Brigade to ensure such proper maintenance.

"(5) In dealing with applications in respect of buildings of the various classes set out hereunder, the following principles shall be considered in conjunction therewith. In any case they may be varied or other conditions imposed as the Council thinks fit. These regulations are to be considered in conjunction with the provisions of the Act, and in no way as a modification of those provisions.

"(A) As regards buildings not more than one storey in height in one occupation, and used exclusively for the manufacture of boilers or machinery in metal, and for all kinds of metal work in connection with metal-framed structures, such cubical extent may be allowed as may in the opinion of the Council be reasonable, having regard to the circumstances of the case, provided that:

"(i) The building be more than two miles distant from St. Paul's Cathedral if the operations of smelting or melting be carried on therein.

"(ii) The building, including fittings and fixtures, but excluding doors, windows, skylights, lantern lights and their frames, be constructed throughout of incombustible materials.

"(iii) Any portion of the building used for the manufacture or storage of oils or varnishes, moulds, models, frames, patterns or other substances or articles of a combustible nature, be properly and efficiently separated from and not included in the portion of the building in respect of which additional cubical extent may be allowed and do not exceed the limits prescribed in the Act.

"(B) As regards buildings of more than one storey in height in one occupation and used exclusively for the use, manufacture or storage of non-inflammable or fire-resisting or incombustible materials, such cubical extent may be allowed as may in the opinion of the Council be reasonable, having regard to the circumstances of the case, provided that:

"(i) The height of the building do not, except with the consent of the Council, exceed 80 ft., measured from the pavement level to the underside of the ceiling of the topmost storey.

"(ii) The building be not less than 40 ft. from any other building, except where it is entirely separated therefrom by an imperforated wall of the thickness required by the London Building Act, 1894, and of the full height of the higher building throughout.

"(iii) The building be constructed throughout of incombustible material, and the doors, windows, skylight, lantern lights and their frames be constructed of fire-resisting materials.

"(iv) All floors be imperforate except as hereinafter provided, and be furnished with scuppers to carry off water poured on during a fire.

"(v) All constructional ironwork below the level of the surface of the floor of the topmost storey be protected by not less than 2 in. of incombustible material.

" (vi) All openings in the external walls or roofs excepting staircase windows and shop windows on the ground floor, when the show spaces are separated from the main building by fire-resisting enclosures, be fitted with fire-resisting frames filled in with fire-resisting doors, shutters or glazing, a sufficient portion of the filling of the openings above the ground storey being made so as to be easily opened from the outside in order to facilitate the ingress of firemen.

" (vii) Vertical shafts for staircases, staircase enclosures, lifts, hoists, shafts or trunks for pipe or wires, and other leads and horizontal ducts, be properly and efficiently separated by walls and fire-resisting doors from the divisions in respect of which additional cubical extent is required, and from any other division communicating therewith, and be so arranged that the access to such shafts or ducts from any floor shall be provided with a secondary incombustible enclosure with self-closing, fire-resisting doors.

" (viii) If any vertical shafts be roofed, the roofs be constructed of light frames, filled in with thin glass and protected on the outside by strong wire guards, and the shafts be carried up not less than 3 ft. above any adjoining roofs.

" In the case of a shaft not carried up to and through the roof, it be sealed over at the top with solid incombustible material not less than 6 in. thick.

" (ix) All horizontal trunks, ducts, etc., be enclosed wtih solid incombustible material not less than 3 in. thick.

" (x) Any portion of the building used for packing or for the storage of packages be properly and efficiently separated from the portion of the building in respect of which additional cubical extent may be allowed.

" (xi) When the building is more than two storeys above the ground storey, and the total extent between party walls exceeds 1,000,000 cu. ft., there be provided at least one unenclosed staircase constructed of brick and concrete, affording access to all floors and the roof through the outer air, without any internal communication with the building.

" (xii) If the building exceed 250,000 cu. ft., one-sixth at least of the total length of the boundaries of the site do abut upon a thoroughfare or thoroughfares not less than 40 ft. wide.

" If the building exceed 1,000,000 cu. ft., one-fourth at least of the total length of the boundaries of the site do abut upon a thoroughfare or thoroughfares not less than 40 ft. wide.

" If the building exceed 2,000,000 cu. ft., one-half at least of the total length of the boundaries of the site do abut upon a thoroughfare or thoroughfares not less than 40 ft. wide.

" If the building exceed 3,000,000 cu. ft., three-fourths at least of the total length of the boundaries of the site do abut upon a thoroughfare or thoroughfares not less than 40 ft. wide.

" If the building exceed 4,000,000 cu. ft., the site be an island site.

" (C) As regards buildings of more than one storey in height in one occupation and used for the sale and storage or manufacture, but not for storage purposes only, of goods of a non-fire-resisting nature, or used for the purposes of a trade involving the use of materials of a non-fire-resisting nature, additional cubical extent may be allowed, subject to the provisos (i) to (xii) set out in paragraph (B), and provided also that:

"(xiii) The basements be entirely cut off from the ground floor and upper floors of the building and approached only by a separate means of access from the outside of the building.

"(xiv) Any lifts from the basements be adjacent to an external wall or do have no internal communication with the storeys above.

"(xv) No divisions or cell on any floor formed by the vertical and horizontal separations do exceed 500,000 cu. ft., and the floor area of such division or cell do not exceed 40,000 sq. ft.

"(xvi) Every floor be of the thickness required by the Council, such thickness not being less than 6 in.

"(D) As regards buildings of not more than three storeys in height in one occupation and used as motor garages or car-sheds, or for other similar purposes, such cubical extent may be allowed as may in the opinion of the Council be reasonable, having regard to the circumstances of the case, provided that:

"(i) The building be constructed throughout of incombustible materials, and be completely separated by imperforate walls from adjoining properties.

"(ii) Any store for petrol be completely separated from the main building.

"(iii) Any portion of the main building used for the manufacture or storage of varnishes or oils, or moulds, models, frames, patterns or other substances or articles of a combustible nature be properly and efficiently separated from and not included in the portion for which additional cubical extent may be allowed and do not exceed the limits prescribed in the Act."

Openings in Party Walls. (See also S. 60 of the Act and p. 235.)—(6) Applications for consent to openings larger than those allowed by the Act in party walls separating divisions of cubical extent will not be entertained unless the buildings affected be constructed throughout of incombustible materials, and unless:

"(i) The openings be fitted with double doors or shutters in accordance with the provisions of the Act at a distance apart of not less than one-fourth of the full width of the proposed opening.

"(ii) The width of all such openings taken together on each floor do not exceed one-half of the length of the party wall on each floor in which they occur.

Hollow Floors and Hollow Roofs.—The Council on 23 June, 1925, passed the following resolutions:

"That the Council in pursuance of the powers vested in it by the First Schedule (Part III) to the London Building Acts (Amendment) Act, 1905, do approve as fire-resisting, in the case of hollow floors and hollow roofs, asbestos, burnt clay, cement mortar, or cement concrete and steel, subject to the following conditions:

"(i) that all materials shall be to the satisfaction of the district surveyor and the whole of the work shall be executed to his satisfaction;

"(ii) that if any blocks are of clay, they shall be thoroughly burnt, free from lime and cracks and other defects;

"(iii) that all concrete which is less than 2 in. thick at any part, and all concrete for hollow brick floors, shall be fine concrete, mixed in the proportion of at least one volume of Portland cement to three volumes of aggregate composed of clean, silicious sand and broken flint of all such sizes as will pass through a mesh $\frac{3}{8}$ in. square measured in the clear"; (i.e. concrete designated I, see p. 35.)

"(iv) that when such aggregate is not 'fully graded,' i.e. when such aggregate is not so varied in size that all voids can be filled with the cement paste, then the proportions shall be not less than one volume of Portland cement to two volumes of aggregate;

"(v) that, in the case of hollow block floors, the ends of the blocks shall be solidly jointed with cement mortar in the proportions of one volume of Portland cement to two volumes of clean sharp sand;

"(vi) that the concrete above the void shall be mixed in the same proportions as in the solid portions of the floor;

"(vii) that apart from any requirements in respect of imposed loads and working stresses: (1) the least diameter of the main tensile reinforcement shall be not less than $\frac{1}{4}$ in., (2) the least diameter of shear reinforcement (if any) shall be not less than $\frac{3}{16}$ in., and (3) both ends of all reinforcement shall be properly hooked to the satisfaction of the district surveyor;

"(viii) that when burnt clay tiles are not used as a part of the covering of the reinforcement, then the thickness of fine concrete (exclusive of any other material) covering the main tensile reinforcement shall be not less than $\frac{3}{4}$ in. thick and not less than the diameter of such main tensile reinforcement (see Fig. 146, p. 128);

"(ix) that when burnt clay tiles are used as a part of the covering of the reinforcement, then the thickness of fine concrete covering the main tensile reinforcement shall be not less than $\frac{1}{2}$ in. and not less than the diameter of such main tensile reinforcement, and the thickness of the tile shall be not less than $\frac{1}{2}$ in. (see Fig. 147, p. 128);

"(x) that the sides and undersides of steel beams and girders shall be protected from the action of fire by fine concrete at least 2 in. thick, rodded and rammed, or $1\frac{1}{2}$ in. thick trowelled round suitable mesh reinforcement, and that in all cases the concrete shall be bedded solidly against the steel and there shall be no intervening cavity;

"(xi) that fillets, strips and blocks of wood or other combustible materials shall not be embedded in the thickness of the fine concrete necessary: (1) to transmit stresses, (2) to provide protection to any steelwork or reinforcement, or (3) to comply with any of these conditions;

"(xii) that when the material above the void is effectively arched or haunched the thickness of the material at the crown shall be not less than 2 in.;

"(xiii) that when the material above the void is not effectively arched or haunched the thickness of the material above the void shall be not less than:

"(1) 2·0 in. in cases in which the width of the void is not more than 4 in.,

"(2) 2·5 in. in cases in which the width of the void is not more than 5 in.,

"(3) 3·0 in. in cases in which the width of the void is 6 in. and over;

"(xiv) that the thickness of the soffit of the voids (exclusive of any plaster) shall be not less than $\frac{3}{4}$ in.

"(xv) that the thickness of the material above the void added to the thickness of the soffit (exclusive of any plaster) shall be not less than 3 in.;

"(xvi) that the dimensions specified in the foregoing conditions shall refer only to questions of fire-resistance and resistance to impact during fires, apart from any requirements in respect of imposed loads and working stresses under normal conditions of use.

Provided that this approval shall not in any way derogate from any of the powers of the Council, and shall not in any way affect the requirements of: (1) the London County Council (General Powers) Act, 1908, with respect to cubical extent of buildings; (2) the London County Council (General Powers) Act, 1909, with respect to the enclosure or encasing of pillars or girders with brickwork, terra-cotta, stone, tiles or other incombustible materials; and (3) the regulations made under the provisions of section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete."

The Council on 2 March, 1926, passed the following resolution:

"That the Council in pursuance of the powers vested in it by the First Schedule (Part III) to the London Building Acts (Amendment) Act, 1905, do approve as fire-resisting hollow floors and hollow roofs constructed of steel filler joists in combination with hollow bricks and concrete, subject to the following conditions:

"(i) that all materials shall be to the satisfaction of the district surveyor, and the whole of the work shall be executed to his satisfaction;

"(ii) that if any blocks are of clay they shall be thoroughly burnt, and be free from lime, cracks and other defects;

"(iii) that the concrete used for filling in, either between around or above the hollow bricks, shall be mixed in proportions of at least one volume of Portland cement, two volumes of various sizes of clean siliceous sand and four volumes of hard broken brick, flint or stone of all the various sizes which will pass through a mesh of $\frac{3}{4}$ in. measured in the clear"; (i.e. concrete designated III, see p. 35.)

"(iv) that there shall be at least 1 in. of fire-resisting material (inclusive of plaster) below the lower flange of the steel filler joist and that all joints between hollow bricks shall be pointed in cement mortar;

"(v) that there shall be at least 1 in. thickness of concrete or other fire-resisting material covering the upper flange of the steel filler joists;

"(vi) that the steel joists shall receive one coat of rust-resistant paint;

"(vii) that the sides and undersides of steel beams and girders (other than filler joists embedded in the thickness of the floor) shall be protected from the action of fire by fine tamped concrete at least 2 in. thick, or burnt clay tiles and cement plaster of an aggregate thickness of 2 in. or fine concrete $1\frac{1}{2}$ in. thick trowelled round suitable mesh reinforcement, and that in all cases the concrete shall be bedded solidly against the steel and there shall be no intervening cavity;

"(viii) that fillets, strips and blocks of wood or other combustible materials shall not be embedded in the thickness of the fine concrete necessary to comply with any of these conditions;

"(ix) that the thickness of the material above the void added to the thickness of the soffit (exclusive of any plaster) shall be not less than 3 in.;

"(x) that the dimensions specified in the foregoing conditions shall refer only to questions of fire-resistance and resistance to impact during fires, apart from any requirements in respect of imposed loads and working stresses under normal conditions of use.

"Provided that this approval shall not in any way derogate from any of the powers of the Council, and shall not in any way affect the requirements of: (1) the London County Council (General Powers) Act, 1908, with respect to cubical extent of buildings; (2) the London County Council (General Powers) Act, 1909, with respect to the enclosure or encasing of pillars or girders with brickwork, terra-cotta, stone, tiles or other incombustible materials; and (3) the regulations made under the provisions of Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete."

Wired Glass.—The Council on 13 July, 1926, passed the following resolution with regard to the use of wired glass for the purpose of fire-resisting glazing:

"That the Council, in pursuance of the powers vested in it by the First Schedule (Part III) of the London Building Acts (Amendment) Act, 1905, do approve as fire-resisting for the glazing of windows, doors, borrowed lights, lanterns and skylights the under-mentioned materials:

"(1) Glass not less than $\frac{1}{4}$ in. thick reinforced with wire at least 0·55 mm. diameter laid to a square mesh measuring $\frac{1}{2}$ in. from centre to centre of wire and electrically welded at the inter-sections; and

"(2) Glass not less than $\frac{1}{4}$ in. thick reinforced with wire at least 0·55 mm. diameter laid to a hexagonal mesh measuring 1 in. across the flat sides;

fixed in panels not exceeding 2 ft. either way, the panels to be secured with fire-resisting materials in fire-resisting frames of hardwood not less than $1\frac{3}{4}$ in. finished thickness, or of iron or bronze or other material having a melting point of not less than 1800° F., subject to the following condition:

"That the district surveyor shall be furnished with a certificate from the makers of the glass stating that the glass is of such quality that a plate $\frac{1}{4}$ in. thick and measuring 6 in. by 6 in. will resist the passage of smoke or flame for a period of one hour when exposed to a temperature of 1500° F. on one side and to the atmosphere at the other and will not fall to pieces on the application of cold water at the end of such period."

Impregnated Wood.—"That the Council in pursuance of the powers vested in it by the First Schedule (Part III) to the London Building Acts (Amendment) Act, 1905, do approve as fire-resisting for the like purposes for which hard timber may be used, the following materials, namely, suitable soft woods which have been impregnated throughout with ammonium phosphate, subject to the following conditions:

"(i) that the district surveyor shall be furnished with a certificate from the firm carrying out the impregnation stating (*a*) that the whole of the wood has been thoroughly impregnated, and (*b*) that a door made of soft wood similarly impregnated and measuring 6 ft. by 3 ft. by 2 in. thick will resist the passage of smoke or flame when exposed to a temperature of 1500° F. for a period of one hour, and that, when so exposed, the increase of temperature registered by an unshielded thermometer 12 in. away from the outer surface of the door will not exceed 10° F.

"(ii) that the persons who have impregnated the wood shall at their own expense make such tests as the district surveyor may consider necessary;

"(iii) that the thickness of the impregnated wood shall be at least equal to that required for hard timber by the First Schedule to the London Building Acts (Amendment) Act, 1905, and that, where this thickness is built up, the pieces of impregnated wood shall be not less than $1\frac{3}{4}$ in. finished thickness;

"(iv) that all panels and boarding shall be properly tongued and grooved;

"(v) that the woodwork shall receive at least two coats of paint, varnish or enamel; and

"(vi) that doors, and other structures of impregnated wood, in respect of which a certificate is furnished as required by the foregoing condition (i) be distinguished by a mark or device cold die-stamped on the outside edge of the hanging style doors and in some easily found inconspicuous position on other structures, and that such mark or device be not used on wood which does not comply with the conditions specified in the above-mentioned certificate."

Fire Protection.—" 132. Buildings to which Section 17 of the London County Council (General Powers) Act, 1908, applies and in which no portion above the ground floor storey is used for trade or manufacture or for warehouse purposes, may be erected to a height not exceeding 100 ft. measured from the pavement level to the underside of the ceiling of the topmost storey, provided that, if such a building be erected to a greater height than 80 ft. measured on the foregoing basis, the following conditions, in addition to any other conditions which the Building Acts Committee may consider necessary in any particular case, shall be complied with. (i) That fire appliances, to include an approved automatic sprinkler installation, shall be provided throughout the whole of the building, including that portion used for other than trade, manufacture or warehouse purposes; and (ii) that the portion of the building above the ground floor storey shall be entirely cut off from the portion used for trade manufacture or warehouse purposes.

Openings in Party Walls.*—" 133. Consents to the formation of openings larger than those allowed by the London County Council (General Powers) Act, 1908, in party walls separating divisions of cubical extent shall be subject to conditions to the following effect and to any other conditions which may be imposed :

" (i) The building shall be constructed throughout of incombustible materials.

" (ii) The openings shall be fitted with double doors or shutters in accordance with the provisions of the Act at a distance apart of not less than one-fourth of the full width of the proposed openings. Details of the doors or shutters must be submitted to the Council for approval.

" (iii) The width of all such openings taken together on each floor shall not exceed one-half of the length of the party wall on each floor in which they occur.

" 134. An inspection shall be made annually of the electrical and heating installations at garages and of the fire appliances at all premises in respect of which the Council has consented to additional cubical extent under Section 17 of the London County Council (General Powers) Act, 1908."

* See also S. 60 of the Act, p. 214 and p. 231.

PART III.

CHAPTER 15

MONOLITHY

(*Moments of Inertia ; Stiffnesses of Members ; Design of Rigid Frames ; and Continuous Beams.*)

In most of the modern buildings the single parts of the structural frames so work in conjunction, that the loading on one member influences adjacent and even remote members.

Whilst the training of most engineers and architects enables them to find the stresses in, and the sizes of, single members, the exact calculation of the effect of one member on the other requires a degree of knowledge, which, at present is not common.

What is more is that a great amount of time is involved in making an exact calculation, and frequently this expenditure of time is not considered to be justified. From the point of view of structural economy as well as from the technical aspect this attitude is to be regretted.

If a frame is designed for a stress distribution which has little relationship to the actual, the safety factor instead of being constant, is very small at certain points and excessively great at others. In consequence cracks develop in buildings and walls become porous whilst elsewhere in the structure material is being wasted.

The aim of this chapter is to explain the monolithic action of framework and to enable the designer to estimate stresses by methods which although sometimes based on approximations, are exact enough, without involving higher mathematics, and without involving an unreasonable amount of time.

To this end many tables have been prepared setting out standard results which may be applied directly to particular cases.

The methods of design which are set out in this chapter are based on exact analysis only up to a point. Beyond that, certain approximations are recommended which give final results sufficiently accurate to be taken as exact for all practical purposes. Approximations are, in fact, sufficient, for there is never any guarantee that in a frame or continuous beam the actual behaviour will correspond entirely with the assumed behaviour. The manner in which a structural system will act can be influenced very greatly by factors that are not taken into consideration in an exact theoretical analysis. For instance, a steel frame may be calculated exactly by a mathematical process and then during construction it may be encased for fire protection. Such encasing may alter the behaviour of the frame considerably.

A method has been developed by the authors especially for the purpose of making it possible to solve statically indeterminate systems without resorting to complex and lengthy calculations. It is hoped that this method will induce designers to calculate structures of this kind in a more thorough way than is usual in order that a uniform coefficient of safety may obtain throughout—a condition which is absolutely essential to the use of the higher stresses now permitted.

The methods given make due allowance for the influence of one member on adjacent members and they carry the investigation only as far as in the authors' opinion practical considerations justify. To assist in the investigation tables have been included giving coefficients for many of the factors which must be used.

The methods apply to any constructions in which rigid joints or continuity occur—as, for instance, in certain riveted steelwork, sometimes in timber structures, and almost invariably in welded steelwork and reinforced concrete. However, to avoid repeating all these cases throughout the chapter, the explanation is given on the assumption that reinforced concrete is being used. The reader must, therefore, bear in mind that the general principles apply also to the other materials.

It has been assumed that a joint is rigid when it is capable of transmitting the whole of the bending moments and shear stresses, due to its stiffness, from one member to another without any permitted stresses being exceeded.

Before designing any reinforced concrete building, for example one must consider the fact that, owing to the monolithic nature of the material, the single parts of the structure cannot be regarded as acting independently of one another, unless special joints are provided. If this is not done, it must be taken that the angle between any two adjoining members remains constant whatever the loads and the deformations of the individual members may be.

In most cases the main problems may be simplified, so far as floor-slabs are concerned, by assuming that the slabs have very little effect on beams and columns, apart from restraining them laterally, that is to say, in position on plan ; but columns and beams must generally be considered as parts of one structural unit.

Only by some arrangement of hinges is it possible to obtain beams and columns which can be regarded as single units ; but the arrangement of such hinges is generally expensive and usually weakens the building considerably. It is preferable, therefore, in the majority of cases (there are, of course, exceptions) to erect a monolithic structure in spite of the fact that the design is a little more involved. In this chapter, an attempt will be made to simplify the design as far as possible, and to give the necessary formulæ which may be used for this purpose.

Strictly speaking, every frame is statically indeterminate at certain points. It may, however, be calculated correctly, according to the theories for such structures. These theories are based on the influences of any load at any point of such a frame on the bending moments and normal forces at any other point, and whilst such influences are generally small and often negligible, it is they which make the exact calculations very involved.

It should be appreciated that the effect of the rigid connection between two members is always the same, whether there is continuity of a beam or whether a column is rigidly connected to the beam. In every case a moment is created at the point of rigidity, which, for example in the beam, tends to reduce the span bending moment from the value it would have if the member were considered free at the ends. It is, therefore, clear that continuous beams obey somewhat similar laws as frames and, therefore, they too are treated in this chapter.

In the Memorandum on the computation of stresses in buildings and chimney-shafts of the London County Council, 1938, formulæ have been given for the calculation of bending moments in exterior columns. These will be accepted generally, and are amplified in order to be taken as the basis for the following investigations.

It should be appreciated first that the rigidity of the structure at any point or connection can work in both ways. That is to say, for example, that by the stiffness of a joint a beam may be restrained and influenced by a column ; but the column, on the other hand, will be affected by the conditions of loading, etc., on the beam.

The amount to which one member influences or is influenced by the adjoining members varies with their relative stiffnesses, and these stiffnesses depend on such properties of the members as cross-sections and spans, etc.

For example, if a long and shallow beam is supported on a short and wide column which is not able to follow the deformation of the beam, the beam will practically be restrained from rotating at the end ; but, conversely, a relatively shallow beam will be unable to assist the column to any appreciable amount in resisting any deformation which may occur due to bending or buckling. On the other hand, a very deep, short beam will be afforded very little restraint by slender columns, and in fact such a stiff beam will restrain the ends of the columns from any rotational movement.

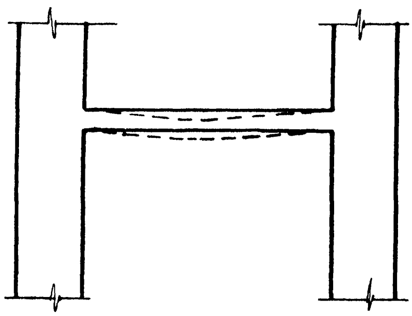


FIG. 279

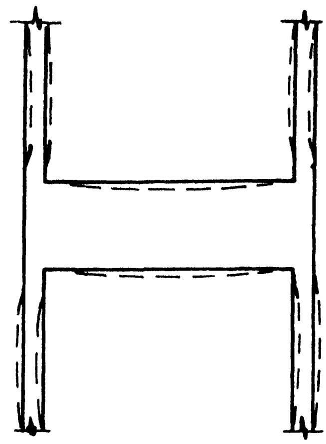


FIG. 280

In Figs. 279 and 280 the dotted lines indicate the shapes into which the frame would *tend* to deform.

Stiffness of a Structural Member.—The stiffness of a beam or column is determined by its moment of intertia and its length. It is clear that the longer a member is the less will be its stiffness, and the greater the moment of inertia the greater will be the stiffness.

Therefore, if

K = stiffness,

l = length

and I = moment of inertia

then
$$K = \frac{I}{l} \qquad (28)$$

It has been pointed out that the stiffness of a member as a structural unit depends on the amount of rigidity with which it is held at both its ends. However, the total difference in stiffness between a column which is hinged at the bottom and

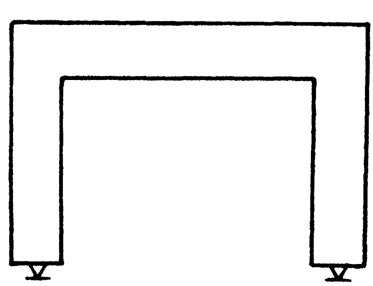
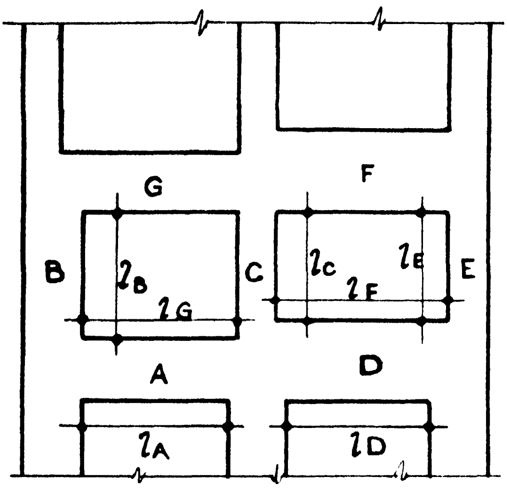


FIG. 281

a column which is rigidly fixed, provided that in either case there can be no relative horizontal movement, is only 25 per cent., and the calculation may be modified to adhere to average formulæ which are given and which take partial fixity into account. It should be remembered, however, that wherever a real hinge is provided the stiffness formula $K = \frac{I}{l}$ should be calculated on the corrected value of l. In the case of a column (or other member) which is hinged at one end, the length taken should be $1 \cdot 25l$, so that the formula for K would be varied to $K = \cdot 8 \frac{I}{l}$, in which l would be the actual length.

The length which should be used in computing the stiffness of a member should



l for member $G = l_G$
l ,, ,, $A = l_A$
l ,, ,, $B = l_B$
l ,, ,, $C = l_C$, etc.

FIG. 282

not be the distance between centre lines of the other members to which it is connected, but the clear distance between them. This will be understood from Fig. 282.

The question arises, what value of the moment of inertia should be used for a beam or column which tapers? It is suggested by the authors that in calculating any joint in a structure the designer use the moment of inertia at a section which is at a distance from the joint:

(*a*) of half the length in the case of a member which increases in size from the joint, and

(*b*) of one-quarter of the length in the case of a member which decreases in size from the joint.

Thus in Fig. 283 in order to investigate the influences of columns (1) and (2) on the joint A, the values of I_1 and I_2 would be adopted in calculating the values of K_1 and K_2, $\left(\frac{I_1}{l_1} \text{ and } \frac{I_2}{l_2}\right)$ respectively.

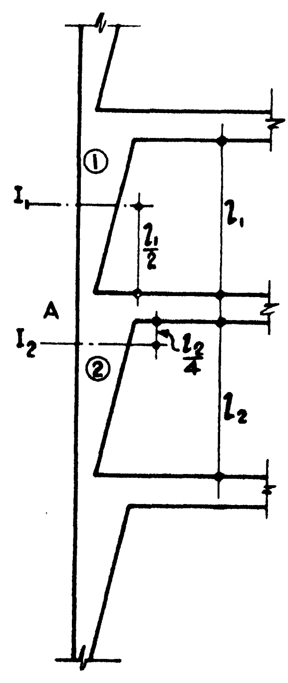


FIG. 283

It should be realised, of course, that such a joint as that of A, in Fig. 283, could be made with the lower end of the top column either virtually pin-jointed or rigid. In the former case no rotational restraint would be afforded to the joint by the top column; but allowance would have to be made for any horizontal thrust which might be set up by the frame above. In the second case, in which the rotational restraint of the top column would be used, the section of column (1) where it intersects the joint would require to be capable of taking the bending moment which the rotational restraint would set up.

Normally, it is sufficient to calculate the moment of inertia of a member on the section of the concrete alone, that is to say, by disregarding reinforcement. However, for heavily reinforced columns the results of this calculation are not in close enough agreement with facts, and therefore it is suggested that when the longitudinal reinforcement exceeds 3 per cent. a second calculation be made, which will indicate the proportion of the error and by which, therefore, the amount of reinforcement may be adjusted.

The Memorandum allows also two other ways of calculating the moment of inertia, provided, of course, that whichever of these ways is adopted is adhered to throughout. These methods are given here, although the authors do not recommend either of them. It is permitted to take either the entire concrete section, including reinforcement, or the compression area of the concrete alone combined with reinforcement. It is clear at the outset that both these ways involve rather complicated assumptions, for while the concrete of a beam has generally a constant moment of inertia throughout, the reinforcement and consequently the neutral axis vary, and therefore the taking of them into consideration involves a great amount of work for the designer.

The three foregoing methods of calculating the moment of inertia may be explained best by the example given in Figs. 284, 285, and 286.

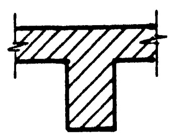


FIG. 284

In Fig. 284 the moment of inertia may be found for the gross area of the concrete alone, i.e. A = concrete (gross).

In Fig. 285 the moment of inertia must be found for the gross area of the concrete, plus the equivalent area of the compression steel regarded as concentrated along

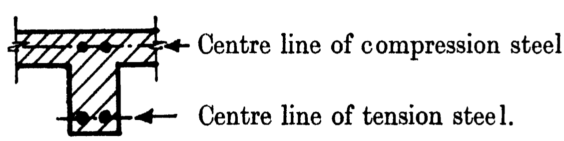


FIG. 285

its centre line, plus the equivalent area of the tension steel regarded as concentrated along its centre line, i.e. :

$$A \text{ concrete (gross)} + (m - 1)\, As \text{ (compression)} + (m - 1)\, As \text{ (tension)}.$$

In Fig. 286 the moment of inertia must be found for the area of the concrete which is in compression, plus the equivalent area of the compression steel regarded as

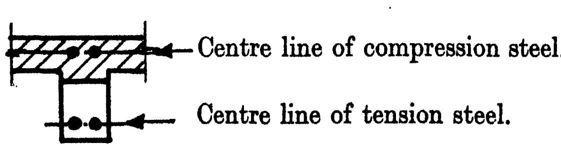


FIG. 286

concentrated along its centre line, plus the equivalent area of the tension steel regarded as concentrated along its centre line, i.e. :

$$A \text{ concrete (compression)} + (m - 1)\, As \text{ (compression)} + mAs \text{ (tension)}$$

It is necessary to introduce here two terms which are very useful in treating of mechanics of sections, etc. The first is "gravity line," which may be any line passing through the centre of gravity of an area. Since the centre of gravity is a point, an area can have an infinite number of gravity lines. Similarly, a group of areas may have gravity lines although some of them may not interesect any of the individual areas.

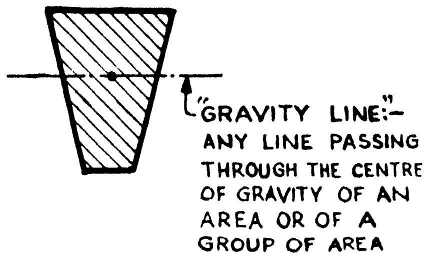


Fig. 287

The second term is "statical moment," which is the product of an area and the distance of its centre of gravity from any given line. Thus in Fig. 288 the statical moment of area A about a line xx is Ay. Or the statical moment of area A in Fig. 289 about a line x_1x_1 is Ay_1.

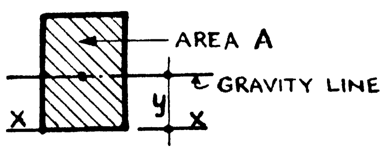


Fig. 288

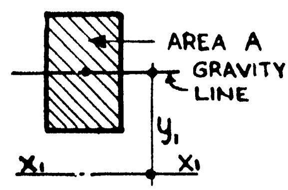


Fig. 289

It should be noted that the distance between the centre of gravity of the area and the line about which the statical moment is required can be taken as the distance between that line and a parallel gravity line of the area.

Figs. 290, 291, and 292 show how the centre-lines of reinforcement are to be taken for computing the moments of inertia of reinforced concrete sections as in connection with Figs. 285 and 286.

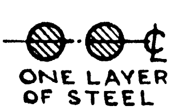


Fig. 290

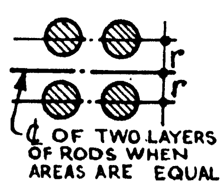


Fig. 291

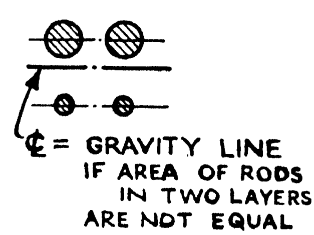


Fig. 292

In arriving at the value of the moment of inertia of tee-beams the actual flange width should be used (provided that it does not exceed the width recommended in the Memorandum), rather than the calculated flange width, which may be smaller. The Memorandum suggests that the maximum width, which should be used be the least of $\frac{1}{3} \times l$, d, or $b + 12t$, in which l is the effective span, d the distance between centre-lines of parallel tee-beams, b the width of the stem, and t the thickness of the flange. It would seem that these limitations have been chosen on the assumption that wider flanges do not behave as normal tee-beam flanges. This may be accepted, and the design must be so arranged that the limitations are not exceeded. The calculated flange width may be much less than the permissible, as may also be the actual width ; but the actual must not, of course, be less than the calculated.

It is important that the moment of inertia of any section be calculated as exactly as possible, and because this calculation becomes very involved where irregular sections obtain, as they do most frequently, it is proposed now to review the main rules by which moments of inertia are calculated and then to investigate the general and some special cases of irregular sections.

Moments of Inertia.*—The moment of inertia of any plane section about any axis xx is given by the expression

$$I_x = \Sigma y^2 \delta A,$$

in which δA is the area of an element of the section, and y is the distance of that element from the axis, y varying from zero in the case of elements at the axis to y_1 in the case of elements farthest from the axis. (See Fig. 293.)

If the width of the element be called δb and the depth δy, then the expression will become :

$$I_x = \Sigma y^2 \delta b \delta y,$$

and this may be rewritten in its more precise mathematical form :

$$I_x = \int b y^2 dy \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

in which b is the total width of the section at any distance y from the neutral axis.

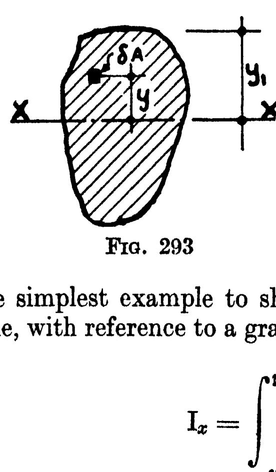


FIG. 293

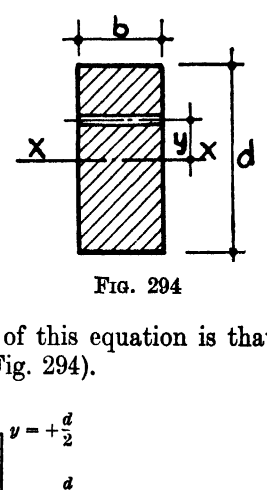


FIG. 294

The simplest example to show the application of this equation is that of the rectangle, with reference to a gravity line xx. (See Fig. 294).

$$I_x = \int_{y=-\frac{d}{2}}^{y=+\frac{d}{2}} by^2 dy = \left[\frac{by^3}{3}\right]_{y=-\frac{d}{2}}^{y=+\frac{d}{2}}$$

$$= \frac{bd^3}{24} - \left(-\frac{bd^3}{24}\right) = \frac{bd^3}{12}$$

The values of I_x, I_y, and I_{xy} for many common sections are given in Table 49. The centrifugal moment of inertia, I_{xy}, is explained on pages 247 and 251.

I_x for any other geometric section can generally be found readily by combining the values given for the simple sections. This may be done by adding the sum of all moments of inertia of the individual sections (about their own gravity lines), to the sum of the products of the individual areas and the squares of the distances

* Readers who are not interested in the mathematics may pass over expressions involving the calculus, but all other investigations will be easily followed.

TABLE 49

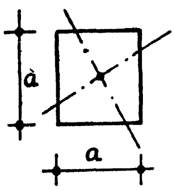
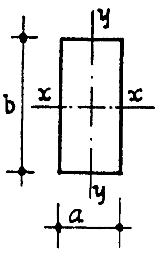
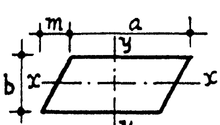
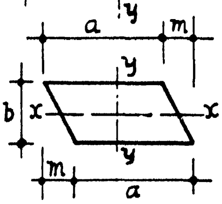
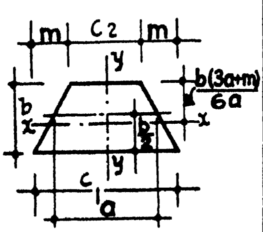
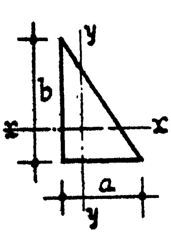
Section	Area	Moments of Inertia about axes through Centre of Gravity		
		I_x	I_y	Centrifugal I_{xy}
	a^2	$\frac{a^4}{12}$	$\frac{a^4}{12}$	0
	ab	$\frac{ab^3}{12}$	$\frac{ba^3}{12}$	0
	ab	$\frac{ab^3}{12}$	$\frac{ab}{12}(a^2+m^2)$	$+\frac{amb^2}{12}$
	ab	$\frac{ab^3}{12}$	$\frac{ab}{12}(a^2+m^2)$	$-\frac{amb^2}{12}$
	ab $=\frac{b(c_1+c_2)}{2}$	$\frac{b^3(3a^2-m^2)}{36a}$ $=\frac{b^3(c_1^2+4c_1c_2+c_2^2)}{36(c_1+c_2)}$	$\frac{ab}{12}(a^2+m^2)$ $=\frac{b}{48}(c_1+c_2)(c_1^2+c_2^2)$	0
	$\frac{ab}{2}$	$\frac{ab^3}{36}$	$\frac{ba^3}{36}$	 $-\frac{a^2b^2}{72}$  $+\frac{a^2b^2}{72}$

TABLE 49—*continued*

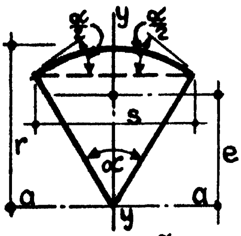
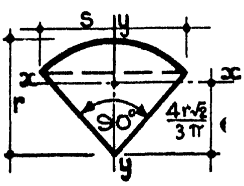
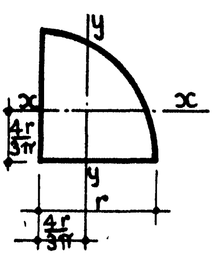
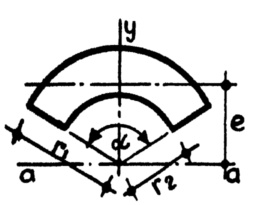
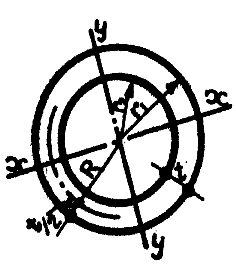
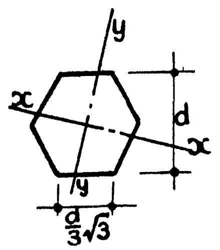
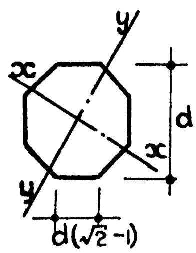
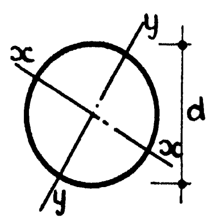
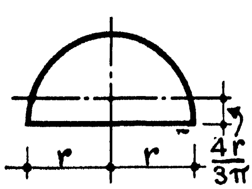
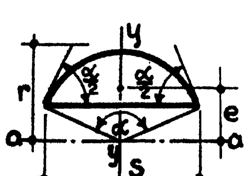
Section	Area	Moments of Inertia about through Centre of Gravity (unless otherwise stated)		
		I_x	I_y	I_{xy}
 $e = \frac{4r}{3} \times \frac{\sin\frac{\alpha}{2}}{\alpha} = \frac{2s}{3\alpha}$	$\frac{r^2\alpha}{2}$ $= \frac{s^2}{2} \times \frac{\alpha}{1-\cos\alpha}$	*for axis* a — a : $\frac{r^4}{8}(\alpha + \sin\alpha)$	$\frac{r^4}{8}(\alpha - \sin\alpha)$	0
	$\frac{r^2\pi}{4}$ $= \frac{s^2\pi}{8}$	$\frac{r^4}{8}\left(\frac{\pi}{2} + 1 - \frac{64}{9\pi}\right)$ $= 0{\cdot}0386r^4$	$\frac{r^4}{16}(\pi - 2)$ $= 0{\cdot}134r^4$	0
	$\frac{r^2\pi}{4}$	$\frac{r^4}{16}\left(\pi - \frac{64}{9\pi}\right)$ $= 0{\cdot}055r^4$	$\frac{r^4}{16}\left(\pi - \frac{64}{9\pi}\right)$ $= 0{\cdot}055r^4$	 $+\frac{r^4}{8}\left(\frac{32}{9\pi} - 1\right)$ $= +0{\cdot}0165r^4$  $-\frac{r^4}{8}\left(\frac{32}{9\pi} - 1\right)$ $= -0{\cdot}0165r^4$
 $e = \frac{4\sin\frac{\alpha}{2}}{3\alpha} \times \frac{r_1^2 + r_1r_2 + r_2^2}{r_1 + r_2}$	$\frac{(r_1^2 - r_2^2)\alpha}{2}$	*for axis* a — a $\frac{(r_1^4 - r_2^4)(\alpha + \sin\alpha)}{8}$	$\frac{(r_1^4 - r_2^4)(\alpha - \sin\alpha)}{8}$	0
	$\pi(r_1^2 - r_2^2)$ $= 2\pi t R$	$\frac{\pi}{4}(r_1^4 - r_2^4)$ If $\frac{t}{R} < 0{\cdot}4$: $\pi t R^3$ approx.	$\frac{\pi}{4}(r_1^4 - r_2^4)$ If $\frac{t}{R} < 0{\cdot}4$: $\pi t R^3$ approx.	0

TABLE 49—*continued*

Section	Area	Moments of Inertia about axes through Centre of Gravity (unless otherwise stated).		
		I_x	I_y	I_{xy}
	$\frac{d^2}{2}\sqrt{3}$ $= 0{\cdot}865d^2$	$\frac{5\sqrt{3}}{144}d^4$ $= 0{\cdot}060d^4$	$\frac{5\sqrt{3}}{144}d^4$ $= 0{\cdot}060d^4$	0
	$2d^2(\sqrt{2}-1)$ $= 0{\cdot}828d^2$	$\frac{4\sqrt{2}-5}{12}d^4$ $= 0{\cdot}055d^4$	$\frac{4\sqrt{2}-5}{12}d^4$ $= 0{\cdot}055d^4$	0
	$\frac{\pi d^2}{4}$ $= 0{\cdot}785d^2$	$\frac{\pi d^4}{64}$ $= 0{\cdot}049d^4$	$\frac{\pi d^4}{64}$ $0{\cdot}049d^4$	0
	$\frac{\pi r^2}{2}$ $= 1{\cdot}57r^2$	$r^4\left(\frac{\pi}{8}-\frac{8}{9\pi}\right)$ $= 0{\cdot}110r^4$	$\frac{\pi r^2}{8}$ $= 0{\cdot}393r^4$	0
 $e = \frac{4r\sin^3\frac{\alpha}{2}}{3(\alpha-\sin\alpha)}$ $= \frac{2s\sin^2\frac{\alpha}{2}}{3(\alpha-\sin\alpha)}$	$\frac{r^2}{2}(\alpha-\sin\alpha)$ $= \frac{s^2}{4}\left(\frac{\alpha-\sin\alpha}{1-\cos\alpha}\right)$	*for axis* a — a: $\frac{r^4}{8}(\alpha-\frac{1}{2}\sin 2\alpha)$	$\frac{r^4}{48}(6\alpha-8\sin+\sin 2\alpha)$	0

of the individual axes from the parallel gravity line of the whole section. Or, algebraically:

$$I_x = \Sigma I_{x_1} + \Sigma A_1 y_1^2 \quad . \quad . \quad . \quad . \quad . \quad (30)$$

in which I_{x_1}, A_1 and y_1 are the moments of inertia, areas and distances (individual axes from main axis) of the individual sections respectively.

The gravity line of the whole section must first be established, and this may be done by finding the centre of gravity—for example, from statical moments about one edge parallel to the axis which has to be established. The relationship:

$$A_1 d_1 + A_2 d_2 + A_3 d_3 \text{ etc.} = Ad$$

must be satisfied.

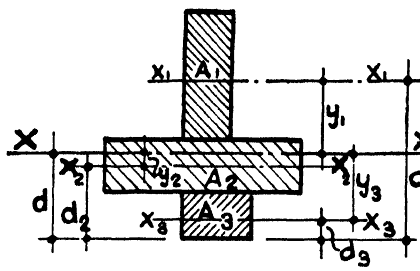


FIG. 295

For the composite section shown in Fig. 295, the moment of inertia about its gravity line would be:

$$I_x = I_{x_1} + I_{x_2} + I_{x_3} + A_1 y_1^2 + A_2 y_2^2 + A_3 y_3^2,$$

the individual moments of inertia being found in the table or from the general formula (29).

The combined moment of inertia of two separate areas A_1 and A_2 of which the

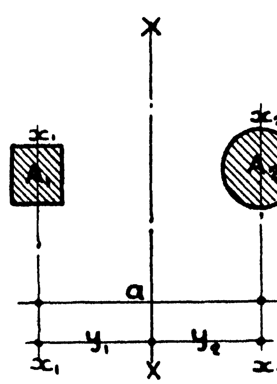


FIG. 296

individual axes x_1x_1 and x_2x_2 are at distances a apart, and y_1 and y_2 from the axis of the composite section, the total area being A, may be found from the formulæ:

$$I_x = I_{x_1} + I_{x_2} + \frac{A_1 A_2 a^2}{(A_1 + A_2)} \quad . \quad . \quad . \quad . \quad . \quad (31)$$

or

$$I_x = I_{x_1} + I_{x_2} + A y_1 y_2 \quad . \quad . \quad . \quad . \quad . \quad (32)$$

Here, and in all following remarks, all moments of inertia refer to gravity lines unless otherwise stated.

Moments of Inertia of Irregular Sections.—Irregular sections obviously have different Moments of inertia about different axes, and the axis about which the moment of inertia has its maximum value and the axis about which the least moment of inertia obtains, will be referred to as the " main axes." It is helpful if certain facts about these main axes be borne in mind.

(*a*) They are invariably at right angles to each other, whatever the shape of the section.

(*b*) Where the section has at least one geometric axis of symmetry, one of the main axes will coincide with it, the other being at right angles.

(*c*) When the moments of inertia for the main axes are already given or calculated, the moment of inertia about any other axis may be calculated directly. If the

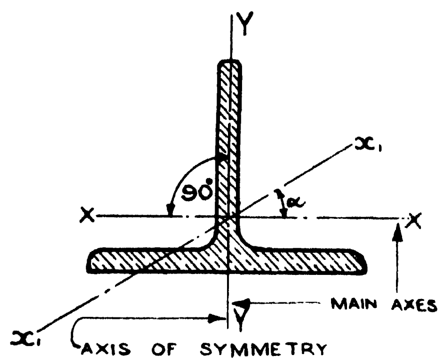


FIG. 297

moment of inertia is required about an axis x_1x_1 which makes an angle α with the main axis xx (see Fig. 297), then I_{x_1} will be given by :

$$I_{x_1} = \frac{I_x + I_y}{2} + \frac{I_x - I_y}{2} . \cos 2\alpha \quad . \quad . \quad . \quad (33)$$

and I_{y_1} about an axis at right angles to x_1x_1 by :

$$I_{y_1} = \frac{I_x + I_y}{2} - \frac{I_x - I_y}{2} . \cos 2\alpha \quad . \quad . \quad . \quad (34)$$

To avoid mistakes it should be borne in mind that these formulæ are valid only if xx and yy are main axes. Any moment of inertia may be calculated by this formula from the main moments of inertia, but not from any other known moment of inertia. (For the other cases, see Formula 30.)

(*d*) If the moments of inertia on the main axes are equal as in the cases of a square, a circle or an octagon, or if there are two or more axes of symmetry in a section in which the axes are not at right angles to each other, as in the cases of an equilateral triangle or a hexagon, then the moments of inertia about all axes are equal and all axes may therefore be regarded as main axes. For instance, the moments of inertia of squares and equilateral triangles of which the sides are a about any axis whatever are :

$$\frac{a^4}{12} \text{ and } \frac{a^4}{96} \sqrt{3} \text{ respectively.}$$

(*e*) In order to arrive at the main axes of an asymmetrical section it is necessary first to introduce the centrifugal moment, which in principle may be given as :

$$\begin{aligned} I_{xy} &= \Sigma \delta A xy \\ &= \Sigma x \delta x \, y \delta y \\ &= \iint x dx \, y dy \text{ (see Fig. 298)} \quad . \quad . \quad . \quad . \quad (35) \end{aligned}$$

The product xy has the following signs, the positions referring to Fig. 298.

TABLE 50

Quadrant				α							
I	II	III	IV	0 to 90°	90° to 180°	180° to 270°	270° to 360°	0 to $\frac{\pi}{2}$	$\frac{\pi}{2}$ to π	π to $\frac{3\pi}{2}$	$\frac{3\pi}{2}$ to 2π
+	−	+	−	+	−	+	−	+	−	+	−

It is essential in using Formula 35 that the designer observe very carefully a strict convention of signs, and it is proposed here to follow the system which is most commonly used, namely, that of regarding the distances y for all points above any original xx line (see Fig. 298) as positive and for all points below as negative, and at the same time of considering the distances x for all points to the right of line yy as positive

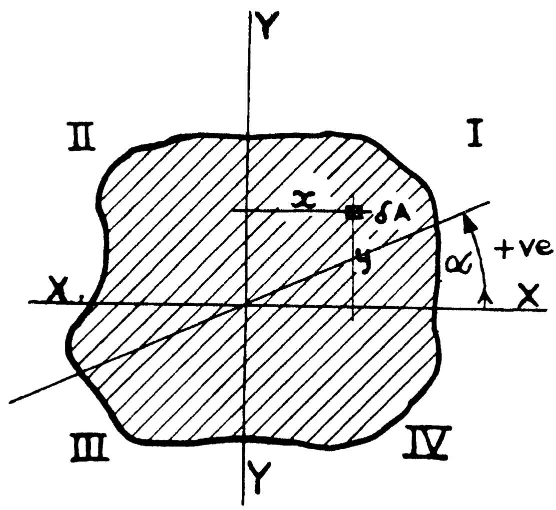


FIG. 298

and for those to the left as negative. Thus, by taking the product of the two signs of the x and y distances of a point in any quadrant, for all points in the top right quadrant the result will be positive ; for those in the top left, negative ; for those in the bottom left, positive ; and for those in the bottom right, negative. Angles measured anti-clockwise are regarded as positive (see Fig. 298).

If the values of I_{x_1}, I_{y_1} and $I_{x_1y_1}$ are known or calculated for any two axes of a section, the angle α which the main axis makes with the original x_1x_1 axis may be found from the equation

$$\tan 2\alpha = -\frac{2\ I_{x_1y_1}}{I_{x_1} - I_{y_1}}$$

or

$$\alpha = \tfrac{1}{2} \text{ arc tan} . \frac{-2\ I_{x_1y_1}}{I_{x_1} - I_{y_1}} \quad . \quad . \quad . \quad . \quad (36)$$

and the sign of this angle will indicate on which side of the original x_1x_1 axis the main axis will lie.

Tan 2α has always two meanings, for $\tan 2\alpha = \tan (2\alpha + \pi) = \tan 2 \left(\alpha + \frac{\pi}{2}\right)$.

The second value differs from the first, and because $\frac{\pi}{2} = 90°$ the second value refers to the other main axis.

The moments of inertia about the main axes xx and yy will be given by :

$$\left.\begin{array}{l} I_x \\ I_y \end{array}\right\} = \frac{I_{x_1} + I_{y_1}}{2} \pm \sqrt{\left(\frac{I_{x_1} - I_{y_1}}{2}\right)^2 + I_{x_1y_1}{}^2} \quad . \quad . \quad (37)$$

Example.—Find the maximum and minimum moments of inertia of the triangle of sides a and b in Fig. 299.

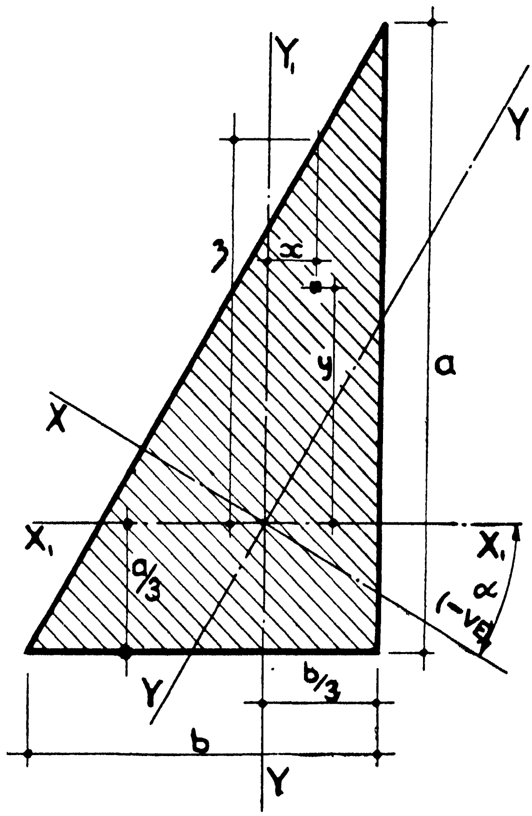


FIG. 299

First $I_{x_1} = \frac{1}{36} a^3b$ and $I_{y_1} = \frac{1}{36} b^3a$. (from Table 49)

In this example the derivation of the formula for the centrifugal moment of a triangle is given. It is not necessary, however, to follow this derivation in order to understand the theory of moments of inertia, as formulæ for the centrifugal moment as well as for the moment of inertia are given in Table 49 for most of the common sections. In this particular case :

$$I_{x_1y_1} = + \frac{a^2b^2}{72}.$$

Attention must be given to the sign of this expression. The centrifugal moment for a triangle with the toe on the right would be

$$I_{x_1y_1} = - \frac{a^2b^2}{72}.$$

The formula from which this expression is derived is :

$$I_{x_1y_1} = \int\int x dx\, y dy = \int x(\int y dy) dx.$$

In the triangle, for any particular value of x, y varies from $-\frac{a}{3}$ to a value $+z$ which is given by :

$$z = \frac{a}{3} + \frac{ax}{b}.$$

Therefore
$$\int_{-\frac{a}{3}}^{+z} y dy = \left[\frac{y^2}{2}\right]_{-\frac{a}{3}}^{\left(\frac{a}{3}+\frac{ax}{b}\right)}$$

$$= \tfrac{1}{2}\left(\frac{a}{3} + \frac{ax}{b}\right)^2 - \frac{a^2}{18}$$

$$= \frac{a^2x}{6b^2}(2b + 3x).$$

Substituting this value in the original equation : $I_{x_1y_1} = \int x \left[\frac{a^2x}{6b^2}(2b + 3x)\right] dx$; and because x varies from $-\frac{2b}{3}$ to $+\frac{b}{3}$ this becomes

$$I_{x_1y_1} = \frac{a^2}{6b^2}\left[\frac{2bx^3}{3} + \frac{3x^4}{4}\right]_{-\frac{2b}{3}}^{+\frac{b}{3}} = \frac{a^2b^2}{72}.$$

The value of α is given by (Formula (36)) :

$$\tan 2\alpha = \frac{-2\, I_{x_1y_1}}{I_x - I_{y_1}}$$

$$= \frac{-2\,(a^2b^2)}{72\left(\frac{a^3b}{36} - \frac{ab^3}{36}\right)}$$

$$= \frac{-a^2b^2}{ab\,(a^2 - b^2)}$$

$$\tan 2\alpha = \frac{-ab}{(a^2 - b^2)}$$

and because a is greater than b this will be negative, and α therefore will be negative.

By substituting the values of I_{x_1}, I_{y_1} and $I_{x_1y_1}$ the main moments of inertia are found to be (Formula (37)) :

$$I_x = \tfrac{1}{2}\left(\frac{a^3b}{36} + \frac{ab^3}{36}\right) + \sqrt{\frac{(a^3b - ab^3)^2}{72^2} + \frac{(a^2b^2)^2}{72^2}},$$

and
$$I_y = \tfrac{1}{2}\left(\frac{a^3b}{36} + \frac{ab^3}{36}\right) - \sqrt{\frac{(a^3b - ab^3)^2}{72^2} + \frac{(a^2b^2)^2}{72^2}}$$

which, being simplified, give :

$$I_x = \frac{ab}{72}\left(a^2 + b^2 + \sqrt{a^4 + b^4 - a^2b^2}\right)$$

and
$$I_y = \frac{ab}{72}\left(a^2 + b^2 - \sqrt{a^4 + b^4 - a^2b^2}\right).$$

In the particular case in which $a = b$, $\alpha = \frac{1}{2}$ arc tan infinity. Arc tan infinity $= 90°$, therefore $\alpha = 45°$, and

$$I_x = \frac{a^2}{72}(2a^2 + a^2) = \frac{a^4}{24}$$

and $$I_y = \frac{a^2}{72}(2a^2 - a^2) = \frac{a^4}{72}.$$

(*f*) The centrifugal moment of a composite section about any axes x_1x_1 and y_1y_1 is equal to the sum of the centrifugal moments of all the individual parts about

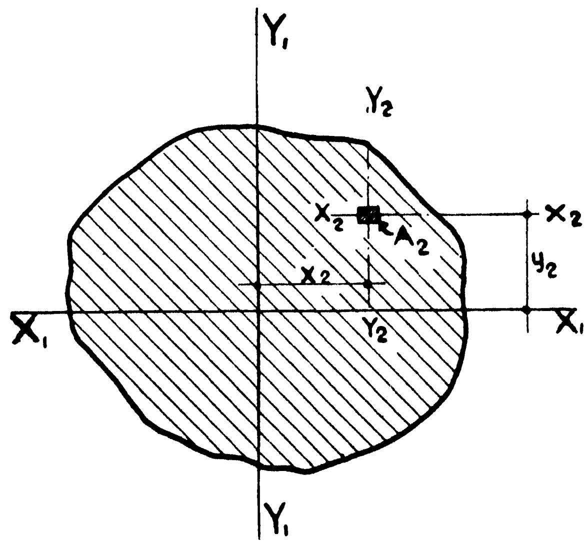


FIG. 300

their own axes (these axes passing through the individual centres of gravity and being parallel to the x_1x_1, y_1y_1 axes of the whole area) plus the sum of the products of the individual areas multiplied by each of the x_2 and y_2 ordinates of their centres of gravity from the x_1x_1 and y_1y_1 axes of the whole area. See Fig. 300.

Or, algebraically :

$$I_{x_1y_1} = \Sigma I_{x_2y_2} + \Sigma A_2x_2y_2 \quad . \quad . \quad . \quad . \quad (38)$$

It is very important that attention be given to the convention of signs, because although moments of inertia are always positive the centrifugal moment may be either positive or negative and obviously the correct sign must be adopted.

To find the maximum and minimum moments of inertia and the position of the main axes of a composite section, one has first to find the moments of inertia about any x_1x_1 and y_1y_1 axes passing through the centre of gravity, and the centrifugal moment of the whole area about these same axes. Obviously it is simplest to select the x_1x_1 and y_1y_1 axes so that they shall be parallel to the main axes of the individual sections or at least to as many of the individual main axes as possible, because the centrifugal moments about such main axes are zero. When they are parallel to all individual main axes the centrifugal moment for the whole area becomes :

$$I_{x_1y_1} = \Sigma A_2x_2y_2 \quad . \quad . \quad . \quad . \quad . \quad (39)$$

Example.—A column section is composed of two rectangles, A_1 and A_2 in Fig. 301.

Two axes x_1x_1 and y_1y_1 are selected passing through the centre of gravity of the whole area and parallel to the main axes of the individual areas.

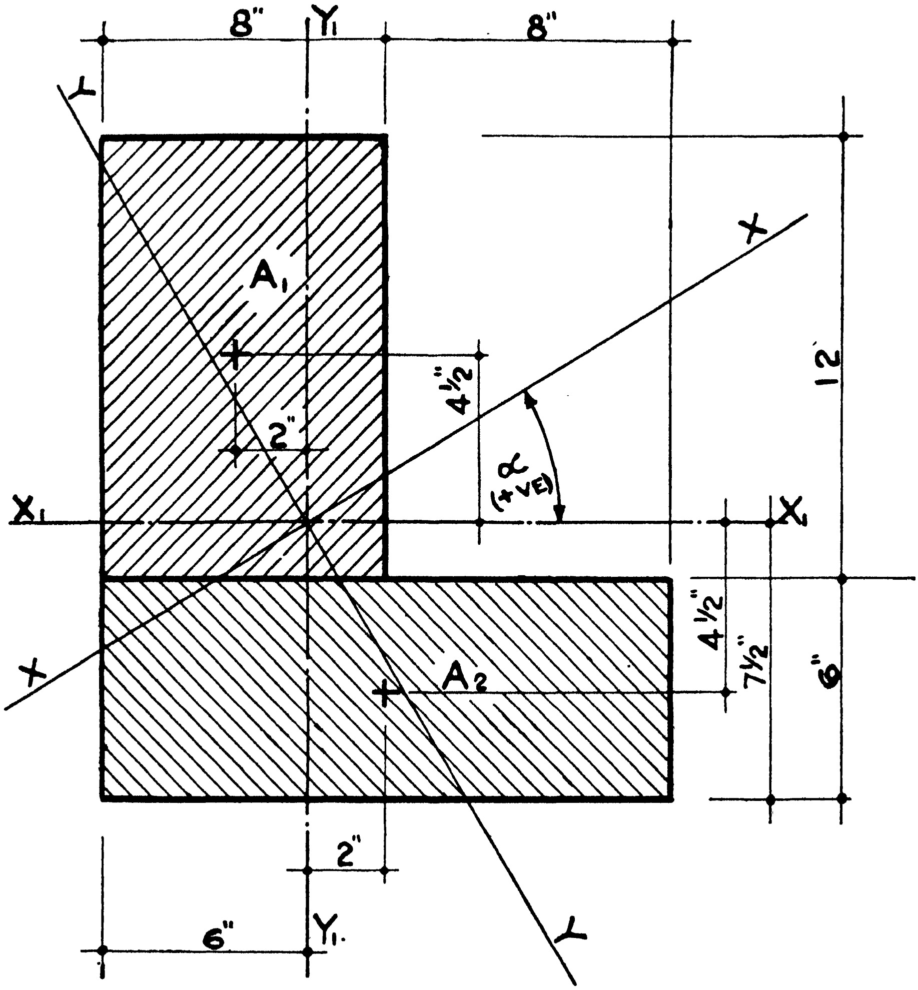


FIG. 301

Then according to the method given previously (Formula (30)):

$$I_{x_1} = \left(\frac{8 \times 12^3}{12}\right) + \left(\frac{16 \times 6^3}{12}\right) + (8 \times 12 \times 4{\cdot}5^2) + (16 \times 6 \times 4{\cdot}5^2)$$
$$= 5328 \text{ in.}^4$$

and

$$I_{y_1} = \left(\frac{12 \times 8^3}{12}\right) + \left(\frac{6 \times 16^3}{12}\right) + (8 \times 12 \times 4{\cdot}5^2) + (16 \times 6 \times 4{\cdot}5^2)$$
$$= 3328 \text{ in.}^4$$

$$I_{x_1y_1} = (A_2 \times (-4{\cdot}5) \times 2) + (A_1 \times 4{\cdot}5 \times (-2))$$

(because individual centrifugal moments are zero)

$$= (16 \times 6 \times -9) + (12 \times 8 \times -9)$$
$$= -1728 \text{ in.}^4$$

Then $\tan 2\alpha = -\left(\frac{2 \times (-1728)}{5328 - 3328}\right) = 1{\cdot}728$

$\alpha = \frac{1}{2} \times 59° \, 57' = 29° \, 58'$ approx. (say 30°) in positive direction.

The maximum moment of inertia, that is the moment of inertia about the main xx axis for the whole area, is given by :

$$I_x = \frac{5328 + 3328}{2} + \sqrt{\left(\frac{5328 - 3328}{2}\right)^2 + (-1728)^2}$$

$$= 4328 + 1992 = 6320 \text{ in.}^4$$

And the minimum moment of inertia, i.e. about yy, is :

$$I_y = 4328 - 1992 = 2336 \text{ in.}^4$$

The smallest radius of gyration is that about the main axis yy and its value is :

$$r \text{ (min.)} = \sqrt{\frac{2336}{192}}$$

$$= 3{\cdot}49 \text{ in.}$$

Example.—A column section is composed of three areas A_1, A_2 and A_3 as shown in Fig. 302. The centre of gravity must first be established, as in the example given

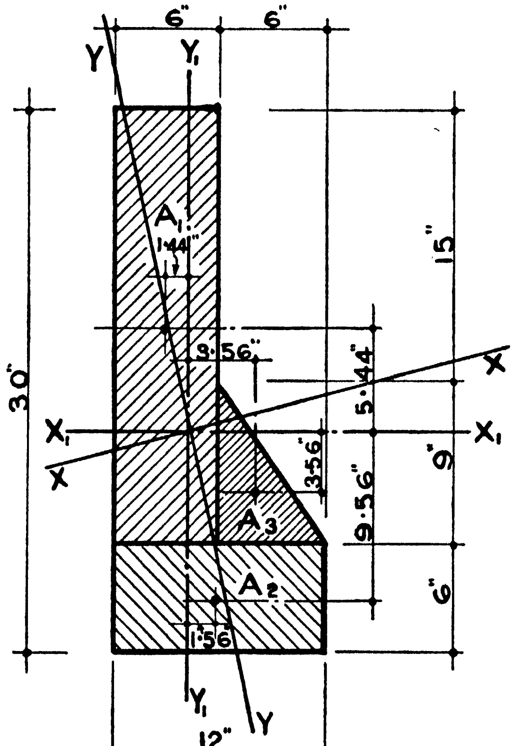


Fig. 302

in Fig. 295. Then, as before, two axes x_1x_1 and y_1y_1 are selected passing through the centre of gravity of the whole area and parallel to the main axes of the areas A_1 and A_2 :

$$I_{x_1} = \left(\frac{6 \times 24^3}{12}\right) + \left(\frac{12 \times 6^3}{12}\right) + \left(\frac{6 \times 9^3}{36}\right) + (6 \times 24 \times 5{\cdot}44^2)$$

$$+ (12 \times 6 \times 9{\cdot}56^2) + \left(\frac{9 \times 6 \times 3{\cdot}56^2}{2}\right)$$

$$= 18{,}433 \text{ in.}^4$$

and

$$I_{y_1} = \left(\frac{24 \times 6^3}{12}\right) + \left(\frac{6 \times 12^3}{12}\right) + \left(\frac{9 \times 6^3}{36}\right) + (24 \times 6 \times 1{\cdot}44^2)$$
$$+ (6 \times 12 \times 1{\cdot}56^2) + \left(\frac{9 \times 6 \times 3{\cdot}56^2}{2}\right)$$
$$= 2165 \text{ in.}^4$$

$$I_{x_1y_1} = (-\,24 \times 6 \times 1{\cdot}44 \times 5{\cdot}44) - (12 \times 6 \times 1{\cdot}56 \times 9{\cdot}56)$$
$$- \left(\frac{9 \times 6}{2} \times 3{\cdot}56 \times 3{\cdot}56\right) - \left(\frac{9^2 \times 6^2}{72}\right)$$
$$= -\,2577 \text{ in.}^4$$

$$\tan 2\alpha = \frac{2 \times 2577}{18{,}433 - 2165}$$
$$= +\,0{\cdot}317$$
$$\alpha = 8^\circ\, 47' \text{ in positive direction.}$$

$$I_x = \frac{18{,}433 + 2165}{2} + \sqrt{\left(\frac{18{,}433 - 2165}{2}\right)^2 + (\,-\,2577)^2}$$
$$= 10{,}299 + 8532 + 18{,}831 \text{ in.}^4$$
$$I_y = 10{,}299 - 8532 - 1767 \text{ in.}^4$$

Least radius of gyration :

$$r = \sqrt{\frac{1767}{243}} = 2{\cdot}695 \text{ in.}$$

It will be noticed in this case, particularly, that the actual least moment of inertia is much less than that about the y_1y_1 axis, which might otherwise be assumed to be sufficient for purposes of calculation. I_y is in fact only 81 per cent. of I_{y_1}, and as the main axis is only 8·75° off the y_1y_1 axis it may reasonably be concluded that for all but very short columns deformation would actually take place about the main yy axis and not about the assumed y_1y_1 axis. It will be seen, therefore, that an exact calculation is not only justified but necessary.

However, it should be realised that there are certain more or less obvious cases in which an irregular-shaped member may be adequately stiffened continuously or at points along its length in such a manner that deformation cannot possibly take place about the main axis of least moment of inertia.

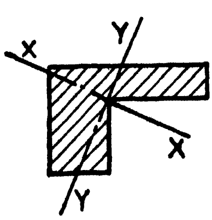


FIG. 303

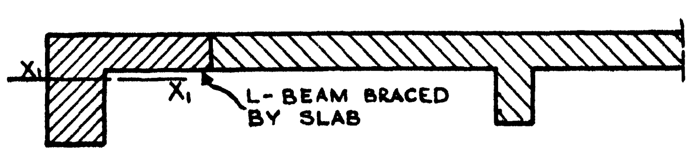


FIG. 304

For instance, the L-shaped beam (Fig. 303) if located at the edge of the slab in Fig. 304 may be assumed to be so restrained by the slab that any movement in the direction of the main axis yy (that is to say about the yy axis) is precluded by the attachment of the beam to the slab, and in this case it would be correct to calculate the member on its properties about the x_1x_1 axis.

Other Considerations

The notes on the preceding pages should be sufficient to enable any problem with which the designer is faced in arriving at any moment of inertia, to be overcome. But before any structure can be investigated by a method in which the relative stiffnesses of the members are taken into consideration, other problems governing the effective lengths of the members must be solved. In a previous paragraph of this chapter it has been pointed out that the length should be taken as the clear distance between adjoining structural members. This is correct for the computation of the stiffness; but for the actual design of columns acting as such (their influences on the rest of the structure having previously been investigated or ignored, as the particular cases warranted), the permitted stress is governed by considerations of buckling, or in other words by the two factors, radius of gyration and " effective length."

For only three of the six conditions of end fixing is the " effective length " defined exactly in each of By-laws 86 and 102, which control structural steel and reinforced concrete columns respectively. For the other three cases of each by-law, limits have been specified between which the " effective length " must lie and the exact values have been made dependent on the efficiency or amount of end restraint. However, no indication is given as to how intermediate values are to be assessed, but it is clear that they must be to the satisfaction of the district surveyor.

Rotational Restraint in Determining Effective Length of a Column.—There is, in actual fact, never a case in which a column is absolutely restrained against rotation at the ends; but in arriving at the " effective lengths," within the limits specified for any particular condition, full rotational restraint may be assumed for the more favourable limit and none for the less.

If the sum of the stiffnesses (see page 238) of all external restraining members at one end of a column be called K and if the column stiffness (calculated on the full clear length or height) be K_c, then a ratio $R = \frac{K}{K + K_c}$ may be found from which the effective length of the column may be obtained. R may vary from zero for hinged joints to 1 for joints which are 100 per cent. restrained against rotation.

Further, if R_T and R_B be the ratios at the top and bottom of a column respectively such that

$$R_T = \frac{K_T}{K_T + K_C} \text{ and } R_B = \frac{K_B}{K_B + K_C}$$

in which formulæ K_T and K_B represent the total external restraint acting at the top and bottom respectively, then R for the whole column becomes

$$R = \frac{R_T + R_B}{2} \quad . \quad . \quad . \quad . \quad . \quad (40)$$

Tables 52 and 53 have been compiled setting out the effective lengths of columns for four of the six conditions covered by By-laws 86 and 102. The six conditions are given in Table 51.

Table 52 gives the length in inches for the conditions A, B, D (see Table 51).

Table 53 is compiled for various values of rotational restraint in case E; but for imperfect positional restraint (cases C and F in Table 51), which will be explained later, one must refer to Table 54.

It should be noted particularly that, although the clear lengths of columns are adopted for the computation of their stiffnesses, the slenderness ratios must be calculated from the " storey height " lengths (i.e. normally, floor to floor) or from these lengths modified in accordance with the end restraints by the method suggested here or on page 260, on which positional restraint is discussed. In other words, the "storey height" length is to be adopted for the purpose of Tables 52 and 53 (and later for Table 54).

For the purpose of Tables 52 and 53 the following are the classifications of the columns:

TABLE 51

	Height of Columns	End Restraint
A	One storey	Full positional and rotational restraints at both ends.
B	One storey	Full positional restraint at both ends. Imperfect rotational restraint at either end.
C	One storey	Full positional and rotational restraints at one end and imperfect positional and rotational restraints at the other end.
D	Two or more storeys	Full positional and rotational restraints at both ends in a storey height.
E	Two or more storeys	Full positional restraint at both ends in a storey height, and imperfect rotational restraint at one or both ends in that storey height.
F	Two or more storeys	Full positional and rotational restraints at one end in a storey height and imperfect positional and rotational restraints at the other end in that storey height.

TABLE 52

EFFECTIVE COLUMN LENGTHS IN TERMS OF ACTUAL LENGTH AND END FIXINGS

Actual Column Length or Storey Height	Effective Length See Table 51		Actual Column Length or Storey Height	Effective Length See Table 51	
	Classes A and D	Class B		Classes A and D	Class B
ft. in.	in.	in.	ft. in.	in.	in.
6 0	54	72	13 6	122	162
6 6	58½	78	14 0	126	168
7 0	63	84	14 6	131	174
7 6	67½	90	15 0	135	180
8 0	72	96	15 6	140	186
8 6	77	102	16 0	144	192
9 0	81	108	16 6	149	198
9 6	85½	114	17 0	153	204
10 0	90	120	17 6	158	210
10 6	95	126	18 0	162	216
11 0	99	132	18 6	167	222
11 6	104	138	19 0	171	228
12 0	108	144	19 6	176	234
12 6	113	150	20 0	180	240
13 0	117	156			

TABLE 53

EFFECTIVE COLUMN LENGTHS IN TERMS OF ACTUAL LENGTH AND END FIXINGS

Actual Column Length or Storey Height	Effective Lengths of Columns in Inches — Ratio of Rotational Restraint at Column Ends for Class "E" in Table 51										
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0
ft. in.											
6 0	72	70	68	67	65	63	61	59	58	56	54
6 6	78	76	74	72	70	68	66	64	62	60	58
7 0	84	82	80	78	76	74	71	69	67	65	63
7 6	90	88	85	83	81	79	76	74	72	69	67
8 0	96	94	91	89	86	84	82	79	77	74	72
8 6	102	99	97	94	92	89	86	84	81	79	76
9 0	108	105	103	100	97	95	92	89	86	84	81
9 6	114	111	108	105	102	99	97	94	91	88	85
10 0	120	117	114	111	108	105	102	99	96	93	90
10 6	126	123	120	116	113	110	107	104	100	97	94
11 0	132	129	125	122	119	115	112	109	105	102	99
11 6	138	135	131	128	124	121	117	114	110	107	103
12 0	144	140	137	133	130	126	122	119	115	111	108
12 6	150	146	142	139	135	131	127	124	120	116	112
13 0	156	152	148	144	140	136	133	129	125	121	117
13 6	162	158	154	150	146	142	138	134	130	126	122
14 0	168	164	160	155	151	147	143	139	134	130	126
14 6	174	170	165	161	156	152	148	144	139	134	130
15 0	180	176	171	167	162	158	153	149	144	139	135
15 6	186	181	177	172	167	162	158	153	149	144	139
16 0	192	187	182	178	173	168	163	158	154	149	144
16 6	198	193	188	183	178	173	168	163	159	153	148
17 0	204	199	194	189	184	178	173	168	163	158	153
17 6	210	205	199	194	189	184	178	173	168	162	157
18 0	216	211	205	200	194	189	184	178	173	167	162
18 6	222	216	211	205	200	194	189	183	178	172	166
19 0	228	222	217	211	205	199	194	188	182	177	171
19 6	234	228	222	216	210	204	199	193	187	181	175
20 0	240	234	228	222	216	210	204	198	192	186	180

The calculation of R will be illustrated by a practical example shown in Fig. 305, in which it is proposed to investigate column (2). It is assumed for simplicity of illustration that the column heights in the floors above and below are the same as that of column 2.

For the top of column 2 the stiffness of each of beams 1 and 4 is :

$$K_1 = K_4 = \frac{I_1}{l_1} = \frac{1414}{15} = 94{\cdot}3,$$

and the stiffness of column 7 is

$$K_7 = \frac{I_7}{l_7} = \frac{108}{10} = 10{\cdot}8.$$

Therefore, the total external restraint at the top of column 2 is :

$$K_{2T} = K_1 + K_4 + K_7 = 2 \times 94{\cdot}3 + 10{\cdot}8 = 199{\cdot}4.$$

The stiffness of column 2 itself is

$$K_2 = \frac{I_2}{l_2} = \frac{108}{10} = 10{\cdot}8,$$

and thus

$$R_{2T} = \frac{K_{2T}}{K_{2T} + K_2} = \frac{199 \cdot 4}{199 \cdot 4 + 10 \cdot 8} = 0 \cdot 95.$$

Also, for the bottom of column 2, $K_3 = K_5 = 94 \cdot 3$ and the stiffness of column 6 is :

$$K_6 = \frac{I_6}{l_6} = \frac{864}{10} = 86 \cdot 4.$$

Therefore, the total external restraint at the bottom of column 2 is :

$$K_{2B} = K_3 + K_5 + K_6 = 2 \times 94 \cdot 3 + 86 \cdot 5 = 275$$

and thus :

$$R_{2B} = \frac{K_{2B}}{K_{2B} + K_2} = \frac{275}{275 + 10 \cdot 8} = 0 \cdot 97.$$

From R_{2T} and R_{2B}, R becomes :

$$\frac{0 \cdot 95 + 0 \cdot 97}{2} = 0 \cdot 96,$$

and from Table 53 the effective length of column 2 will be seen to be 100 in.

Note that this value has been taken from the "storey height" length of column, namely, 11 ft. 0 in.

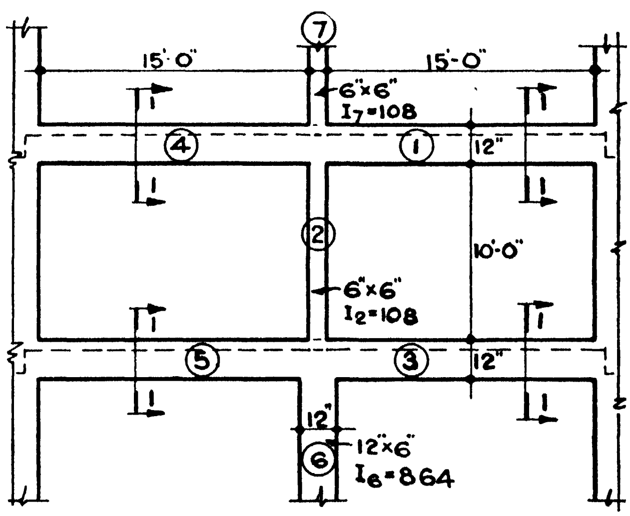


FIG. 305

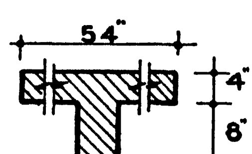


FIG. 306

In the foregoing example it has been assumed that column 2 was braced adequately in the opposite direction, that is to say, at right angles to the plane of the frame which was considered to afford the restraint. If, however, there is no such bracing and if also there is just the 4-in. concrete slab at beam levels spanning in the transverse direction, it would be necessary to investigate the column for its stiffness in that transverse direction.

One is immediately faced with the principal question of the amount of slab which may be considered to restrain the column. It would appear reasonable to take a width of slab of three times the column dimension in the same direction for this purpose. At the same time some consideration should be given to the fact that the slab thickness will have a considerable influence on this width ; but although the suggested value of three times the column width is conservative for narrow columns

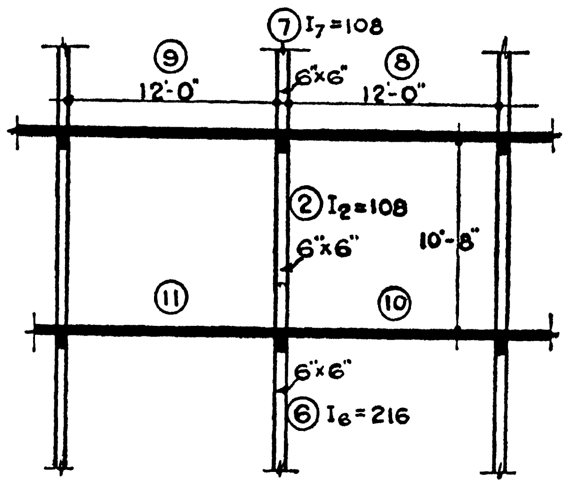


FIG. 307

and perhaps slightly excessive for wide columns, the authors, nevertheless, recommend its use, as it applies to the average case.

In Fig. 307 a section through a building is given at right angles to the section in Fig. 305. Then if a width of slab of 3×6 in. $= 18$ in. be taken as stiffening the 6-in. wide column (2), I_8 and I_9 will become :

$$\frac{18 \times 4^3}{12} = 96 ;$$

and
$$K_8 \text{ and } K_9 = \frac{I_8}{l_8} = \frac{96}{12} = 8 ;$$

$$K_7 = \frac{108}{10 \cdot 67} = 10 \cdot 1.$$

Therefore
$$K_{2T} = K_7 + K_8 + K_9 = 2 \times 8 + 10 \cdot 1 = 26 \cdot 1$$
$$K_2 = 10 \cdot 1$$

and
$$R_{2T} = \frac{K_{2T}}{K_{2T} + K_2} = \frac{26 \cdot 1}{26 \cdot 1 + 10 \cdot 1} = 0 \cdot 72 ;$$

$$K_{10} = K_{11} = K_8 = 8 ;$$

$$K_6 = \frac{I_6}{l_6} = \frac{216}{10 \cdot 67} = 20 \cdot 3 ;$$

$$K_{2B} = K_6 + K_{10} + K_{11} = 20 \cdot 3 + 2 \times 8 = 36 \cdot 3.$$

Then
$$R_{2B} = \frac{K_{2B}}{K_{2B} + K_2} = \frac{36 \cdot 3}{36 \cdot 3 + 10 \cdot 1} = 0 \cdot 78.$$

From which

$$R_2 = \frac{R_{2T} + R_{2B}}{2} = \frac{0{\cdot}72 + 0{\cdot}78}{2} = 0{\cdot}75.$$

The effective length will be 108 in. from Table 53.

Another problem on which there is no scientific evidence to guide the designer is that of the amount of rotational restraint which may be afforded by the foundations or bases of columns. The nature of the ground on which the foundation stands will obviously have some influence on the settlement which can take place at any point, and also any earth pressure round the sides will contribute to the restraint. In regard to the first of these influences, there is the fact to be considered that weakness of the ground will be compensated to a large extent by the reduced bearing pressure which will be allowed and the consequent wider spread of the base. In the absence of any reliable scientific data on the whole of this problem, assumptions must be made, and for the average case it is suggested that if the width of the foundation block or slab be not less than four times the column width, measured in the same direction, and if also its thickness immediately below the column be not less than twice the column thickness, then the column may be regarded as 100 per cent. restrained against rotation; on this assumption the value of unity for R could be used in arriving at the column buckling length. *

Positional Restraint.—Consideration must also be given to the amount of positional restraint which is afforded to a column. Instances of columns not fully restrained in position are given in Figs. 308, 309, and 310.

In the first of these A represents a column carrying a gantry rail tied at the top

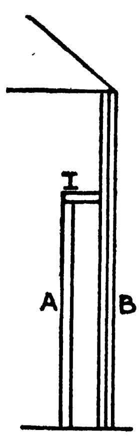


FIG. 308

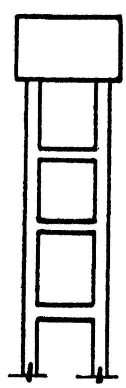


FIG. 309

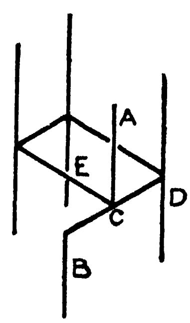


FIG. 310

to another column B, which continues past the gantry level to form the main stanchion of the building. The main column may be slender or stiff, depending on external requirements, and therefore the amount of positional restraint which the gantry column will receive, at least in one direction, will depend entirely on the stiffness of column B. In the second case an elevated water-tank is shown carried on four columns, there being intermediate horizontal beams or slabs at various levels between the tank and foundations. Then, because all four columns may move in the same direction, there will be no positional restraint whatever if the joints are free or virtually hinged and only that amount afforded by the internal stiffness of the columns themselves if the joints are rigid.

In Fig. 310 an arrangement is shown of a small portion of a framed structure in which it has not been possible to carry column A through to the lower level, and

* See also remarks on p. 314 under Note 3.

its load together with that of beam E has been brought to column B by means of beam C. Then the restraint of the top of column B in position will depend on the lateral stiffness of beam C—that is to say, on the stiffness of B about the axis at right angles to that on which its section for load carrying was determined.

In order to find a measure of the positional restraint it is necessary to take the resistance to deflection of the actual member in relation to the total resistance of the whole structure. In princple, if δ be the deflection under a lateral force of unity then the resistance to positional deformation at the top, D may be defined as $\frac{1}{\delta}$. This is more or less obvious, because the greater the resistance the less will be the deflection. Then if the symbol D be applied to the total resistance of all external members restraining the column and if the internal restraint of the column itself be D_1 a ratio P may be found :

$$P = \frac{D}{D + D_1}$$

from which the effective length of the column may be obtained.

This applies to cases C and F in Table 51, and for these Table 54 may be used.

This formula is analogous to that for R $\left(R = \frac{K}{K + Kc}\right)$, in which the stiffnesses of all the external members and of the individual member are replaced by the resistance to lateral deformation of all the external members and of the individual member

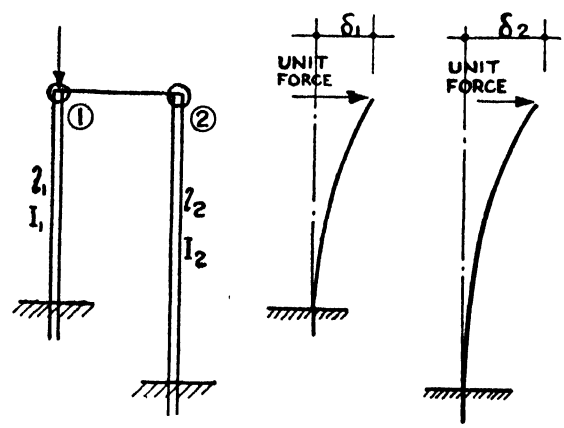


FIG. 311

respectively. In the same manner P will vary from zero when there is no external resistance to unity when the external resistance is 100 per cent. According to By-laws 86 and 102 the effective length will vary from the actual length when $P = 1$ to twice that length $P \rightarrow 0$. An example will probably make this clearer. See Fig. 311.

A column (1), free at the top except for a hinged connection to another column (2), carries a vertical load and has a length l_1 and a moment of inertia I_1, the length of the second column and the moment of inertia being l_2 and I_2 respectively.

A unit horizontal force at the top of column (1) would produce a deflection :

$$\delta_1 = \frac{l_1^3}{3EI_1};$$

and a similar unit force at the top of column (2) would produce a deflection :

$$\delta_2 = \frac{l_2^3}{3EI_2}.$$

Then $D_1 = \frac{1}{\delta_1} = \frac{3EI_1}{l_1^3}$, and $D = D_2 = \frac{1}{\delta_2} = \frac{3EI_2}{l_2^3}$. Therefore

$$P = \frac{D}{D + D_1} = \frac{\frac{3EI_2}{l_2^3}}{\frac{3EI_2}{l_2^3} + \frac{3EI_1}{l_1^3}}$$

$$= \frac{\frac{I_2}{l_2^3}}{\frac{I_2}{l_2^3} + \frac{I_1}{l_1^3}}$$

$$= \frac{I_2 l_1^3}{I_2 l_1^3 + I_1 l_2^3} \quad . \quad . \quad . \quad . \quad . \quad (41)$$

If, for example, there were yet another external column (3) of resistance D_3, then D would become $D_2 + D_3$; and so for all the external bracing members D would become $D_2 + D_3 + D_4$, etc.

Table 54 gives the values of the effective or buckling lengths of columns for various values of P and of the actual column length.

TABLE 54

Actual Column Length or Storey Height	Effective Lengths of Columns in Inches — Ratio of Positional Restraint at Column Ends—for Classes C and F in Table 51										
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	1·0
ft. in.											
6 0	144	137	130	122	115	108	101	94	86	79	72
6 6	156	148	140	133	125	117	109	101	94	86	78
7 0	168	160	151	143	135	126	118	109	101	93	84
7 6	180	171	162	153	144	135	126	117	108	99	90
8 0	192	182	173	163	154	144	134	125	115	106	96
8 6	204	194	184	173	163	153	143	133	122	112	102
9 0	216	205	194	184	174	162	151	140	130	119	108
9 6	228	217	205	194	182	171	160	148	137	125	114
10 0	240	228	216	204	192	180	168	156	144	132	120
10 6	252	239	227	214	202	289	176	164	151	139	126
11 0	264	251	238	224	211	198	185	172	158	145	132
11 6	276	262	248	235	221	207	193	179	166	152	138
12 0	288	274	259	245	230	216	202	187	173	158	144
12 6	300	285	270	255	240	225	210	195	180	165	150
13 0	312	296	281	265	250	234	218	203	187	172	156
13 6	324	308	292	275	259	243	227	211	194	178	162
14 0	336	319	302	286	267	252	235	218	202	185	168
14 6	348	331	313	296	278	261	244	226	209	191	174
15 0	360	342	324	306	288	270	252	234	216	198	180
15 6	372	353	335	316	298	279	260	242	223	205	186
16 0	384	365	346	326	307	288	269	250	230	211	192
16 6	396	376	356	337	317	297	277	257	238	218	198
17 0	408	388	367	347	326	306	286	265	245	224	204
17 6	420	399	378	357	336	315	294	273	252	231	210
18 0	432	410	389	367	346	324	302	281	259	238	216
18 6	444	422	400	377	355	333	311	289	266	244	222
19 0	456	433	410	388	365	342	319	296	273	251	228
19 6	468	445	421	398	374	351	328	304	281	257	234
20 0	480	456	432	408	284	360	336	312	288	264	240

Distribution of Moments

End Columns.—Bending moments may occur in any column, but in certain columns, for which approximately symmetrical beam and loading arrangements obtain, the bending moments may be neglected. Such symmetrical arrangements can however exist only for centre columns and criteria are given on page 299 for what may be regarded as symmetrical conditions. End columns, however, are subject to bending moments in all circumstances, and in the Memorandum a method is set out by which the negative bending moments in "external columns" may be computed. This method applies to columns on which the end of a beam is supported but over which the beam is not continuous. There may actually obtain such conditions that an internal column must be calculated as for an "external" and, conversely, an external column as though it were "internal." As illustrations the cases may be cited, first, of an interior column of a building on which the end of a beam is supported, the beam not being continuous; and, secondly, of a column in an external wall over which a beam, carrying the floors

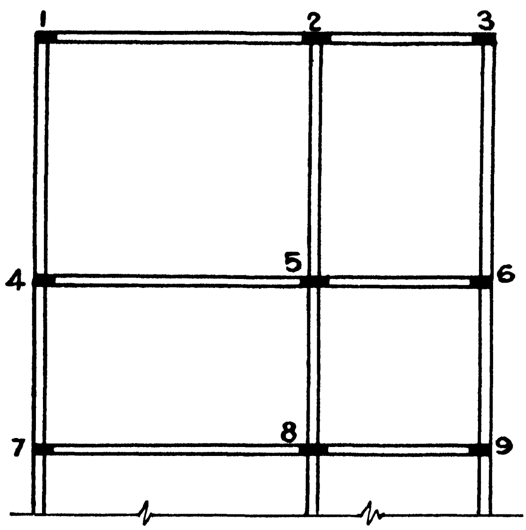


Fig. 312

in addition to the wall, may be continuous. To avoid any ambiguity in referring to such columns it is suggested that any column of a frame over which a beam does not continue in the same plane or approximately so should be called an " end " column, and that any column over which a beam forming part of the frame does continue should be called a " centre " column. Quite frequently frames are arranged at right angles or approximately at right angles to each other in the same structural system, and in such cases it is quite possible for a column to be an end column for one frame and a centre column for the other.

In Fig. 312 column 1 is an end column for each of frames 3–2–1 and 7–4–1; column 4 an end column for 6–5–4, but a centre column for frame 1–4–7; and column 5 is a centre column for each of frames 2–5–8 and 6–5–4.

The effects on a column of bending moments induced by beams not in the same line must be considered together. The Memorandum gives approximate formulæ only for the bending moments in end columns, namely:

$$M_u = M_e\left(\frac{K_u}{K_l + K_m + \frac{1}{2}K_b}\right) \qquad (42)$$

$$M_l = M_e\left(\frac{K_l}{K_l + K_u + \frac{1}{2}K_b}\right) \qquad (43)$$

For frames of one bay. (See Fig. 313.)

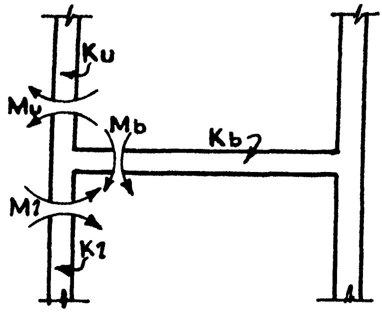


FIG. 313

And

$$\left.\begin{aligned} M_u &= M_e\left(\frac{K_u}{K_l + K_u + K_b}\right) \quad \ldots \ldots \ldots \ldots (44) \\ M_l &= M_e\left(\frac{K_l}{K_l + K_u + K_b}\right) \quad \ldots \ldots \ldots \ldots (45) \end{aligned}\right\} \text{For frames of two or more bays. (See Fig. 314.)}$$

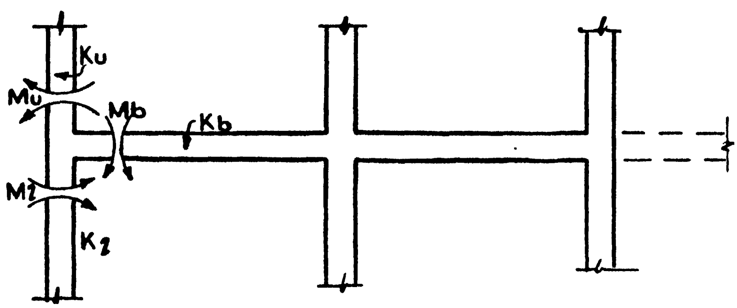


FIG. 314

With regard to the signs of these moments reference should be made to p. 300.

In all cases M_e is the calculated end bending moment, assuming the beam to be fully fixed. (See Fig. 315.) In using these formulæ the stiffnesses K are computed as explained previously, namely from :

$$K = \frac{I}{l};$$

and in order to simplify the application of the formulæ the values of M_e (the end

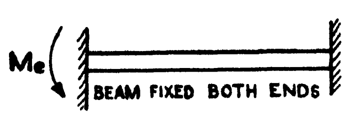


FIG. 315

bending moment for any span assuming the beam to be fully rigid at both ends) are set out in Table 55 for a wide range of loadings. For an explanation of the suffixes L and R see the Notes on pp. 284 and 300.

For any composite arrangement of loading which can be resolved into individual loadings given in Table 55, the ultimate value of M_e may be found by adding together

TABLE 55

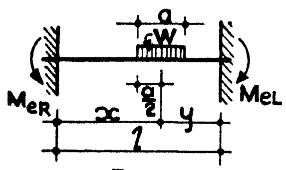
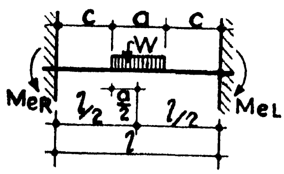
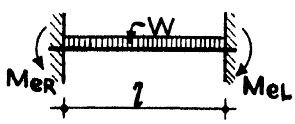
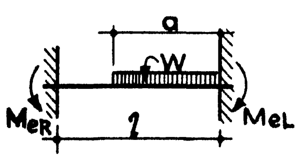
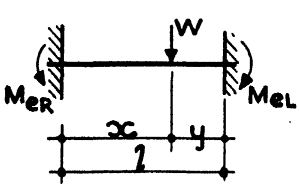
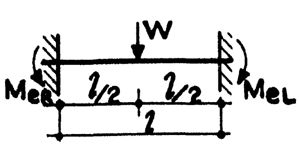
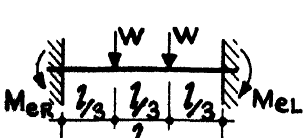
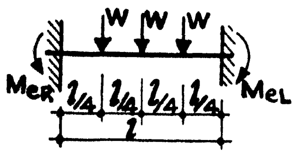
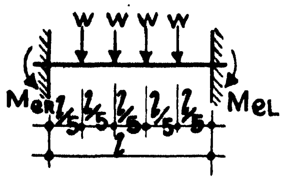
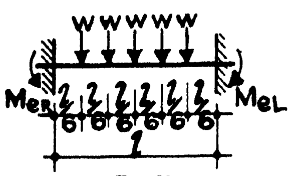
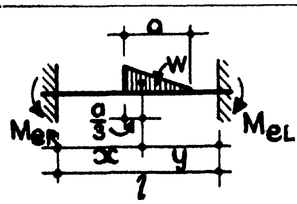
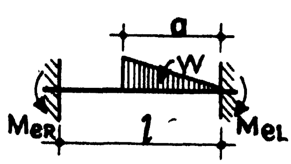
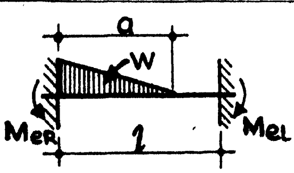
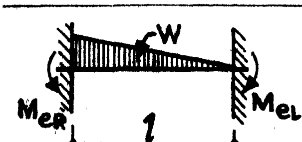
Loading Diagram	Frames of two or more Bays		Frames of one Bay
	$M_{e_R} =$	$M_{e_L} =$	$M_{e_L} = M_{e_R} =$
 Fig. 316	$-\frac{W}{l^2}\left[y^2x + \frac{a^2}{12}(x - 2y)\right]$	$-\frac{W}{l^2}\left[x^2y + \frac{a^2}{12}(y - 2x)\right]$	$-\frac{W}{2l}\left(xy - \frac{a^2}{12}\right)$
 Fig. 317	$-\frac{W}{24l}(3l^2 - a^2)$ for $\frac{c}{l} < 0{\cdot}1$: $-\frac{W}{12}(l + 2c)$ approx.	$-\frac{W}{24l}(3l^2 - a^2)$ for $\frac{c}{l} < 0{\cdot}1$: $-\frac{W}{12}(l + 2c)$ approx.	$-\frac{W}{24l}(3l^2 - a^2)$ for $\frac{c}{l} < 0{\cdot}1$: $-\frac{W}{12}(l + 2c)$ approx.
 Fig. 318	$-\frac{Wl}{12}$	$-\frac{Wl}{12}$	$-\frac{Wl}{12}$
 Fig. 319	$-\frac{Wa^2}{12l^2}(4l - 3a)$	$-\frac{Wa}{12l^2}(6l^2 - 8al + 3a^2)$	$-\frac{Wa}{12l}(3l - 2a)$
 Fig. 320	$-\frac{Wxy^2}{l^2}$	$-\frac{Wx^2y}{l^2}$	$-\frac{Wxy}{2l}$
 Fig. 321	$-\frac{Wl}{8}$	$-\frac{Wl}{8}$	$-\frac{Wl}{8}$
 Fig. 322	$-\frac{2Wl}{9}$	$-\frac{2Wl}{9}$	$-\frac{2Wl}{9}$

TABLE 55—*continued*

Loading Diagram	Frames of two or more Bays		Frames of one Bay
	$M_{e_R} =$	$M_{e_L} =$	$M_{e_L} = M_{e_R} =$
 FIG. 323	$-\frac{5Wl}{16}$	$-\frac{5Wl}{16}$	$-\frac{5Wl}{16}$
 FIG. 324	$-\frac{2Wl}{5}$	$-\frac{2Wl}{5}$	$-\frac{2Wl}{5}$
 FIG. 325	$-\frac{35Wl}{72}$	$-\frac{35Wl}{72}$	$-\frac{35Wl}{72}$
 FIG. 326	$-\frac{Wxy^2}{l^2} + \frac{Wa^2}{18l^2}\left(2y - x - \frac{2}{15}a\right)$	$-\frac{Wx^2y}{l^2} + \frac{Wa^2}{18l^2}\left(2x - y + \frac{2}{15}a\right)$	$-\frac{Wxy}{2l} + \frac{Wa^2}{36l}$
 FIG. 327	$-\frac{Wa^2}{10l^2}(5l - 4a)$	$-\frac{Wa}{15l^2}(10l^2 - 15la + 6a^2)$	$-\frac{Wa}{12l}(4l - 3a)$
 FIG. 328	$-\frac{Wa}{30l^2}(10l^2 - 10al + 3a^2)$	$-\frac{Wa^2}{30l^2}(5l - 3a)$	$-\frac{Wa}{12l}(2l - a)$
 FIG. 329	$-\frac{Wl}{10}$	$-\frac{Wl}{15}$	$-\frac{Wl}{12}$

the individual values. Fig. 330 shows an example of such a loading diagram, for a

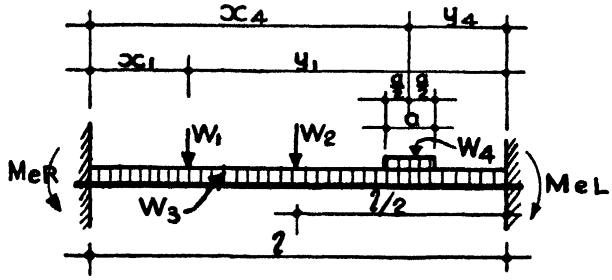


FIG. 330

span of a frame of more than one bay. From Table 55 we get:

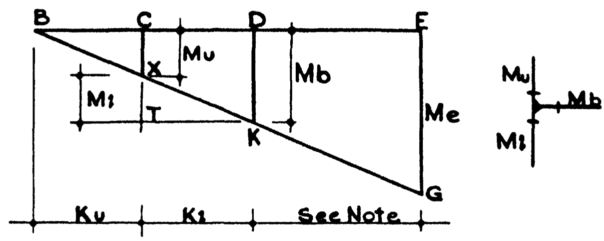
$$M_{e_R} = -\frac{W_1x_1y_1^2}{l^2} - \frac{W_2l}{8} - \frac{W_3l}{12} - \frac{W_4}{l^2}\left[y^2{}_4x_4 + \frac{a^2}{12}(x_4 - 2y_4)\right]$$

and

$$M_{e_L} = -\frac{W_1x_1^2y_1}{l^2} - \frac{W_2l}{8} - \frac{W_3l}{12} - \frac{W_4}{l^2}\left[x^2{}_4y_4 + \frac{a^2}{12}(y_4 - 2x_4)\right]$$

See Figs. 320, 321, 318 and 316

Fig. 331 shows a simple graphical method devised by the authors for determining M_b, M_u, and M_l:



For frames of two or more bays:

$$DE = K_b.$$

For frames of only one bay:

$$DE = \frac{K_b}{2}.$$

FIG. 331.

For a frame of two or more bays, draw a horizontal line BE = BC + CD + DE = $K_u + K_l + K_b$ and vertical lines through B, C, D, and E. Make EG = M_e. The sloping line BG determines the values CX = M_u and DK = M_b on the vertical lines through C and D respectively. Now draw a horizontal line through K. This fixes the value XT = M_l on the vertical line through C. For a frame of only one bay an exactly similar construction is used, but DE is made to represent only one-half of K_b.

As seen in Table 55, the value of M_e sometimes varies for frames of one bay only from that for frames of several bays. The reason will be seen from the following considerations.

In a frame of only one bay, the horizontal force must be equal in both columns forming the struts of the frame, in order to satisfy the condition of equilibrium that the sum of horizontal forces should be zero.

If the horizontal forces are equal and if the point of contraflexure is fixed (which is the assumption on which the formulæ in the memorandum are based), the bending moments on both ends of the beam must be equal, even if the load is asymmetrical. This corresponds to the exact theory of a two-hinged frame.

An ordinary beam, fixed rigidly at both ends, would show different negative moments at the ends. For the purpose of calculating M_e the average of both is to be taken when a frame of one bay is considered. If a frame of two or more bays is under consideration, then the equilibrium of forces allows different horizontal forces to act on the columns, as only the sum of all of them must be equal to nought, and in this case the actual fixed moment on the point which is considered is to be taken into account.

The negative bending moment at the end of the beam is :

$M_b = - M_l + M_u$ (according to the convention of signs M_u itself has a negative value, therefore $M_b = M_l + M_u$ numerically),

for which adequate top reinforcement has to be provided.

This moment affects all other bending moments in the beam, although the effect on the more remote spans is rather small. For the span adjoining the column concerned and the one next to it, however, it might be found advantageous to take the variation—which, in most cases, is a reduction—into account.

Reasonable approximations to the bending moment diagrams when the end moments are superimposed on those of the continuous beam on free supports, are shown in Figs. 332 to 335.

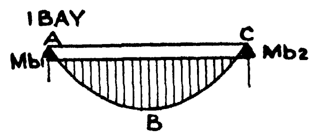


FIG. 332

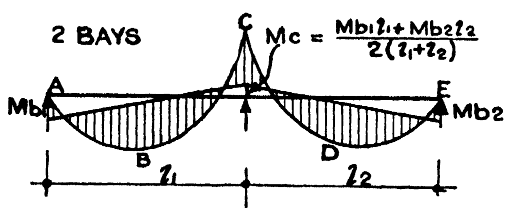


FIG. 333

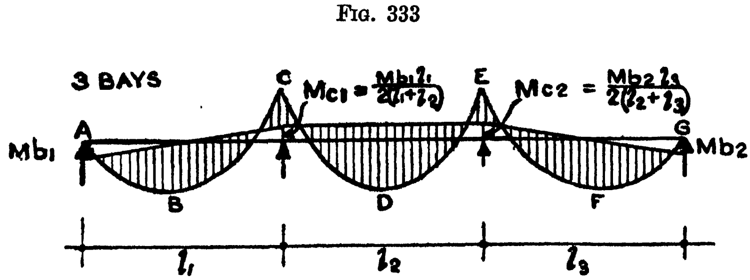


FIG. 334

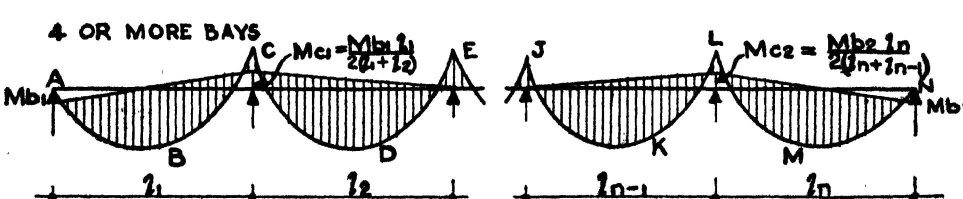


FIG. 335

The calculations for the end moments given here are to be recommended only where there are either no centre columns (one bay only) or where centre columns can be assumed to take no bending moments. For criteria, see p. 299 ; and for the calculation of frames in which the centre columns would take bending moments, see p. 299.

Example : (1) *End Columns.*—The properties of, and the loads on the frame chosen as an example are shown in Fig. 336 and the joint A is to be investigated.

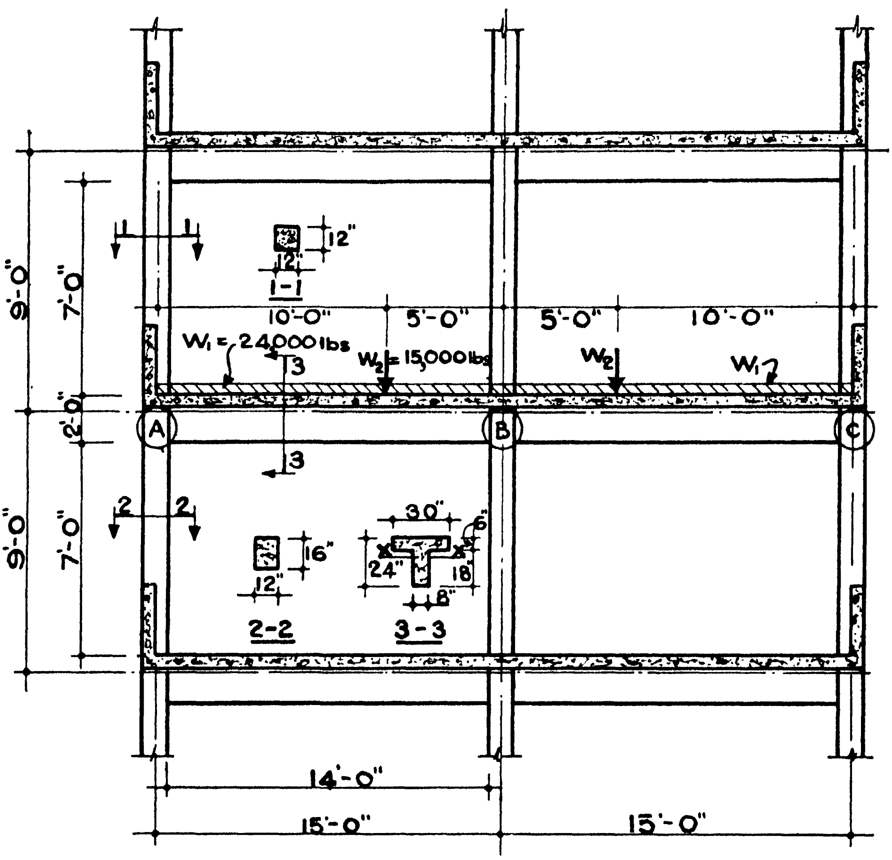


Fig. 336

First the stiffnesses of the three members (upper column, lower column and beam), connected at point A are computed.

Upper Column.—$I_u = \dfrac{12^4}{12} = 1728 \text{ in.}^4$

$l_u = 7$ ft. 0 in. $= 84$ in. (between beams).

$$K_u = \frac{1728}{84} = 20{\cdot}6 \text{ in.}^3$$

Lower Column.—$I_l = \dfrac{16 \times 12^3}{12} = 2304 \text{ in.}^4$

$l_l = 84$ in., as for upper column.

$$K_l = \frac{2304}{84} = 27{\cdot}4 \text{ in.}^3$$

Beam.—The moment of inertia of a T-section can best be found from Formula (31). The section is composed of two rectangles 18 in. × 8 in. and 30 in. × 6 in. respectively. The distance between the centres of gravity of these two rectangles equals half the total depth of the beam, i.e. $\frac{24}{2} = 12$ in.

$$I_{x_1} = \frac{8 \times 18^3}{12} = 3888 \text{ in.}^4 \quad I_{x_2} = \frac{30 \times 6^3}{12} = 540 \text{ in.}^4$$

$$A_1 = 8 \times 18 = 144 \text{ in.}^2 \quad A_2 = 6 \times 30 = 180 \text{ in.}^2$$

$$I_b = 3888 + 540 + \frac{144 \times 180 \times 12^2}{144 + 180} = 15{,}948 \text{ in.}^4$$

$$l_b = 14 \text{ ft. } 0 \text{ in.} = 168 \text{ in.}$$

$$K_b = \frac{15{,}948}{168} = 95 \text{ in.}^3$$

The value M_e must now be found, i.e. the beam moment at A if the beam were rigidly fixed at A. From Table 55, for W_1, i.e. uniformly distributed load over whole length:

$$M_e = -\frac{W_1 \times l}{12} \text{ (Fig. 318)};$$

for W_2, i.e. point load at x from A:

$$M_e = -\frac{W_2\, x\, y^2}{l^2} \text{ (Fig. 320).}$$

Therefore:

$$M_e = -\frac{24{,}000 \times 15{\cdot}0}{12} - \frac{15{,}000 \times 10 \times 5^2}{15^2}$$
$$= -30{,}000 - 16{,}700 = -46{,}700 \text{ lb.-ft.}$$

Thus, from Formulæ (44 and 45):

$$M_u = -46{,}700\left(\frac{20{\cdot}6}{20{\cdot}6 + 27{\cdot}4 + 95{\cdot}0}\right) = -6750 \text{ lb.-ft.}$$

$$M_l = +46{,}700\left(\frac{27{\cdot}4}{20{\cdot}6 + 27{\cdot}4 + 95{\cdot}0}\right) = +8950 \text{ lb.-ft.},$$

and from Formula (46):

$$M_b = M_u - M_l = -(6750 + 8950) = -15{,}700 \text{ lb.-ft.}$$

These three values may also be determined by the graphical method described on p. 267, and the actual figure for this particular example is given below as Fig. 337.

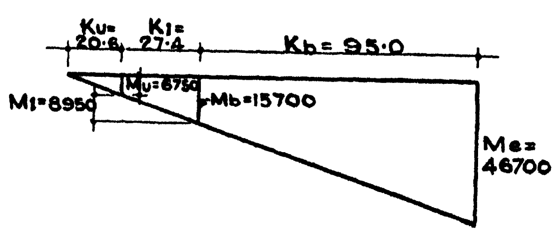


FIG. 337

The investigation of point A is now complete and the influence of the beam moment at A upon all the remaining bending moments can now be determined. We first calculate the support moment at B of a continuous beam ABC. An investigation of continuous beams is given on pp. 272 to 298 and the formula used here may be accepted from that investigation. Thus :

$$M^1_B = -\frac{24{,}000 \times 15}{8} - 2 \times 15{,}000 \times \frac{5 \times 10 \times (10 + 15)}{4 \times 15 \times 15}$$
$$= -45{,}000 - 41{,}670 = -86{,}670 \text{ lb.-ft.}$$

According to Fig. 332 we may deduct from this a value :

$$M = \frac{M_{b_1} \times l_1 + M_{b_2} \times l_2}{2(l_1 + l_2)}$$

which in this particular case, with $M_{b_1} = M_{b_2}$ and $l_1 = l_2$, reduces to $\frac{1}{4}M_b$. Therefore :

$$M_B = -\left(86{,}670 - \frac{15{,}700}{2}\right) = -78{,}820 \text{ lb.-ft.}$$

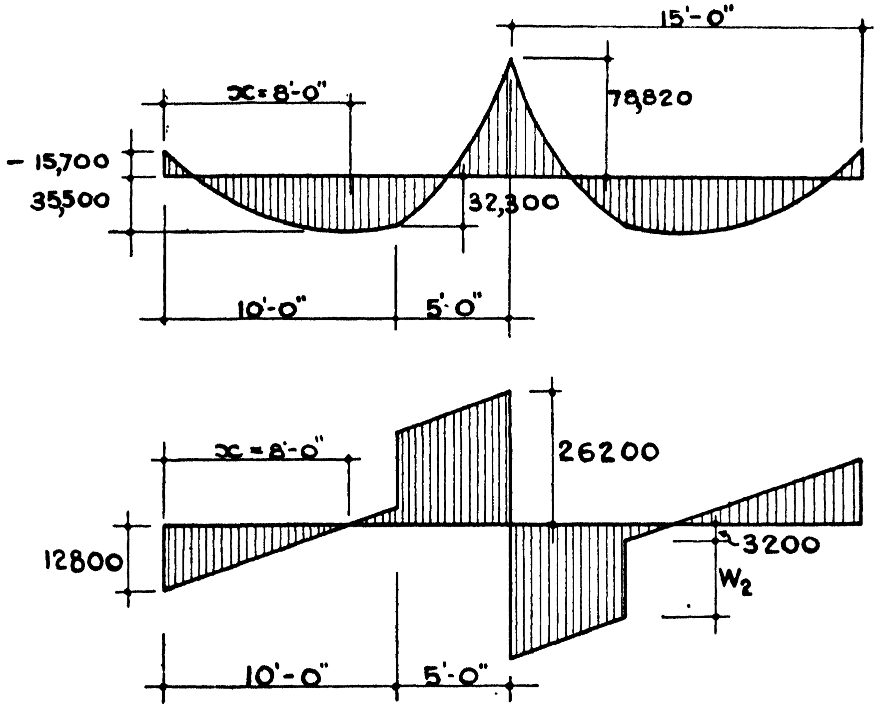


FIG. 338

Reactions at A and C, for simply supported span :

$$R^1_A = R^1_C = \frac{24{,}000}{2} + \frac{15{,}000 \times 5{\cdot}0}{15{\cdot}0} = 17{,}000 \text{ lb.}$$

Influence of end moments :

$$-\frac{78{,}820 - 15{,}700}{15{\cdot}0} = -4{,}200 \text{ lb.}$$

$$R_A = R_C = 17{,}000 - 4{,}200 = 12{,}800 \text{ lb.}$$

Shear forces at B equal reaction at A or C respectively less total load on span l_1 or l_2 respectively. Therefore :

$$S_{Bl} = -S_{Br} = +12{,}800 - 24{,}000 - 15{,}000 = -26{,}200 \text{ lb.}$$

Bending moment under point load:

$$M = -15{,}700 + (12{,}800 \times 10{\cdot}0) - \left(\frac{24{,}000}{15{\cdot}0} \times \frac{10{\cdot}0^2}{2}\right)$$

$$= -15{,}700 + 128{,}000 - 80{,}000 = +32{,}300 \text{ lb.-ft.}$$

Max. span moment occurs at x where:

$$x = \frac{12{,}800 \times 15{\cdot}0}{24{,}000} = 8{\cdot}0 \text{ ft. from A.}$$

$$M \text{ (span)} = -15{,}700 + \left(12{,}800 \times \frac{8{\cdot}0}{2}\right) = 35{,}500 \text{ lb.-ft.}$$

The bending and shear diagrams are shown in Fig. 338.

Continuity.—We revert now to continuous beams and to beams rigidly fixed to centre columns. Actually, continuous beams can be regarded as special cases of frames in which the stiffnesses of the columns are so small that they can be neglected. Continuous beams are the simpler in application, and therefore are considered first.

The Memorandum suggests that continuous beams should be calculated exactly; formulæ which simplify the calculations are given, but are to be applied only if the spans are approximately equal and carry a uniformly distributed load. Spans may be considered to be equal for this purpose if the difference in their lengths is not more than 15 per cent. of the length of the longer.

The Memorandum gives coefficients by which the bending moments for a beam on two supports should be multiplied in order to arrive at the span and support moments of continuous beams.

These coefficients and their appropriate moments are:

TABLE 56

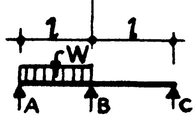
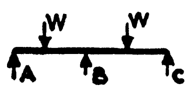
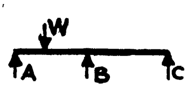
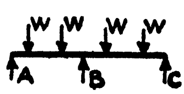
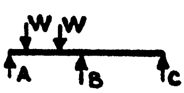
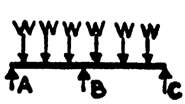
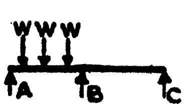
	Near Middle of End Span	At Support next to End Support	At Middle of Interior Spans	At Other Supports
Coefficient . . .	0·8	− 0·8	0·667	− 0·667
Equally distributed load	$\frac{wl^2}{10}$	$-\frac{wl^2}{10}$	$\frac{wl^2}{12}$	$-\frac{wl^2}{12}$
Point load in centre of span.	$\frac{Wl}{5}$	$-\frac{Wl}{5}$	$\frac{Wl}{6}$	$-\frac{Wl}{6}$

The formulæ are given for uniformly distributed loads and for a load in the centre, although, strictly speaking, the application is limited to uniformly distributed loads by the wording of the Memorandum. There is no distinction made between dead and live load, and where it is a question of economy in the design, the application of more exact formulæ is desirable.

The adjoining tables, 57 to 60 inclusive, give the support and panel bending moments, calculated exactly, for a greater number of cases (two, three, four, and five equal panels for different loadings, and also two unequal panels with regular loads), and it is suggested that the values given in these tables be used in preference to those of the Memorandum formulæ.

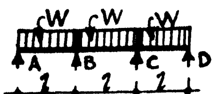
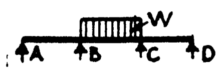
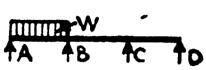
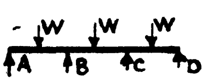
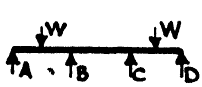
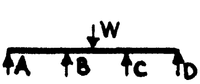
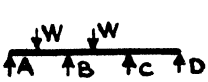
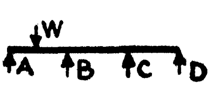
Tables 61 to 63 give the coefficients for bending moments and shear forces for certain continuous beams of two, three, and four spans when all the spans are not of the same length, but when (except in the case of a two-span beam) the arrangement is symmetrical. There will be many cases, however, which will comprise panels of different lengths or irregular loads, so that neither the L.C.C. formulæ nor the tables can be applied.

TABLE 57 (see notes below)

Diagram	Span Bending Moment A—B and B—C		Support Bending Moment M_B	Shear Forces and Reactions: A	$\left.\begin{matrix}B_L\\B_R\end{matrix}\right\}$	B	C
	α			β			
	0·070	0·070	– *0·125*	0·375	– 0·625 0·625	*1·250*	0·375
	*0·096*	– 0·025	– 0·063	*0·437*	– 0·563 0·063	0·625	– 0·063
	0·156	0·156	– *0·188*	0·312	– 0·688 0·688	*1·376*	0·312
	0·203	– 0·047	– 0·094	*0·406*	– 0·594 0·094	0·688	– 0·094
	0·222	0·222	– *0·333*	0·667	– 1·334 1·334	*2·667*	– 0·667
	*0·278*	– 0·056	– 0·167	*0·833*	– 1·167 0·167	1·334	– 0·167
	0·266	0·266	– *0·469*	1·042	– 1·958 1·958	*3·916*	1·042
	*0·383*	– 0·117	– 0·234	*1·266*	– 1·734 0·234	1·968	– 0·234

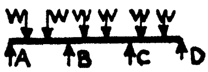
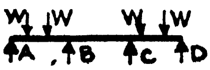
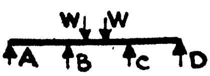
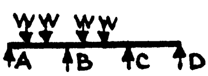
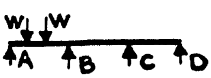
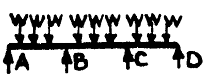
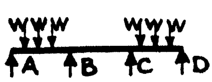
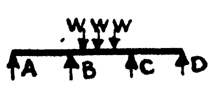
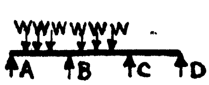
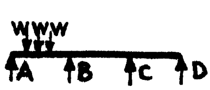
$M = \alpha \times W \times l$.
Reactions and shear forces $= \beta \times W$.
W equals *one* point load or distributed load for *one* span, as the case may be.
Spans are equal.

TABLE 58

Diagram	Span Bending Moment		Support Bending Moment		Shear Forces and Reactions					
	AB	BC	M_B	M_C	A	B_L / B_R	B	C_L / C_R	C	D
	α				β					
	0·080	0·025	− 0·100	−0·100	0·400	− 0·600 0·500	1·100	− 0·500 0·600	1·100	0·400
	*0·101*	− 0·050	− 0·050	− 0·050	*0·450*	− 0·550 0·000	0·550	0·000 0·550	0·550	0·450
	− 0·025	*0·075*	− 0·050	− 0·050	− 0·050	− 0·050 0·500	0·550	− 0·500 0·050	0·550	− 0·050
	—	—	*− 0·117*	− 0·033	0·383	− 0·617 0·583	*1·200*	− 0·417 0·033	0·450	− 0·033
	—	—	− 0·067	0·017	0·433	− 0·567 0·083	0·650	0·083 − 0·017	− 0·100	0·017
	0·175	0·100	− 0·150	− 0·150	0·350	− 0·650 0·500	1·150	− 0·500 0·650	1·150	0·350
	*0·213*	− 0·075	− 0·075	− 0·075	*0·425*	− 0·575 0·000	0·575	0·000 0·575	0·575	0·425
	− 0·038	*0·175*	− 0·075	− 0·075	− 0·075	− 0·075 0·500	0·575	− 0·500 0·075	0·575	− 0·075
	—	—	*− 0·175*	− 0·050	0·325	− 0·675 0·625	*1·300*	− 0·375 0·050	0·425	−0·050
	—	—	− 0·100	0·025	0·400	− 0·600 0·125	0·725	0·125 −0·025	− 0·150	0·025

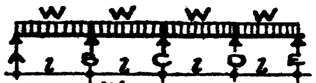
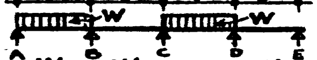
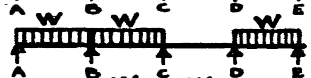
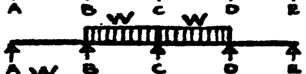
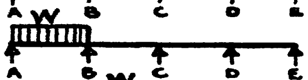
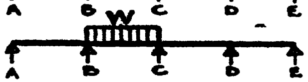
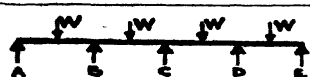
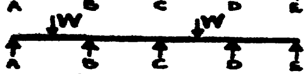
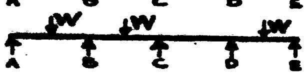
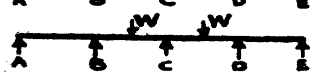
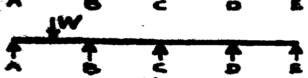
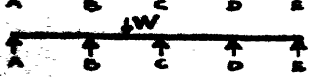
M = $\alpha \times W \times l$.
Reactions and shear forces = $\beta \times W$.
W equals *one* point load or distributed load for *one* span, as the case may be.
Spans are equal.

TABLE 58—*continued*

Diagram	Span Bending Moment		Support Bending Moment		Shear Forces and Reactions					
	AB	BC	M_B	M_C	A	B_L B_R	B	C_L C_R	C	D
	α				β					
	0·244	0·067	– 0·267	– 0·267	0·733	– 1·267 1·000	2·267	– 1·000 1·267	2·267	0·733
	*0·289*	– 0·133	– 0·133	–0·133	*0·866*	– 1·133 0·000	1·133	0·000 1·133	1·133	0·866
	– 0·044	*0·200*	– 0·133	– 0·133	– 0·133	– 0·133 1·000	1·133	– 1·000 0·133	1·133	– 0·133
	—	—	– *0·311*	– 0·089	0·689	– 1·311 1·222	*2·533*	– 0·778 0·089	0·867	– 0·089
	—	—	0·178	0·044	0·822	– 1·178 0·222	1·400	0·222 – 0·044	– 0·266	0·044
	0·313	0·125	– 0·375	– 0·375	1·125	– 1·875 1·500	3·375	– 1·500 1·875	3·375	1·125
	*0·406*	– 0·188	– 0·188	– 0·188	*1·313*	– 1·688 0·000	1·688	0·000 1·688	1·688	1·313
	– 0·094	*0·313*	– 0·188	– 0·188	– 0·188	– 0·188 1·500	1·688	– 1·500 0·188	1·688	– 0·188
	—	—	– *0·437*	– 0·125	1·063	– 1·938 1·812	*3·750*	– 1·188 0·125	1·313	– 0·125
	—	—	– 0·250	0·062	1·250	– 1·750 0·312	2·062	0·312 – 0·062	– 0·374	0·062

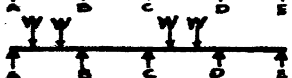
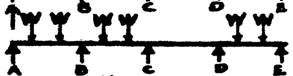
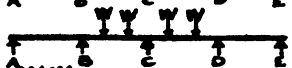
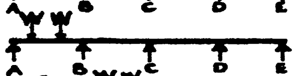
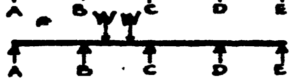
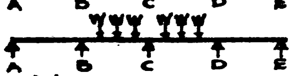
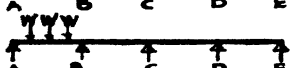
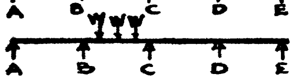
$M = \alpha \times W \times l$.
Reactions and shear forces = $\beta \times W$.
W equals *one* point load or distributed load for *one* span, as the case may be.
Spans are equal.

TABLE 59

Diagram	Span Bending Moment				Support Bending Moment			Shear Forces and Reactions							
	AB	BC	CD	DE	M_B	M_C	M_D	A	B_L } B_R	B	C_L } C_R	C	D_L } D_R	D	E
	α				α			β							
	0·077	0·036	0·036	0·770	— 0·107	— 0·071	— 0·107	0·393	— 0·607 0·536	1·143	— 0·464 0·464	0·929	— 0·536 0·607	1·143	0·393
	*0·100*	— 0·045	*0·081*	— 0·023	— 0·054	— 0·036	— 0·054	*0·446*	— 0·554 0·018	0·572	0·018 0·482	0·464	— 0·518 0·054	0·572	— 0·054
	—	—	—	—	— *0·121*	— 0·018	— 0·058	0·380	— 0·620 0·603	*1·223*	— 0·397 — 0·040	0·357	— 0·040 0·558	0·598	0·442
	—	—	—	—	— 0·036	— *0·017*	— 0·036	— 0·036	— 0·036 0·429	0·465	— 0·571 0·571	*1·143*	— 0·429 0·036	0·465	— 0·036
	—	—	—	—	— 0·067	0·018	— 0·004	0·433	— 0·567 0·085	0·652	0·083 — 0·022	— 0·107	— 0·022 0·004	0·027	— 0·004
	—	—	—	—	— 0·049	— 0·054	0·013	— 0·049	— 0·049 0·496	0·545	— 0·504 0·067	0·571	0·067 — 0·013	— 0·080	0·013
	0·169	0·116	0·116	0·169	— 0·161	— 0·107	— 0·161	0·339	— 0·661 0·553	1·214	— 0·446 0·446	0·892	— 0·553 0·661	1·214	0·339
	*0·210*	— 0·067	*0·183*	— 0·040	— 0·080	— 0·054	— 0·080	*0·420*	— 0·580 0·027	0·607	0·027 0·473	0·446	— 0·527 0·080	0·607	— 0·080
	—	—	—	—	— *0·181*	— 0·027	— 0·087	0·319	— 0·681 0·654	*1·335*	— 0·346 — 0·060	0·286	— 0·060 0·587	0·647	0·413
	—	—	—	—	— 0·054	— *0·161*	— 0·054	— 0·054	— 0·054 0·393	0·446	— 0·607 0·607	*1·214*	— 0·393 0·054	0·446	— 0·054
	—	—	—	—	— 0·100	0·027	— 0·007	0·400	— 0·600 0·127	0·728	0·127 — 0·033	— 0·161	— 0·033 0·007	0·040	— 0·007
	—	—	—	—	— 0·074	— 0·080	0·020	— 0·074	— 0·074 0·493	0·567	— 0·507 0·100	0·607	0·100 — 0·020	— 0·121	0·020

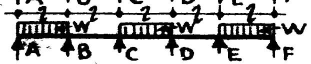
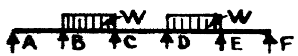
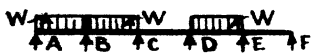
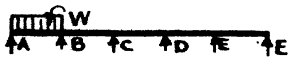
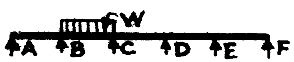
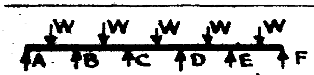
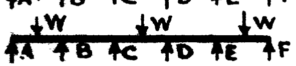
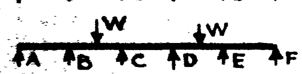
$M = \alpha \times W \times l$. Reactions and shear forces $= \beta \times W$. W equals *one* point load or distributed load for *one* span, as the case may be. Spans are equal.

TABLE 59—*continued*

Diagram	Span Bending Moment				Support Bending Moment			Shear Forces and Reactions							
	AB	BC	CD	DE	M_B	M_C	M_D	A	B_L / B_R	B	C_L / C_R	C	D_L / D_R	D	E
	α				α			β							
	0·238	0·111	0·111	0·238	−0·286	−0·191	−0·286	0·714	−1·286 1·095	2·381	−0·905 0·905	1·810	−1·095 1·286	2·381	0·714
	*0·286*	−0·111	*0·222*	−0·048	−0·143	−0·095	−0·143	*0·857*	−1·143 0·048	1·190	−0·048 0·952	0·904	−1·048 0·143	1·191	−0·143
	—	—	—	—	*−0·321*	−0·048	−0·155	0·679	−1·321 1·274	*2·595*	−0·726 −0·107	0·619	−0·107 1·155	1·262	0·845
	—	—	—	—	−0·095	*−0·286*	−0·095	−0·095	−0·095 0·810	0·905	−1·190 1·190	*2·381*	−0·810 0·095	0·905	−0·095
	—	—	—	—	−0·178	0·048	−0·012	0·821	−1·178 0·226	1·405	0·226 −0·060	−0·286	−0·060 0·012	0·071	−0·012
	—	—	—	—	−0·131	0·143	0·036	−0·131	−0·131 0·988	1·119	−1·012 0·178	1·190	0·178 −0·036	−0·214	0·036
	0·299	0·165	0·165	0·299	−0·402	−0·268	−0·402	1·098	−1·902 1·634	3·536	−1·366 1·366	2·732	−1·634 1·902	3·536	1·098
	*0·400*	−0·167	*0·333*	−0·101	−0·201	−0·134	−0·201	*1·299*	−1·701 0·067	1·768	0·067 1·433	1·366	−1·567 0·201	1·768	−0·201
	—	—	—	—	*−0·542*	−0·067	−0·218	1·048	−1·952 1·885	*3·837*	−1·115 −0·151	0·964	−0·151 1·718	1·869	1·282
	—	—	—	—	−0·134	*−0·402*	−0·134	−0·134	−0·134 1·232	1·366	−1·768 1·768	*3·536*	−1·232 0·134	1·368	−0·134
	—	—	—	—	−0·251	0·067	−0·017	1·249	−1·751 0·318	2·069	0·318 −0·084	−0·402	−0·084 0·017	0·101	−0·017
	—	—	—	—	−0·184	−0·201	0·050	−0·184	−0·184 1·483	1·667	−1·517 0·251	1·768	0·251 −0·050	−0·301	0·050

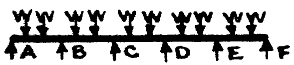
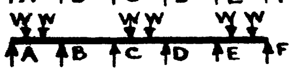
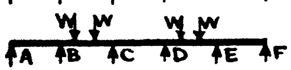
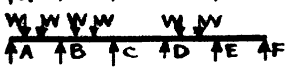
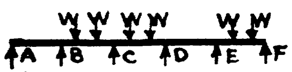
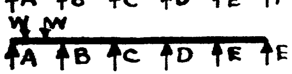
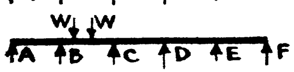
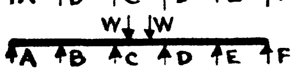
$M = \alpha \times W \times l$. Reactions and shear forces $= \beta \times W$. W equals *one* point load or distributed load for *one* span, as the case may be. Spans are equal.

Table 60.

Diagram	Span Bending Moment			Support Bending Moment				Shear Forces and Reactions									
	AB	BC	CD	M_B	M_C	M_D	M_E	A	B_L B_R	B	C_L C_R	C	D_L D_R	D	E_L E_R	E	F
	α			α				β									
	0·0781	0·0331	0·0462	— 0·105	— 0·079	— 0·079	— 0·105	0·395	— 0·606 0·526	1·132	— 0·474 0·500	0·974	— 0·500 0·474	0·974	— 0·526 0·606	1·132	0·395
	*0·100*	— 0·0461	*0·0855*	— 0·033	— 0·040	— 0·040	— 0·053	*0·447*	— 0·553 0·013	0·566	0·013 0·500	0·487	— 0·500 — 0·013	0·487	— 0·013 0·553	0·566	0·447
	— 0·0263	*0·0787*	— 0·0395	— 0·053	— 0·040	— 0·040	— 0·053	— 0·053	— 0·053 0·513	0·566	— 0·487 0·000	0·487	0·000 0·487	0·487	— 0·513 0·053	0·566	— 0·053
	—	—	—	— *0·119*	— 0·022	— 0·044	— 0·051	0·380	— 0·620 0·598	*1·218*	— 0·402 — 0·023	0·379	— 0·023 0·493	0·516	— 0·507 0·052	0·559	— 0·052
	—	—	—	— 0·035	— *0·111*	— 0·020	— 0·057	— 0·035	— 0·035 0·424	0·458	— 0·576 0·591	*1·167*	— 0·409 — 0·037	0·372	— 0·037 0·557	0·594	0·443
	—	—	—	— 0·067	+ 0·018	— 0·005	0·001	0·433	— 0·567 0·085	0·652	0·085 — 0·023	— 0·108	— 0·023 0·006	0·029	0·006 — 0·001	— 0·007	0·001
	—	—	—	— 0·049	— 0·054	0·014	— 0·004	— 0·049	— 0·049 0·495	0·544	— 0·505 0·068	0·573	0·068 — 0·018	— 0·086	— 0·018 0·004	0·002	— 0·004
	—	—	—	0·013	— 0·053	— 0·053	0·013	0·013	0·013 — 0·066	— 0·079	— 0·066 0·500	0·566	— 0·500 0·066	0·566	0·066 — 0·013	— 0·079	0·013
	0·171	0·112	0·132	— 0·158	— 0·118	— 0·118	— 0·158	0·342	— 0·653 0·540	1·198	— 0·460 0·500	0·960	— 0·500 0·460	0·960	— 0·540 0·658	1·198	0·342
	*0·211*	— 0·069	*0·191*	— 0·079	— 0·059	— 0·059	— 0·079	*0·421*	— 0·579 0·020	0·599	0·020 0·500	0·480	— 0·550 — 0·020	0·480	— 0·020 0·579	0·598	0·421
	— 0·039	*0·181*	— 0·059	— 0·079	— 0·059	— 0·059	— 0·079	— 0·079	— 0·079 0·520	0·599	— 0·480 0·000	0·480	0·000 0·480	0·480	— 0·520 0·079	0·598	— 0·079
	—	—	—	— *0·179*	— 0·032	— 0·066	— 0·077	0·321	— 0·679 0·647	*1·326*	— 0·353 — 0·034	0·319	— 0·034 0·489	0·523	— 0·511 0·077	0·588	— 0·077

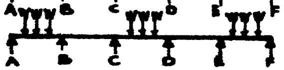
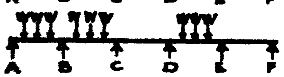
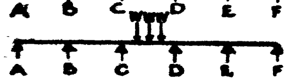
$M = \alpha \times W \times l$. Reactions and shear forces $= \beta \times W$. W equals *one* point load or distributed load for *one* span, as the case may be. Spans are equal.

TABLE 60.—*continued.*

Diagram	Span Bending Moment AB	BC	CD	Support Bending Moment M_B	M_C	M_D	M_E	Shear Forces and Reactions A	B_L B_R	B	C_L C_R	C	D_L D_R	D	E_L E_R	E	F
	α			α				β									
	—	—	—	− 0·052	−*0·167*	− 0·031	− 0·086	− 0·052	− 0·052 0·385	0·437	− 0·615 0·637	*1·251*	− 0·363 − 0·056	0·307	− 0·056 0·586	0·642	0·414
	—	—	—	− 0·100	0·027	− 0·007	0·002	0·400	− 0·600 0·127	0·727	0·127 − 0·034	− 0·161	− 0·034 0·009	0·043	0·009 − 0·002	− 0·011	0·002
	—	—	—	− 0·073	− 0·081	0·022	− 0·005	− 0·073	− 0·073 0·493	0·566	− 0·507 0·102	0·609	0·102 − 0·027	− 0·129	− 0·027 0·005	0·032	− 0·005
	—	—	—	0·020	− 0·079	− 0·079	− 0·020	0·020	0·020 − 0·099	− 0·118	− 0·099 0·500	0·599	− 0·500 0·099	0·599	0·099 − 0·020	− 0·118	0·020
	0·240	0·100	0·122	− 0·281	− 0·211	− 0·211	− 0·281	0·719	− 1·281 1·070	2·351	− 0·930 1·000	1·930	− 1·000 0·930	1·930	− 1·070 1·281	2·351	0·719
	*0·287*	− 0·117	*0·228*	− 0·140	− 0·105	− 0·105	− 0·140	0·860	− 1·140 0·035	1·175	0·035 1·000	0·965	− 1·000 − 0·035	0·965	− 0·035 1·140	1·175	0·860
	− 0·047	*0·216*	− 0·105	− 0·105	− 0·105	− 0·105	− 0·140	− 0·140	− 0·140 1·035	1·175	− 0·965 0·000	0·965	0·000 0·965	0·965	− 1·035 0·140	1·175	− 0·140
	—	—	—	− *0·319*	− 0·057	− 0·118	− 0·137	0·681	− 1·319 1·262	*2·581*	− 0·738 − 0·061	0·677	− 0·061 0·981	1·042	− 1·019 0·137	1·156	− 0·137
	—	—	—	− 0·093	− *0·297*	− 0·054	− 0·153	− 0·093	− 0·093 0·796	0·889	− 1·204 1·243	*2·447*	− 0·757 − 0·099	0·658	− 0·099 1·153	1·252	0·847
	—	—	—	− 0·179	0·048	− 0·013	0·003	0·821	− 1·179 0·227	1·405	0·227 − 0·061	− 0·287	− 0·061 0·016	0·077	0·016 − 0·003	− 0·019	0·003
	—	—	—	− 0·131	− 0·144	0·038	− 0·010	− 0·131	− 0·131 0·987	1·118	− 1·013 0·182	1·195	0·182 − 0·048	− 0·230	−0·048 0·010	0·058	− 0·010
	—	—	—	0·035	− 0·140	− 0·140	0·035	0·035	0·035 − 0·175	− 0·210	− 0·175 1·000	1·175	− 1·000 0·175	− 1·175	0·175 − 0·035	− 0·210	0·035

$M = \alpha \times W \times l$. Reactions and shear forces $= \beta \times W$. W equals *one* point load or distributed load for *one* span, as the case may be. Spans are equal.

TABLE 60.—*continued.*

Diagram	Span Bending Moment			Support Bending Moment				Shear Forces and Reactions									
	AB	BC	CD	M_B	M_C	M_D	M_E	A	B_L } B_R	B	C_L } C_R	C	D_L } D_R	D	E_L } E_R	E	F
	α			α				β									
	0·302	0·155	0·204	− 0·395	− 0·296	− 0·296	− 0·395	1·105	− 1·895 1·599	3·494	− 1·401 1·500	2·901	− 1·500 1·401	2·901	− 1·599 1·895	3·494	1·105
	*0·401*	− 0·173	*0·352*	− 0·198	− 0·148	− 0·148	− 0·198	*1·302*	− 1·697 0·050	1·747	0·050 1·500	1·450	− 1·500 − 0·050	1·450	− 0·050 1·697	1·747	1·302
	− 0·099	*0·327*	− 0·148	− 0·198	− 0·148	− 0·148	− 0·198	− 0·197	− 0·197 1·550	1·747	− 1·450 0·000	1·450	0·000 1·450	1·450	− 1·550 0·197	1·747	− 0·197
	—	—	—	*− 0·449*	− 0·081	− 0·166	− 0·193	1·051	− 1·949 1·867	*3·816*	− 1·133 − 0·085	1·048	− 0·085 1·473	1·558	− 1·527 0·193	1·720	− 0·193
	—	—	—	− 0·130	*− 0·417*	− 0·076	− 0·215	− 0·130	− 0·130 1·213	1·343	− 1·787 1·841	*3·628*	− 1·159 − 0·139	1·020	− 0·139 1·715	1·854	1·285
	—	—	—	− 0·251	0·067	− 0·018	0·004	1·249	− 1·751 0·318	2·069	0·318 − 0·085	− 0·403	− 0·085 0·022	0·107	0·022 − 0·004	− 0·027	0·004
	—	—	—	− 0·184	− 0·202	0·054	− 0·013	− 0·184	− 0·184 1·482	1·666	− 1·518 0·258	1·774	0·256 − 0·067	− 0·323	− 0·067 0·013	0·081	− 0·013
	—	—	—	0·049	− 0·197	0·197	0·049	0·049	0·049 − 0·247	− 0·296	− 0·247 1·500	1·747	− 1·500 0·247	1·747	0·247 − 0·049	− 0·296	0·049

M = α × W × *l*. Reactions and shear forces = β × W. W equals *one* point load or distributed load for *one* span, as the case may be. Spans are equal.

TABLE 61

Proportions of Span	Bending Moments							Reactions and Shear Forces									
	Span l_1		Span l_2			Support B		A			B_l		B_r		C_l		
	Dead $w_d l_1^2$	Live $w_l l_1^2$	Dead $w_d l_2^2$	+ Live $w_l l_2^2$	− Live $w_l l_2^2$	Dead $w_d l_2^2$	Live $w_l l_2^2$	Dead $w_d l_1$	+ Live $w_l l_1$	− Live $w_l l_1$	Dead $w_d l_1$	Live $w_l l_1$	Dead $w_d l_2$	Live $w_l l_2$	Dead $w_d l_2$	+ Live $w_l l_2$	− Live $w_l l_2$
1 : 1	0·070	0·096	0·070	0·096	—	0·125	0·125	0·375	0·437	− 0·063	0·625	0·625	0·625	0·625	0·375	0·437	− 0·063
1 : 1·05	0·068	0·097	0·072	0·095	—	0·120	0·120	0·368	0·439	− 0·071	0·632	0·632	0·620	0·620	0·380	0·437	− 0·056
1 : 1·1	0·065	0·097	0·074	0·094	—	0·115	0·115	0·361	0·440	− 0·079	0·639	0·639	0·615	0·615	0·385	0·435	− 0·049
1 : 1·2	0·060	0·098	0·077	0·903	—	0·108	0·108	0·345	0·443	− 0·098	0·655	0·655	0·608	0·608	0·392	0·432	− 0·039
1 : 1·3	0·053	0·099	0·079	0·092	—	0·103	0·103	0·326	0·446	− 0·119	0·674	0·674	0·603	0·603	0·397	0·429	− 0·032
1 : 1·4	0·047	0·100	0·080	0·091	—	0·099	0·099	0·305	0·448	− 0·143	0·695	0·695	0·599	0·599	0·401	0·427	− 0·027
1 : 1·5	0·040	0·101	0·081	0·090	—	0·097	0·097	0·281	0·450	− 0·169	0·719	0·719	0·597	0·597	0·403	0·425	− 0·022

TABLE 62

1 : 1·5 : 1	0·055	0·107	0·050	0·067	− 0·017	0·075	0·082	0·332	0·462	− 0·130	0·668	0·685	0·500	0·532	—	—	—
1 : 1·4 : 1	0·061	0·106	0·048	0·069	− 0·021	0·077	0·085	0·349	0·460	− 0·111	0·651	0·668	0·500	0·538	—	—	—
1 : 1·3 : 1	0·066	0·105	0·045	0·070	− 0·025	0·080	0·090	0·365	0·458	− 0·093	0·635	0·652	0·500	0·545	—	—	—
1 : 1·2 : 1	0·072	0·104	0·040	0·071	− 0·031	0·085	0·096	0·378	0·454	− 0·077	0·622	0·639	0·500	0·554	—	—	—
1 : 1·1 : 1	0·076	0·103	0·034	0·073	− 0·039	0·091	0·105	0·390	0·453	− 0·063	0·610	0·627	0·500	0·567	—	—	—
1 : 1 : 1	0·080	0·101	0·025	0·075	− 0·050	0·100	0·117	0·400	0·450	− 0·050	0·600	0·617	0·500	0·583	—	—	—
1 : 0·95 : 1	0·082	0·101	0·019	0·076	− 0·057	0·106	0·124	0·404	0·448	− 0·044	0·596	0·612	0·500	0·594	—	—	—
1 : 0·90 : 1	0·083	0·100	0·011	0·077	− 0·066	0·114	0·134	0·408	0·447	− 0·039	0·592	0·608	0·500	0·606	—	—	—
1 : 0·85 : 1	0·085	0·099	0·002	0·078	− 0·076	0·123	0·145	0·411	0·445	− 0·034	0·589	0·605	0·500	0·621	—	—	—
1 : 0·80 : 1	0·086	0·098	0·009	0·080	− 0·089	0·134	0·160	0·414	0·443	− 0·029	0·586	0·602	0·500	0·640	—	—	—
1 : 0·75 : 1	0·087	0·097	0·024	0·081	− 0·105	0·149	0·177	0·416	0·441	− 0·025	0·584	0·600	0·500	0·662	—	—	—
1 : 0·70 : 1	0·087	0·096	0·042	0·082	− 0·124	0·167	0·199	0·418	0·439	− 0·021	0·582	0·598	0·500	0·689	—	—	—
1 : 0·60 : 1	0·088	0·094	0·097	0·086	− 0·183	0·222	0·264	0·420	0·434	− 0·014	0·580	0·595	0·500	0·767	—	—	—
1 : 0·50 : 1	0·088	0·092	0·196	0·089	− 0·286	0·321	0·379	0·420	0·429	− 0·009	0·580	0·595	0·500	0·900	—	—	—

TABLE 63

Proportions of Span	Bending Moments									Reactions and Shear Forces									
	Span l_1		Span l_2			Support B		Support C		A			B_l		B_r		C_l		
	Dead $w_d l_1^2$	Live $w_l l_1^2$	Dead $w_d l_2^2$	+ Live $w_l l_2^2$	− Live $w_l l_2^2$	Dead $w_d l_2^2$	Live $w_l l_2^2$	Dead $w_d l_2^2$	Live $w_l l_2^2$	Dead $w_d l_1$	+ Live $w_l l_1$	− Live $w_l l_1$	Dead $w_d l_1$	Live $w_l l_1$	Dead $w_d l_2$	Live $w_l l_2$	Dead $w_l l_2$	Li e $w_l l_2$
1 : 1·5 : 1·5 : 1	0·058	0·115	0·045	0·072	− 0·027	0·070	0·086	0·090	0·103	0·342	0·480	− 0·138	0·658	0·693	0·408	0·547	0·520	0·559
1 : 1·4 : 1·4 : 1	0·063	0·112	0·044	0·073	− 0·029	0·074	0·089	0·088	0·104	0·355	0·473	− 0·118	0·645	0·674	0·486	0·553	0·514	0·561
1 : 1·3 : 1·3 : 1	0·067	0·109	0·043	0·074	− 0·032	0·079	0·093	0·086	0·104	0·367	0·466	− 0·099	0·633	0·658	0·493	0·560	0·507	0·564
1 : 1·2 : 1·2 : 1	0·071	0·106	0·041	0·076	− 0·032	0·085	0·100	0·082	0·105	0·377	0·459	− 0·082	0·623	0·643	0·503	0·570	0·497	0·566
1 : 1·1 : 1·1 : 1	0·074	0·103	0·039	0·078	− 0·039	0·094	0·108	0·078	0·106	0·386	0·453	− 0·067	0·614	0·631	0·516	0·583	0·484	0·569
1 : 1 : 1 : 1	0·077	0·100	0·036	0·081	− 0·045	0·107	0·121	0·071	0·107	0·393	0·446	− 0·054	0·607	0·620	0·536	0·603	0·464	0·571
1 : 0·95 : 0·95 : 1	0·078	0·098	0·035	0·082	− 0·048	0·116	0·129	0·067	0·108	0·396	0·443	− 0·048	0·604	0·616	0·548	0·615	0·452	0·573
1 : 0·90 : 0·90 : 1	0·079	0·097	0·033	0·084	− 0·052	0·126	0·139	0·062	0·108	0·398	0·440	− 0·042	0·602	0·612	0·564	0·630	0·436	0·575
1 : 0·85 : 0·85 : 1	0·080	0·096	0·031	0·086	− 0·056	0·138	0·151	0·056	0·109	0·400	0·437	− 0·037	0·600	0·609	0·582	0·649	0·418	0·577
1 : 0·80 : 0·80 : 1	0·081	0·094	0·030	0·088	− 0·062	0·153	0·165	0·048	0·109	0·402	0·434	− 0·032	0·598	0·606	0·605	0·672	0·395	0·578
1 : 0·75 : 0·75 : 1	0·081	0·093	0·028	0·091	− 0·068	0·172	0·184	0·039	0·110	0·403	0·431	− 0·028	0·597	0·604	0·633	0·700	0·367	0·580
1 : 0·70 : 0·70 : 1	0·082	0·092	0·028	0·095	− 0·076	0·196	0·207	0·027	0·111	0·404	0·428	− 0·024	0·596	0·602	0·669	0·736	0·331	0·582
1 : 0·60 : 0·60 : 1	0·082	0·089	0·033	0·104	− 0·096	0·265	0·276	− 0·008	0·112	0·405	0·422	− 0·017	0·596	0·599	0·773	0·840	0·227	0·586
1 : 0·50 : 0·50 : 1	0·081	0·086	0·069	0·122	− 0·122	0·386	0·396	− 0·068	0·114	0·403	0·415	− 0·012	0·597	0·599	0·955	1·021	0·045	0·591

For such cases the continuous beam can be calculated by means of the equations of the Three Moments; but an alternative approximate method is suggested which, while saving a great deal of calculation, especially for beams of many spans, will give satisfactory results. The exact method is first described and later the suggested approximate method is given and comparisons made.

(1) *Equations of the Three Moments.*

If for a continuous beam on several supports, the supports are numbered, 0, 1, 2, 3, etc., from left to right, any one of these supports being called n, then the following equation exists for the moments at the supports $(n - 1)$, n, $(n + 1)$ (Fig. 339):

$$M_{(n-1)}\frac{l_n}{I_n} + 2M_n\left(\frac{l_n}{I_n} + \frac{l_{(n+1)}}{I_{(n+1)}}\right) + M_{(n+1)}\frac{l_{(n+1)}}{I_{(n+1)}} = 2M_{nL} \times \frac{l_n}{I_n} + 2M_{nR} \times \left(\frac{l_{(n+1)}}{I_{(n+1)}}\right) \quad . \quad . \quad (47)$$

And if the beam has the same moment of inertia throughout, this becomes:

$$M_{(n-1)}\,l_n + 2M_n\,(l_n + l_{(n+1)}) + M_{(n+1)}\,l_{(n+1)} = 2M_{nL}\,l_n + 2M_{nR}\,l_{(n+1)} \quad . \quad . \quad (48)$$

where M_{nL} and M_{nR} would be the bending moments on the left and right, respectively, of the support n, if the beam were absolutely rigidly fixed at n, but hinged at $(n - 1)$ and at $(n + 1)$ (see Fig. 340).

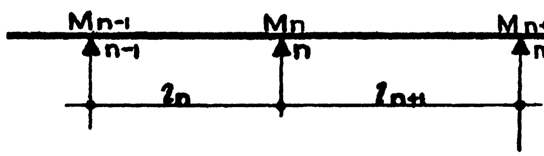


FIG. 339

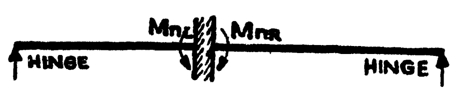


FIG. 340

This formula is known also in the form:

$$M_{(n-1)}\,l_n + 2M_n(l_n + l_{(n+1)}) + M_{(n+1)}\,l_{(n+1)} = 6R_D \quad . \quad . \quad . \quad (49)$$

where R_D is the sum of the reactions at n of the two adjoining spans if there were two simply supported beams carrying the free bending moment diagrams as loads (see Fig. 343, Table 64). The units of these reactions are products of a bending moment and a linear dimension.

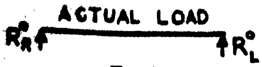
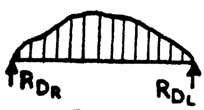
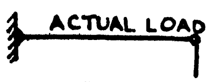
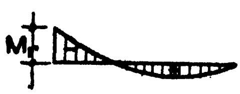
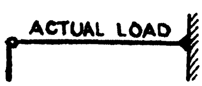
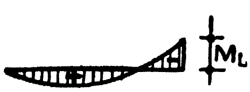
If the continuous beam has m supports, there are $m - 2$ equations, and as it is generally necessary to make separate calculations for dead and live loads, these calculations become very involved if there are more than four or at the most five, supports.

The values of R_{DL} and R_{DR} or of M_{nL} and M_{nR} may be obtained from Table 64, which gives bending moments and diagram reactions for several frequently prevailing loads. Either of the Formulæ 48 or 49 may be applied readily by this table.

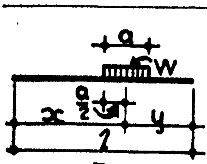
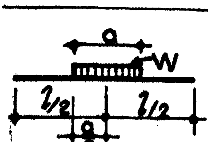
There will be as many equations as unknowns and the support moments can be calculated. Care has to be taken that the maxima and minima values are found by loading only certain panels.

For every load it is recommended that the bending moments and reactions at all supports be computed and tabulated as functions of that load.

TABLE 64

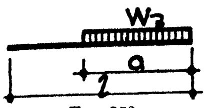
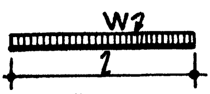
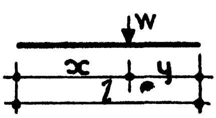
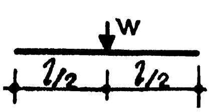
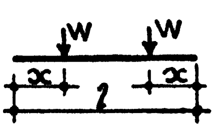
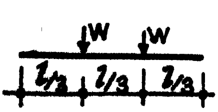
 FIG. 341	Span regarded as on two single supports, loaded *ab. lib.*, giving reactions R^0_R and R^0_L.
 FIG. 342	Bending Moment Diagram for the above beam.
 FIG. 343	B.M.—Diagram taken as loading diagram of simply-supported beam, giving reactions R_{D_R} and R_{D_L}.
 FIG. 344	Beam regarded as fixed at left-hand end, hinged at other end to find M_R.
 FIG. 345	B.M.—Diagram for the above beam, giving M_R.
 FIG. 346	Beam regarded as fixed at right-hand end, hinged at other end to find M_L.
 FIG. 347	B.M.—Diagram for the above beam, giving M_L.

NOTE.—The suffices R and L refer to the right- and left-hand sides of the columns, and *not* to the right- and left-hand ends of the beam.

Loading Diagram *	R^0_R, R^0_L	R_{D_R} R_{D_L}	M_R M_L
 FIG. 348	$R^0_R = \frac{Wy}{l}$ $R^0_L = \frac{Wx}{l}$	$R_{D_R} = -\frac{Wxy(l+y)}{6l} + \frac{Wa^2y}{24l}$ $R_{D_L} = -\frac{Wxy(l+x)}{6l} + \frac{Wa^2x}{24l}$	$M_R = -\frac{Wxy(l+y)}{2l^2} + \frac{Wa^2y}{8l^2}$ $M_L = -\frac{Wxy(l+x)}{2l^2} + \frac{Wa^2x}{8l^2}$
 FIG. 349	$R^0_R = \frac{W}{2}$ $R^0_L = \frac{W}{2}$	$R_{D_R} = R_{D_L} = -\frac{Wl^2}{16} + \frac{Wa^2}{48}$	$M_R = M_L = -\frac{3Wl}{16} + \frac{Wa^2}{16l}$

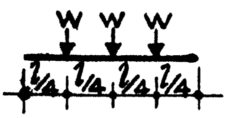
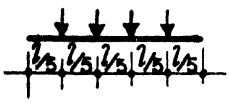
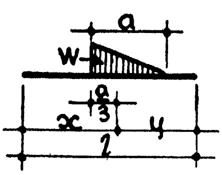
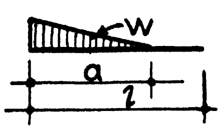
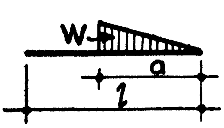
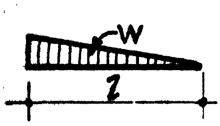
* W represents the total distributed or point load in each case.

TABLE 64—*continued*

Loading Diagram *	R^0_R, R^0_L	R_{D_R} R_{D_L}	M_R M_L
 FIG. 350	$R^0_R = \frac{Wa}{2l}$ $R^0_L = \frac{W(2l - a)}{2l}$	$R_{D_R} = -\frac{Wa(2l^2 - a^2)}{24l}$ $R_{D_L} = -\frac{Wa(2l - a)^2}{24l}$	$M_R = -\frac{Wa(2l^2 - a^2)}{8l^2}$ $M_L = -\frac{Wa(2l - a)^2}{8l^2}$
 FIG. 351	$R^0_R = \frac{W}{2}$ $R^0_L = \frac{W}{2}$	$R_{D_R} = R_{D_L} = -\frac{Wl^2}{24}$	$M_R = M_L = -\frac{Wl}{8}$
 FIG. 352	$R^0_R = \frac{Wy}{l}$ $R^0_L = \frac{Wx}{l}$	$R_{D_R} = -\frac{Wxy(l + y)}{6l}$ $R_{D_L} = -\frac{Wxy(l + x)}{6l}$	$M_R = -\frac{Wxy(l + y)}{2l^2}$ $M_L = -\frac{Wxy(l + y)}{2l^2}$
 FIG. 353	$R^0_R = \frac{W}{2}$ $R^0_L = \frac{W}{2}$	$R_{D_R} = R_{D_L} = -\frac{Wl^2}{16}$	$M_R = M_L = -\frac{3Wl}{16}$
 FIG. 354	$R^0_R = W$ $R^0_L = W$	$R_{D_R} = R_{D_L} = -\frac{Wx(l - x)}{2}$ if $x = \frac{1}{4}l$: $R_{D_R} = R_{D_L} = -\frac{3}{32} Wl^2$	$M_R = M_L = -\frac{3Wx(l - x)}{2l}$ if $x = \frac{1}{4}l$: $M_R = M_L = -\frac{9}{32} Wl$
 FIG. 355	$R^0_R = W$ $R^0_L = W$	$R_{D_R} = R_{D_L} = -\frac{Wl^2}{9}$	$M_R = M_L = -\frac{Wl}{3}$

* W represents the total distributed or point load in each case.

TABLE 64—*continued*

Loading Diagram *	R^0_R, R^0_L	R_{D_R} R_{D_L}	M_R M_L
 FIG. 356	$R^0_R = \frac{3W}{2}$ $R^0_L = \frac{3W}{2}$	$R_{D_R} = R_{D_L} = -\frac{5}{32} Wl^2$	$M_R = M_L = -\frac{15}{32} Wl$
 FIG. 357	$R^0_R = 2W$ $R^0_R = 2W$	$R_{D_R} = R_{D_L} = -\frac{Wl^2}{5}$	$M_R = M_L = -\frac{2}{5} Wl$
 FIG. 358	$R^0_R = \frac{Wy}{l}$ $R^0_L = \frac{Wx}{l}$	$R_{D_R} = -\frac{Wxy(l+y)}{6l} + \frac{Wa^2}{36l}(y - \frac{2}{45}a)$ $R_{D_L} = -\frac{Wxy(l+x)}{6l} + \frac{Wa^2}{36l}(x + \frac{2}{45}a)$	$M_R = -\frac{Wxy(l+y)}{2l^2} + \frac{Wa^2}{12l^2}(y - \frac{2}{45}a)$ $M_L = -\frac{Wxy(l+x)}{2l^2} + \frac{Wa^2}{12l^2}(x + \frac{2}{45}a)$
 FIG. 359	$R^0_R = \frac{2Wa}{3l}$ $R^0_L = \frac{W(3l - 2a)}{3l}$	$R_{D_R} = -\frac{Wa(l^2 - 0{\cdot}6a^2)}{9l}$ $R_{D_L} = -\frac{Wa}{9l}(2l^2 - 2{\cdot}25la + 0{\cdot}6a^2)$	$M_R = -\frac{Wa(l^2 - 0{\cdot}6a^2)}{3l^2}$ $M_L = -\frac{Wa}{3l^2}(2l^2 - 2{\cdot}25la + 0{\cdot}6a^2)$
 FIG. 360	$R^0_R = \frac{W(3l - a)}{3l}$ $R^0_L = \frac{Wa}{3l}$	$R_{D_R} = -\frac{Wa}{9l}(l^2 - 0{\cdot}75la + 0{\cdot}15a^2)$ $R_{D_L} = -\frac{Wa}{9l}(0{\cdot}5l^2 - 0{\cdot}15a^2)$	$M_R = -\frac{Wa}{3l^2}(l^2 - 0{\cdot}75la + 0{\cdot}15a^2)$ $M_L = -\frac{Wa}{3l^2}(0{\cdot}5l^2 - 0{\cdot}15a^2)$
 FIG. 361	$R^0_R = \frac{2W}{3}$ $R^0_L = \frac{W}{3}$	$R_{D_R} = -\frac{2}{45} Wl^2$ $R_{D_L} = -\frac{7}{180} Wl^2$	$M_R = -\frac{2}{15} Wl$ $M_L = -\frac{7}{60} Wl$

* W represents the total distributed or point load in each case.

By considering any of the loads, the bending moment for any combination of loads can easily be found, and especially the maximum and minimum moments.

Once the negative moments and reactions are found, the diagrams for the bending moment and shear forces are to be superimposed on the diagrams of bending moments and shear forces for a beam on two supports.

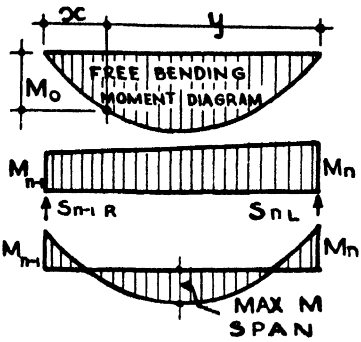


FIG. 362

For the bending moment and the shear force in the beam, at any point distant x from the left support, as shown in Fig. 362, the general formulæ obtain :

$$M = M_o + \left(\frac{M_{(n-1)}\, y + M_n x}{l}\right) \quad . \quad . \quad . \quad . \quad (50)$$

in which M_o would be the moment at the particular point for the beam on two simple supports, and $M_{(n-1)}$ and M_n are the bending moments (negative) at the left and right supports respectively ; and

$$S = S_o - \left(\frac{M_{(n-1)} - M_n}{l}\right) \quad . \quad . \quad . \quad . \quad . \quad (51)$$

in which S_o would be the shear at the particular point of the beam on two simple supports. The correct signs must be given to each of the individual factors in these formulæ.

The total centre support reaction equals the difference of the two shear forces on either side. Again signs must be carefully observed.

Shear forces and reactions, too, may be given as functions of all the loads, as previously shown for the support moments. For simple cases of loading, e.g. uniformly distributed load, the span moments may also be obtained from :

$$M_{span} = -S_{(n-1)R}\, x - \frac{wx^2}{2} + M_{(n-1)} \text{ (at the point distant } x \text{ from } n-1)$$

$$\text{Max. } M_{span} = \frac{S^2_{(n-1)R}}{2w} + M_{(n-1)} \quad . \quad . \quad . \quad . \quad (52)$$

The following examples (pp. 287 to 296) show the application of the Three Moment equations, and are to show at the same time how it is suggested that the values be set out in order that application and examination become easy.

Example 1.—*Beam on Three Supports.* (Exact calculation.)

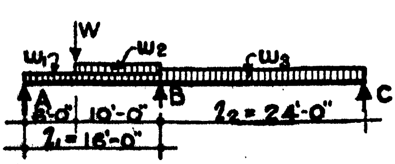


FIG. 363

$w_1 = 625$ lb. per ft. run (dead load).

$w_2 = 2000$ lb. per ft. run (live load).

$w_3 = 667$ lb. per ft. run (dead load).

$+ 1250$ lb. per ft. run (live load).

1917 lb. per ft. run.

$W = 27{,}040$ lb. (live load).

The moments of inertia of the T-sections (Fig. 364) can be found as shown on p. 240, thus:

Span AB:

$$I_1 = \frac{12^4}{12} + \frac{36 \times 8^3}{12} + \frac{12^2 \times 36 \times 8 \times 10^2}{12^2 + 36 \times 8}$$
$$= 1728 + 1536 + 9600 = 12{,}864 \text{ in.}^4$$

Span BC:

$$I_2 = \frac{12 \times 24^3}{12} + \frac{36 \times 8^3}{12} + \frac{12 \times 24 \times 36 \times 8 \times 16^2}{12 \times 24 + 36 \times 8}$$
$$= 13{,}824 + 1536 + 36{,}864 = 52{,}224 \text{ in.}^4$$

As only the ratio of these moments of inertia is important:

$$I_2 = \frac{52{,}224}{12{,}864} = 4{\cdot}06 \quad \text{when } I_1 = 1.$$

Section of span AB. Section of span BC.

FIG. 364

The equation of the Three Moments reduces to:

$$2\left(\frac{l_1}{I_1} + \frac{l_2}{I_2}\right)M_B = 2M_{BL} \times \frac{l_1}{I_1} + 2M_{BR}\frac{l_2}{I_2} \quad . \quad . \quad . \quad (53)$$

because $M_{(n-1)} = M_A$ and $M_{(n+1)} = M_C$ are zero.

The values of M_{BL} and M_{BR} are taken from Table 64, thus:

Influence of w_1 (Fig. 351) $M_{BL} = -\dfrac{(w_1 l_1) \times l_1}{8} = -32{\cdot}0\, w_1.$

Influence of w_2 (Fig. 350) $M_{BL} = -\dfrac{(w_2 a) \times a \times (2l_1 - a)^2}{8l^2}.$

$$= -\frac{w_2 \times 10^2 \times 22^2}{8 \times 16^2} = 23{\cdot}65\, w_2.$$

Influence of W (Fig. 352) $M_{BL} = -\dfrac{Wxy(l_1 + x)}{2l_1^{\,2}}$

$$= -\frac{W \times 6 \times 10 \times 22}{2 \times 16^2} = 2{\cdot}575 \text{ W}.$$

Influence of w_3 (Fig. 351) $M_{BR} = -\dfrac{(w_3 l_2) \times l_2}{8} = -72{\cdot}0\, w_3.$

Therefore by substituting these values in equation 53 above:

$$2\left(\frac{16}{1} + \frac{24}{4{\cdot}06}\right)M_B = -2 \times 32 \times \frac{16}{1}w_1 - 2 \times 23{\cdot}65 \times \frac{16}{1}w_2$$
$$- 2 \times 25{\cdot}75 \times \frac{16}{1}W - 2 \times 72{\cdot}0 \times \frac{24}{4{\cdot}06}w_3.$$
$$43{\cdot}84M_B = -1024w_1 - 756w_2 - 82{\cdot}5W - 851{\cdot}3w_3.$$
$$M_B = -23{\cdot}3w_1 - 17{\cdot}2w_2 - 1{\cdot}88W - 19{\cdot}4w_3.$$

With M_B having been found, the formulæ for all shear forces and reactions can now be determined:

$$S_{AL} = R_A = S^o_{AL} + \frac{M_B}{l_1}$$

$$= \left(\frac{16\cdot0}{2}w_1 + \frac{w_2 \times 10 \times 5}{16} + \frac{10}{16}W\right) - \left(\frac{23\cdot3}{16\cdot0}w_1 + \frac{17\cdot2}{16\cdot0}w_2 + \frac{1\cdot88}{16\cdot0}W + \frac{19\cdot4}{16\cdot0}w_3\right)$$

$$= +\ 6\cdot56w_1 + 2\cdot05w_2 + 0\cdot5W - 1\cdot21w_3.$$

$$S_{BL} = S_{AL} - \text{sum of all loads on span } l_1$$

$$= (6\cdot56 - 16\cdot0)w_1 + (2\cdot05 - 10\cdot0)w_2 + (0\cdot5 - 1\cdot0)W - 1\cdot21w_3$$

$$= -\ 9\cdot44w_1 - 7\cdot95w_2 - 0\cdot50W - 1\cdot21w_3.$$

Similarly:

$$S_{BR} = S^o_{BR} - \frac{M_B}{l^2} = \frac{24\cdot0}{2}w_3 + \frac{23\cdot3}{24\cdot0}w_1 + \frac{17\cdot2}{24\cdot0}w_2 + \frac{1\cdot88}{24\cdot0}W + \frac{19\cdot4}{24\cdot0}w_3$$

$$= 0\cdot96w_1 + 0\cdot72w_2 + 0\cdot08W + 12\cdot87w_3.$$

$$R_B = S_{BR} - S_{BL} = 10\cdot40w_1 + 8\cdot67w_2 + 0\cdot58W + 14\cdot02w_3.$$

$$R_C = -\ 0\cdot96w_1 - 0\cdot72w_2 - 0\cdot08W + (24\cdot0 - 12\cdot81)w_3$$

$$= -\ 0\cdot96w_1 - 0\cdot27w_2 - 0\cdot08W + 11\cdot19w_3.$$

Then from the above expressions the following numerical values are obtained:

$$\text{Max } M_B = -\ 23\cdot3 \times 625 - 17\cdot2 \times 2000 - 1\cdot88 \times 24{,}000 - 19\cdot4 \times 1917$$

$$= -\ 14{,}400 - 34{,}400 - 45{,}100 - 37{,}200$$

$$= -\ 131{,}100 \text{ lb.-ft.}$$

$$\text{Max. } R_A = 6\cdot56 \times 625 + 2\cdot05 \times 2000 + 0\cdot50 \times 24{,}000 - 1\cdot21 \times 667$$

$$= 4100 + 4100 + 12{,}000 - 810 = 19{,}390 \text{ lb.}$$

$$\text{Max. } S_{BL} = -\ 9\cdot44 \times 625 - 7\cdot95 \times 2000 - 0\cdot50 \times 24{,}000 - 1\cdot21 \times 1917$$

$$= -\ 5900 - 15{,}900 - 12{,}000 - 2300$$

$$= -\ 36{,}100 \text{ lb.}$$

$$\text{Max. } S_{BR} = 0\cdot96 \times 625 + 0\cdot72 \times 2000 + 0\cdot08 \times 24{,}000 + 12\cdot81 \times 1917$$

$$= 600 + 1440 + 1920 + 24{,}540$$

$$= 28{,}500 \text{ lb.}$$

$$\text{Max. } R_B = 36{,}100 + 28{,}500$$

$$= 64{,}600 \text{ lb.}$$

$$\text{Max. } R_C = -\ 0\cdot96 \times 625 + 11\cdot19 \times 1917 = 20{,}800 \text{ lb.}$$

The bending moments in the spans can be found from the end reactions. For panel AB it is found that the maximum bending moment occurs under the load W.

$$\text{Max. } M_{l_1} = 19{,}390 \times 6\cdot0 - \frac{625 \times 6\cdot0^2}{2}, \text{ at } x = 6 \text{ ft. } 0 \text{ in. from A}$$

$$= 116{,}340 - 11{,}240 = 105{,}100 \text{ lb.-ft.}$$

$$\text{Max. } M_{l_2} = \frac{20{,}800^2}{2 \times 1917}, \text{ at } x = \frac{20{,}800}{1917} = 10\cdot9 \text{ ft. from C}$$

$$= 113{,}000 \text{ lb.-ft.}$$

The shape of the bending moment and shear force diagrams is shown in Fig. 365.

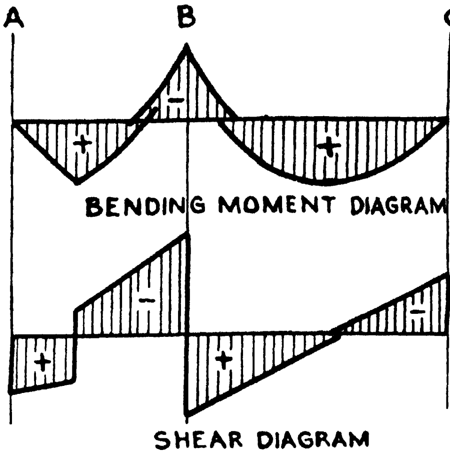
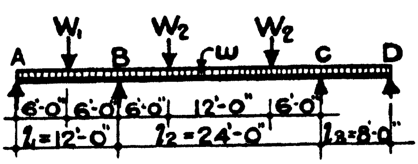


FIG. 365

Example 2.—*Beam on Four Supports.* (Exact calculation.) (Moments of inertia taken as being constant.)



$w = 800$ lb. per ft. run (dead load).
$W_1 = 12{,}000$ lb. (live load).
$W_2 = 30{,}000$ lb. (live load).

FIG. 366

The procedure is similar to that adopted for the previous example. First the two equations for M_B and M_C have to be established from Formula 48 or 49, for which purpose Table 64 can be used as before.

After solving these equations, formulæ for all shear forces and reactions can be found, and consequently the numerical values of all bending moments and shear forces are obtained. For diagrams of these bending moments and shear forces see Fig. 367.

$$2(12+24)M_B + 24M_C = -\frac{w(12^3+24^3)}{4} - \frac{3W \times 12^2}{8} - \frac{9W_2}{16} \times 24^2$$

$$= -3888w - 54W_1 - 324W_2. \quad . \quad . \quad (54)$$

$$24M_B + 2(24+8)M_C = -\frac{w(24^3+8^3)}{4} - 324W_2$$

$$= -3584w - 324W_2. \quad . \quad . \quad . \quad . \quad (55)$$

Divide equation (54) by 3 :

$$24M_B + 8M_C = -1296w - 18W_1 - 108W_2 \quad . \quad . \quad (56)$$

Take equation (54) away from equation (55) :

$$56M_C = -2288w + 18W_1 - 216W_2$$

$$M_C = -40{\cdot}86w + 0{\cdot}32W_1 - 3{\cdot}86W_2.$$

Divide equation (54) by 72 :

$$M_B = -54w - 0{\cdot}75W_1 - 4{\cdot}50W_2 - \frac{M_C}{3}$$
$$= -54w - 0{\cdot}75W_1 - 4{\cdot}50W_2 + 13{\cdot}62w - 0{\cdot}11W_1 + 1{\cdot}29W_2$$
$$= -40{\cdot}38w - 0{\cdot}86W_1 - 3{\cdot}21W_2$$
$$M_B - M_C = +0{\cdot}48w - 1{\cdot}18W_1 + 0{\cdot}65W_2$$

Reactions and Shear Forces :

$$R_A = \frac{12{\cdot}0}{2}w + 0{\cdot}5W_1 - \frac{40{\cdot}38}{12{\cdot}0}w - \frac{0{\cdot}86}{12{\cdot}0}W_1 - \frac{3{\cdot}21}{12{\cdot}0}W_2$$
$$= 2{\cdot}64w + 0{\cdot}43W_1 - 0{\cdot}27W_2.$$
$$S_{BL} = -(12{\cdot}0 - 2{\cdot}64)w - (1{\cdot}00 - 0{\cdot}43)W_1 - 0{\cdot}27W_2$$
$$= -9{\cdot}36w - 0{\cdot}57W_1 - 0{\cdot}27W_2$$
$$S_{BR} = \frac{24{\cdot}0}{2}w + W_2 - \frac{0{\cdot}48}{24{\cdot}0}w + \frac{1{\cdot}18}{24{\cdot}0}W_1 - \frac{0{\cdot}65}{24{\cdot}0}W_2$$
$$= 11{\cdot}98w + 0{\cdot}05W_1 + 0{\cdot}97W_2.$$
$$R_B = -S_{BL} + S_{BR} = 21{\cdot}34w + 0{\cdot}62W_1 + 1{\cdot}24W_2.$$
$$S_{CL} = -(24{\cdot}0 - 11{\cdot}98)w + 0{\cdot}05W_1 - (2 - 0{\cdot}97)W_2$$
$$= -12{\cdot}02w + 0{\cdot}05W_1 - 1{\cdot}03W_2.$$
$$S_{CR} = \frac{8{\cdot}0}{2}w + \frac{40{\cdot}86}{8{\cdot}0}w - \frac{0{\cdot}32}{8{\cdot}0}W_1 + \frac{3{\cdot}86}{8{\cdot}0}W_2$$
$$= 9{\cdot}11w - 0{\cdot}04W_1 + 0{\cdot}48W_2.$$
$$R_C = -S_{CL} + S_{CR} = 21{\cdot}13w - 0{\cdot}09W_1 + 1{\cdot}51W_2.$$
$$R_D = (8{\cdot}0 - 9{\cdot}11)w + 0{\cdot}04W_1 - 0{\cdot}48W_2$$
$$= -1{\cdot}11w + 0{\cdot}04W_1 - 0{\cdot}48W_2.$$

The numerical values become :

Support moments.

Max. $M_B = -40{\cdot}38 \times 800 - 0{\cdot}86 \times 12{,}000 - 3{\cdot}21 \times 30{,}000$
$= -138{,}900$ lb.-ft.

Max. $M_C = -40{\cdot}86 \times 800 - 3{\cdot}86 \times 30{,}000$
$= -148{,}500$ lb.-ft.

Reactions and Shear Forces.

Max. $R_A = 2{\cdot}64 \times 800 + 0{\cdot}43 \times 12{,}000 = 7270$ lb.

Min. $R_A = 2{\cdot}64 \times 800 - 0{\cdot}27 \times 30{,}000 = -5990$ lb. (uplift).

Max. $S_{BL} = -9{\cdot}36 \times 800 - 0{\cdot}57 \times 12{,}000 - 0{\cdot}27 \times 30{,}000$
$= -7490 - 6840 - 8100 = -22{,}430$ lb.

Max. $S_{BR} = 11{\cdot}98 \times 800 + 0{\cdot}05 \times 12{,}000 + 0{\cdot}97 \times 30{,}000$
$= 9580 + 600 + 29{,}100 = 39{,}280$ lb.

Max. $R_B = 22{,}430 + 39{,}280 = 61{,}710$ lb.

Max. $S_{CL} = -12{\cdot}02 \times 800 - 1{\cdot}03 \times 30{,}000$
$= -9620 - 30{,}900 = -40{,}520$ lb.

Max. $S_{C_R} = 9{\cdot}11 \times 800 + 0{\cdot}48 \times 30{,}000$

$= 7290 + 14{,}400 = 21{,}690$ lb.

Max. $R_C = 40{,}520 + 21{,}690 = 62{,}210$ lb.

Max. $R_D = -1{\cdot}11 \times 800 + 0{\cdot}04 \times 12{,}000$ (negative).

Min. $R_D = -1{\cdot}11 \times 800 - 0{\cdot}48 \times 30{,}000 = -15{,}290$ lb. (uplift).

Min. R_D and Max. R_D are calculated in this case as, being negative, they may affect the support construction.

Bending moments in spans.

Span 1: The maximum moment occurs under the load W_1 ($x = 6{\cdot}0$ ft. from A)

$$\text{Max. } M_{l_1} = 7260 \times 6{\cdot}0 - \frac{800 \times 6{\cdot}0^2}{2} = 29{,}200 \text{ lb.-ft.}$$

Span 2: To find Max. M_{l_2} (l_2 fully loaded), we must first find M_B and S_{B_R} for the load which produces the maximum moment in the centre span, namely dead load, plus live load on the centre span only.

$M_B = -40{\cdot}38 \times 800 - 3{\cdot}21 \times 30{,}000$

$= -32{,}300 - 96{,}300 = -128{,}600$ lb.-ft.

$S_{B_R} = 11{\cdot}98 \times 800 + 0{\cdot}97 \times 30{,}000$

$= 9580 + 29{,}100 = 38{,}680$ lb.

$$\text{Max. } M_{l_2} \text{ is at } x = \frac{38{,}680 - 30{,}000}{800} = 10{\cdot}83 \text{ ft. from B.}$$

$$\text{Max. } M_{l_2} = 38{,}680 \times 10{\cdot}83 - \frac{800 \times 10{\cdot}83^2}{2} - 30{,}000 \times 4{\cdot}83 - 128{,}600$$

$= 420{,}000 - 47{,}000 - 145{,}000 - 128{,}600$

$= 99{,}400$ lb.-ft.

Span 3: The bending moment is always negative throughout, as R_D is always negative.

The diagrams of bending moments and shear force are given in Fig. 367.

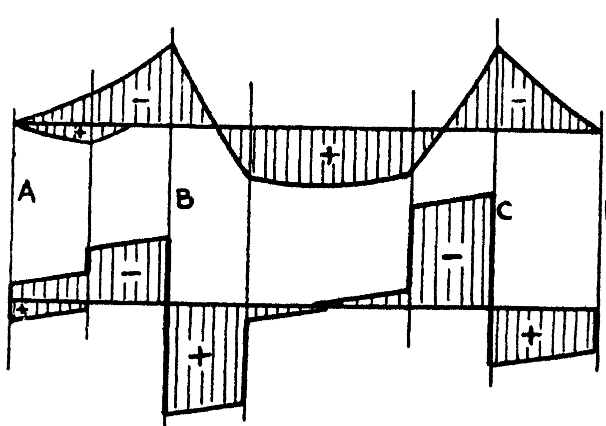
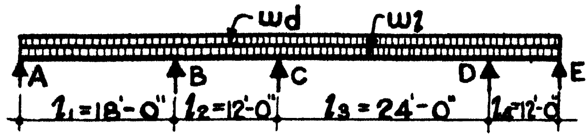


Fig. 367

Example 3.—*Beam on Five Supports.* (Exact calculation.) (For explanatory notes see foregoing examples.)



w_d=1000 lb. per ft. run (dead load).

$w_l = \frac{1000 \text{ lb. per ft. run}}{2000 \text{ lb. per ft. run}}$ (live load).

Fig. 368

The three equations for M_B, M_C, M_D are :

$$(1)\ 2(18+12)M_B + 12M_C = -\frac{18^3}{4}w_1 - \frac{12^3}{4}w_2.$$

$$(2)\ 12M_B + 2\,(12+24)M_C + 24M_D = -\frac{12^3}{4}w_2 - \frac{24^3}{4}w_3.$$

$$(3)\ 24M_C + 2(24+12)M_D = -\frac{24^3}{4}w_3 - \frac{12^3}{4}w_4.$$

$$(1)\ 60M_B + 12M_C = -1458w_1 - 432w_2.$$
$$(2)\ 12M_B + 72M_C + 24M_D = -432w_2 - 3456w_3.$$
$$(3)\ 24M_C + 72M_D = -3456w_3 - 432w_4.$$

$$(1)\ M_B = -\frac{1458}{60}w_1 - \frac{432}{60}w_2 - \frac{12}{60}M_C.$$

$$= -24{\cdot}3w_1 - 7{\cdot}2w_2 - \frac{M_C}{5}.$$

$$(2)\ M_D = -\frac{3456}{72}w_3 - \frac{432}{72}w_4 - \frac{M_C}{3}.$$

$$= -48w_3 - 6w_4 - \frac{M_C}{3}.$$

Divide equation (2) by 12 and substitute the above values of M_B and M_D :

$$M_B + 6M_C + 2M_D = -36w_2 - 288w_3$$

and thus

$$-24{\cdot}3w_1 + 7{\cdot}2w_2 - \frac{M_C}{5} + 6M_C - 96w_3 - 12w_4 - \frac{2M_C}{3} = -36w_2 - 288w_3.$$

$$\frac{77}{15}M_C = +24{\cdot}3w_1 - 28{\cdot}8w_2 - 192w_3 + 12w_4.$$

Finally :

$$M_C = +4{\cdot}74w_1 - 5{\cdot}61w_2 - 37{\cdot}4w_3 + 2{\cdot}34w_4.$$

$$M_B = -24{\cdot}3w_1 - 7{\cdot}2w_2 - \frac{4{\cdot}74}{5}w_1 + \frac{5{\cdot}61}{5}w_2 + \frac{37{\cdot}4}{5}w_3 - \frac{2{\cdot}34}{5}w_4$$

$$= -25{\cdot}25w_1 - 6{\cdot}08w_2 + 7{\cdot}48w_3 - 0{\cdot}47w_4.$$

$$M_D = -48w_3 - 6w_4 - \frac{4{\cdot}74}{3}w_1 + \frac{5{\cdot}61}{3}w_2 + \frac{37{\cdot}4}{3}w_3 - \frac{2{\cdot}34}{3}w_4$$

$$= -1{\cdot}58w_1 + 1{\cdot}87w_2 - 35{\cdot}55w_3 - 6{\cdot}78w_4.$$

$$M_B - M_C = -29{\cdot}99w_1 - 0{\cdot}47w_2 + 44{\cdot}88w_3 - 2{\cdot}81w_4$$

$$M_C - M_D = +6{\cdot}32w_1 - 7{\cdot}48w_2 - 1{\cdot}87w_3 + 9{\cdot}12w_4.$$

Reactions and Shear Forces:

$$R_A = \frac{w_1 l_1}{2} + \frac{M_B}{l_1} = 9{\cdot}0w_1 - 1{\cdot}40w_1 - 0{\cdot}34w_2 + 0{\cdot}42w_3 - 0{\cdot}03w_4$$
$$= 7{\cdot}60w_1 - 0{\cdot}34w_2 + 0{\cdot}42w_3 - 0{\cdot}03w_4.$$
$$S_{BL} = -w_1 l_1 + R_A = -10{\cdot}4w_1 - 0{\cdot}34w_2 + 0{\cdot}42w_3 - 0{\cdot}03w_4.$$
$$S_{BR} = \frac{w_2 l_2}{2} - \frac{M_B - M_C}{l_2} = 6{\cdot}0w_2 + 2{\cdot}5w_1 + 0{\cdot}04w_2 - 3{\cdot}74w_3 + 0{\cdot}23w_4$$
$$= 2{\cdot}50w_1 + 6{\cdot}04w_2 - 3{\cdot}74w_3 + 0{\cdot}23w_4.$$
$$R_B = -S_{BL} + S_{BR} = 12{\cdot}90w_1 + 6{\cdot}38w_2 - 4{\cdot}16w_3 + 0{\cdot}26w_4.$$
$$S_{CL} = -w_2 l_2 + S_{BR} = +2{\cdot}50w_1 - 5{\cdot}96w_2 - 3{\cdot}74w_3 - 0{\cdot}23w_4.$$
$$S_{CR} = \frac{w_3 l_3}{2} - \frac{M_C - M_D}{l_3} = 12{\cdot}00w_3 - 0{\cdot}26w_1 + 0{\cdot}31w_2 + 0{\cdot}08w_3 - 0{\cdot}38w_4$$
$$= -0{\cdot}26w_1 + 0{\cdot}31w_2 + 12{\cdot}08w_3 - 0{\cdot}38w_4.$$
$$R_C = -S_{CL} + S_{CR} = -2{\cdot}76w_1 + 6{\cdot}27w_2 + 15{\cdot}82w_3 - 0{\cdot}61w_4.$$
$$S_{DL} = -w_3 l_3 + S_{CR} = -0{\cdot}26w_1 + 0{\cdot}31w_2 - 11{\cdot}92w_3 - 0{\cdot}38w_4.$$
$$S_{DR} = \frac{w_4 l_4}{2} + \frac{M_D}{l_4} = 6{\cdot}0w_4 + 0{\cdot}13w_1 - 0{\cdot}16w_2 + 2{\cdot}96w_3 + 0{\cdot}57w_4$$
$$= 0{\cdot}13w_1 - 0{\cdot}16w_2 + 2{\cdot}96w_3 + 6{\cdot}57w_4.$$
$$R_D = -S_{DL} + S_{DR} = 0{\cdot}39w_1 - 0{\cdot}47w_2 + 14{\cdot}88w_3 + 6{\cdot}95w_4.$$
$$R_E = w_4 l_4 - S_{DR} = -0{\cdot}13w_1 + 0{\cdot}16w_2 - 2{\cdot}96w_3 + 5{\cdot}43w_4.$$

Then the numerical values are as follows:

Support Moments:

$$\text{Max. } M_B = -(25{\cdot}25 + 6{\cdot}08 + 0{\cdot}47)\,2000 + 7{\cdot}48 \times 1000$$
$$= -63{,}600 + 7480 = \underline{-56{,}120 \text{ lb.-ft.}}$$
$$\text{Max. } M_C = -(5{\cdot}61 + 37{\cdot}4)\,2000 + (4{\cdot}74 + 2{\cdot}34)\,1000$$
$$= -86{,}020 + 7080 = \underline{-78{,}940 \text{ lb.-ft.}}$$
$$\text{Max. } M_D = -(1{\cdot}58 + 35{\cdot}53 + 6{\cdot}78)\,2000 + 1{\cdot}87 \times 1000$$
$$= -87{,}780 + 1870 = \underline{-85{,}910 \text{ lb.-ft.}}$$

Reactions and Shear Forces:

$$\text{Max. } R_A = (7{\cdot}60 + 0{\cdot}42)\,2000 - (0{\cdot}34 + 0{\cdot}03)\,1000$$
$$= 16{,}040 - 370 = 15{,}670 \text{ lb.}$$
$$\text{Max. } S_{BL} = (10{\cdot}40 + 0{\cdot}34 + 0{\cdot}03)\,2000 - 0{\cdot}42 \times 1000$$
$$= 21{,}540 - 420 = 21{,}120 \text{ lb.}$$
$$\text{Max. } S_{BR} = (2{\cdot}50 + 6{\cdot}04 + 0{\cdot}23)\,2000 - 3{\cdot}74 \times 1000$$
$$= 17{,}540 - 3740 = 13{,}800 \text{ lb.}$$
$$\text{Max. } R_B = 21{,}120 + 13{,}800 = 34{,}920 \text{ lb.}$$
$$\text{Max. } S_{CL} = -(250 + 0{\cdot}23)\,1000 + (5{\cdot}96 + 3{\cdot}74)\,2000$$
$$= -2730 + 19{,}400 = 16{,}670 \text{ lb.}$$
$$\text{Max. } S_{CR} = -(0{\cdot}26 + 0{\cdot}38)\,1000 + (0{\cdot}31 + 12{\cdot}08)\,2000$$
$$= -640 + 24{,}780 = 24{,}140 \text{ lb.}$$
$$\text{Max. } R_C = 16{,}670 + 24{,}140 = 40{,}810 \text{ lb.}$$
$$\text{Max. } S_{DL} = (0{\cdot}26 + 11{\cdot}92 + 0{\cdot}38)\,2000 - 0{\cdot}31 \times 1000$$
$$= 25{,}120 - 310 = 24{,}810 \text{ lb.}$$

$$\text{Max. } S_{D_R} = (0{\cdot}13 + 2{\cdot}96 + 6{\cdot}57)\ 2000 - 0{\cdot}16 \times 1000$$
$$= 19{,}320 - 160 = 19{,}160 \text{ lb.}$$
$$\text{Max. } R_D = 24{,}810 + 19{,}160 = 43{,}970 \text{ lb.}$$
$$\text{Max. } R_E = -(0{\cdot}13 + 2{\cdot}96)\ 1000 + (0{\cdot}16 + 5{\cdot}43)\ 2000$$
$$= -3{\cdot}09 \times 1000 + 5{\cdot}59 \times 2000$$
$$= -3090 + 11{,}180 = 8090 \text{ lb.}$$

Alternatively:

$$R_E = -3{\cdot}09 \times 2000 + 5{\cdot}59 \times 1000$$
$$= -6180 + 5590 = -590 \text{ lb. (uplift).}$$

Span Moments.

Span 1:

$$\text{Max. } M_{l_1} = \frac{15{,}670^2}{2 \times 2000} = \underline{61{,}200 \text{ lb.-ft.}}$$

Span 2: To find the maximum and minimum value of M_{l_2}, two cases have to be considered:

(*a*) Live load on span l_2 and l_4 (max. M_{l_2}).

(*b*) Live load on span l_1 and l_3 (min. M_{l_2}).

In the first case we find:

$$M_B = -(25{\cdot}25 - 7{\cdot}48)\ 1000 - (6{\cdot}08 + 0{\cdot}47)\ 2000$$
$$= -17{\cdot}77 \times 1000 - 6{\cdot}55 \times 2000$$
$$= -17{,}770 - 13{,}100 = -30{,}870 \text{ lb.-ft.}$$
$$S_{B_R} = (2{\cdot}50 - 3{\cdot}74)\ 1000 + (6{\cdot}04 + 0{\cdot}23)\ 2000$$
$$= -1{\cdot}24 \times 1000 + 6{\cdot}27 \times 2000 = 11{,}300 \text{ lb.}$$
$$\text{Max. } M_{l_2} = \frac{11{,}300^2}{2 \times 2000} - 30{,}870 = \underline{+\ 1100 \text{ lb.-ft.}}$$

at a point $x = \dfrac{11{,}300}{2000} = 5{\cdot}65$ ft. from B.

Alternatively (case *b*):

$$M_B = -17{\cdot}77 \times 2000 - 6{\cdot}55 \times 1000 = -42{,}090 \text{ lb.-ft.}$$
$$S_{B_R} = 1{\cdot}24 \times 2000 + 6{\cdot}27 \times 1000 = 3790 \text{ lb.}$$
$$M_{l_2} = \frac{3790^2}{2 \times 1000} - 42{,}090 = \underline{-\ 34{,}900 \text{ lb.-ft.}}$$

at a point $x = \dfrac{3790}{1000} = 3{\cdot}79$ ft. from B.

Span 3: To find max. M_{l_3}, we assume spans 1 and 3 fully loaded. For these loads we find M_C and S_{C_R}.

$$M_C = (4{\cdot}74 - 37{\cdot}4)\ 2000 - (5{\cdot}61 - 2{\cdot}34)\ 1000$$
$$= -65{,}320 - 3270 = -68{,}590 \text{ lb.-ft.}$$
$$S_{C_R} = (-0{\cdot}26 + 12{\cdot}08)\ 2000 + (0{\cdot}31 - 0{\cdot}38)\ 1000$$
$$= 23{,}640 - 70 = 23{,}570 \text{ lb.}$$
$$\text{Max. } M_{l_3} = \frac{23{,}570^2}{2 \times 2000} - 68{,}590 = \underline{70{,}000 \text{ lb.-ft.}}$$

at a point, $x = \frac{23,570}{2000} = 11{\cdot}8$ ft. from C.

$$\text{Max. } M_{l_4} = \frac{8090^2}{2 \times 2000} = \underline{16,350 \text{ lb.-ft.}}$$

at a point $x = \frac{8090}{2000} = 4{\cdot}05$ ft. from E.

M_{l_4} may be negative throughout if R_E is negative.

The shapes of the bending moment and shear force diagrams are shown in Fig. 369.

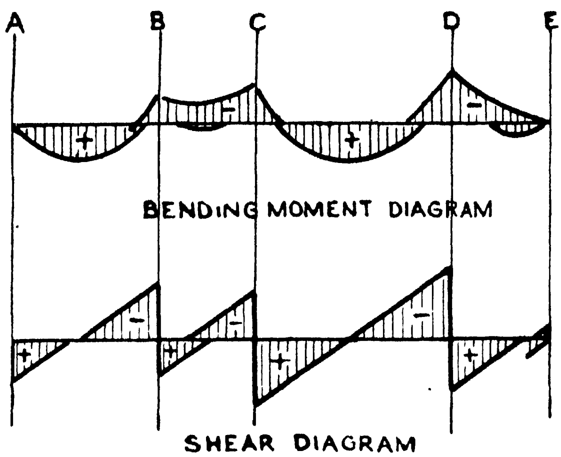


FIG. 369

Recommended Approximate Method.—The suggested approximate calculation for continuous beams involves the stiffnesses of adjoining members, in the same way as it does with the end columns of frames.

It is suggested that the investigation be carried out by two approximations, which can be made by two stages, the first approximation involving the processes set out immediately below and the formulæ 57, 58, 59, and 60, and the second involving the processes on p. 297 and the formulæ 61, 62, 63, 64, and 65.

For this method it is assumed that every load influences its own panel and the immediately adjoining panels, but no others—an assumption which, in many cases, follows the actual facts closer than the exact theory. This method is always preferable for slabs supported by tee-beams, quite apart from the saving in time which derives from its use.

In a beam on three supports, if the left panel were rigidly fixed at B, and hinged at the other support, a negative moment would occur, which is called M_L (Fig. 371).

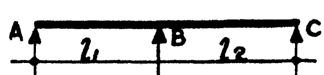


FIG. 370

FIG. 371

Attention is drawn to the fact that this moment, for which values due to various loadings are given in Table 64, is not identical with M_e for the end columns of frames, for here only one sided rigidity is assumed, whilst for the end columns of frames there is rigidity on both sides.

The actual negative moment at B, from the left-hand panel load, then will be:

$$M = M_L \left(\frac{l_1}{l_1 + l_2}\right) \text{ if the moments of inertia are equal.}$$

Or, if the moments of inertia are unequal:

$$M = M_L \left(\frac{\frac{l_1}{I_1}}{\frac{l_1}{I_1} + \frac{l_2}{I_2}}\right) = M_L \left(\frac{\frac{I_2}{l_2}}{\frac{I_2}{l_2} + \frac{I_1}{l_1}}\right)$$

$$= M_L \left(\frac{K_2}{K_1 + K_2}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (57)$$

where K_1 and K_2 are the stiffnesses, as defined before.

If there were a load on the right panel also which would create a bending moment M_R at B, if B were a rigid joint, the actual bending moment at B due to this load on the right would be:

$$M = M_R \left(\frac{K_1}{K_2 + K_1}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (58)$$

Therefore the total bending moment M_B at B due to the loads on both sides would be:

$$M_B = \frac{M_L K_2 + M_R K_1}{K_1 + K_2} \quad . \quad . \quad . \quad . \quad . \quad (59)$$

Or, if the moments of inertia are equal,

$$M_B = \frac{M_L l_1 + M_R l_2}{l_1 + l_2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (60)$$

These moments are correct if a beam on three supports only is considered; but they become merely the first approximation when the bending moment at one of several supports of a continuous beam is being considered. The values obtained from these formulæ would then be too high, due to the fact that the influence of the negative moments on the adjoining supports has been neglected.

To correct this, or in other words to proceed to the second approximation, we must first find the moments of the first approximation for the supports next on either side of the support under consideration. For these bending moments formulæ 59 and 60 are applied in exactly the same way as for M_B and the correct suffixes must be adopted. Thus if three consecutive supports are $(n - 1)$, (n), and $(n + 1)$ and if the bending moment at n is being investigated and if the moments of the first approximation are called $M'_{(n-1)}$, M'_n, and $M'_{(n-1)}$, then:

$$M'_{(n-1)} = \frac{M_{(n-1)L} K_n + M_{(n-1)R} K_{(n-1)}}{K_{(n-1)} + K_n} \quad . \quad . \quad . \quad (61)$$

$$M'_n = \frac{M_{nL} K_{(n+1)} + M_{nR} K_n}{K_n + K_{(n+1)}} \quad . \quad . \quad . \quad . \quad . \quad (62)$$

And:

$$M'_{(n+1)} = \frac{M_{(n+1)L} K_{(n+2)} + M_{(n+1)R} K_{(n+1)}}{K_{(n+1)} + K_{(n+2)}} \quad . \quad . \quad . \quad (63)$$

In all of which the symbols, $M_{(n-1)R}$, for example, represent their appropriate moments in the manner that $M_{(n-1)R}$ is the moment at the support $(n - 1)$ on the right-hand side of that support and due to the loading of the right-hand span, on the assumption that the beam is rigidly fixed at the support $(n - 1)$, but hinged at the adjacent support on the right. The values are given in Table 64.

Then for the moment at support n, in the second approximation, the authors have derived formulæ 64 and 65, which give consistent results of sufficient accuracy to be taken as the final moments.

Thus:

$$M_n = M'_n - \left(\frac{M'_{(n-1)} K_{(n+1)} + M'_{(n+1)} K_n}{3(K_n + K_{(n+1)})}\right) \quad . \quad . \quad (64)$$

If the moment of inertia is constant throughout, this formula becomes:

$$M_n = M'_n - \left(\frac{M'_{(n-1)} l_n + M'_{(n+1)} l_{(n+1)}}{3(l_n + l_{(n+1)})}\right) \quad . \quad . \quad (65)$$

After these support moments have been found the procedure for determining the shear forces, reactions, and span moments is the same as with the exact method.

It will be seen from the examples that the calculations to find bending moments and shears, which are sufficiently accurate for all practical purposes, are relatively short.

To calculate without risk of error it is advisable to tabulate all the values as they are computed, and this has been done for the following examples in Tables 65, 66, and 67. Such tables makes it very easy to find the maximum and minimum moments, or the bending moments for any given load.

The following examples are the same as those investigated by the exact method before, and the results of the exact method have been repeated at the ends of Tables 65, 66 and 67 to show the amount of error due to the approximation.

It does not follow that the variations between the results obtained by this method and by the exact method represent the differences between the actual behaviour and that calculated by the approximate method; on the contrary, as the behaviour of a beam is only approximately defined even by exact methods of calculations, this behaviour will differ from the exact calculations within a limit, which embrace the above mentioned error.

It will be seen that the approximation is close for all the more important cases of loading, i.e. for loads on the spans adjacent to the support under consideration.

The Memorandum allows the support moments to be reduced by an amount up to 15 per cent. provided that the span moments be increased correspondingly.

This reduction should not be made for beams where the spans or the loads differ considerably. Even in case of approximately equal spans and loads, the designer should always use his discretion, whether or not, to reduce the support moments. The negative moments decrease much quicker than the span moments. Therefore it will generally be found economical to supply rather heavy, but short, reinforcement over the supports. Only where the placing of this steel provides difficulties, especially with regard to passing the vertical bars of a heavily reinforced column, the facilities provided by the Memorandum will be found useful. Even then a check should be made to find if the full 15 per cent. reduction must be used or whether a smaller reduction will suffice.

Frames

The investigation will now be extended to continuous beams which are not free to rotate at their supports, i.e. to cases where the loads on the beam induce bending moments in the centre columns.

Beyond stating that where there exists an approximately symmetrical arrangement of beams and loading the bending moments in centre columns may be ignored, the Memorandum gives no further lead; and the first, and most important, question which must be answered, is that of the limits within which this symmetry may be assumed.

Since the whole of this method of design is approximate, reasonable assumptions must be made in settling these limits, and it is suggested that the following be adopted:

Bending moments in the column can be neglected if either of the following conditions obtain:

(a) If when W_1 and W_2 are the total loads on two adjacent spans of lengths l_1 and l_2 respectively, W_1l_1 and W_2l_2 do not differ by more than 30 per cent. of the smaller; and if the live load on either span, being an equally distributed live load, does not exceed 100 lb. per sq. ft., on the floor.

(b) If the stiffness of the beam is more than twenty times the stiffness of the column.

Where these conditions do not obtain it is considered that a symmetrical arrangement of loading cannot be assumed, and that the intervening column, therefore, should be investigated more correctly for the effects of the induced bending moments.

For the investigation of a centre column which does not fulfil the above conditions, the following method may be applied; and for this purpose the following notations have been adopted for any support n:

TABLE 68

SYMBOLS USED IN FRAME CALCULATIONS

K_{L_n} = stiffness of the beam to the left of the joint n.

K_{u_n} = stiffness of the upper part of column n.

K_{l_n} = stiffness of the lower part of column n.

K_{R_n} = stiffness of the beam to the right of the joint n. (See Fig. 372.)

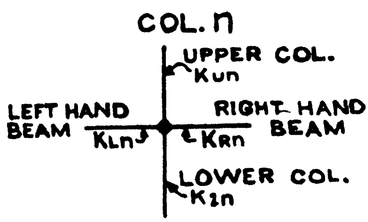


FIG. 372

$K_n = K_{L_n} + K_{u_n} + K_{l_n} + K_{R_n}$ = sum of the stiffnesses of all members connected to the joint.

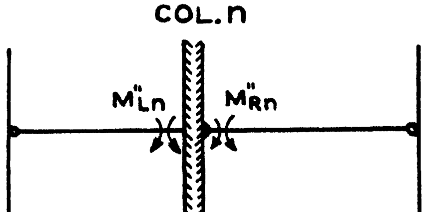


FIG. 373

M''_{L_n} = bending moment at the end of the left-hand beam if the beam were rigidly fixed at the joint and hinged at the other end. (See Fig. 373.) This is the same moment as M_L in Table 64.

M''_{R_n} = bending moment at the end of the right-hand beam if the beam were rigidly fixed at the joint and hinged at the other end. This is the same moment as M_R in Table 64.

M'_{L_n} = bending moment at support n at the end of the left-hand beam, in the first approximation, as defined by formula 66.

M'_{R_n} = bending moment at support n at the end of the right-hand beam, in the first approximation, as defined by formula 67.

M_{R_n} = bending moment at the end of the right-hand beam in second approximation, i.e. final bending moment as far as this approximate method is concerned.

TABLE 68—*continued.*

M_{L_n} = bending moment at the end of the left-hand beam in second approximation, i.e. final bending moment as far as this approximate method is concerned.
M_{u_n} = bending moment at the bottom of the upper column.
M_{l_n} = bending moment at the top of the lower column.

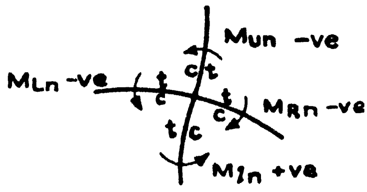


FIG. 374

Note.—The indices L and R refer to the beams on either side of the column, *not* to left-hand and right-hand ends of a span. Therefore the *left*-hand end of a beam spanning between columns $n - l$ and n has the index $R_{(n-1)}$, and the *right*-hand end of the same beam the index L_n. Consequently, as the two stiffnesses $K_{R(n-1)}$ and K_{L_n} refer to the same beam, $K_{R(n-1)} = K_{L_n}$ and similarly $K_{R_n} = K_{L(n+1)}$. See Fig. 375.

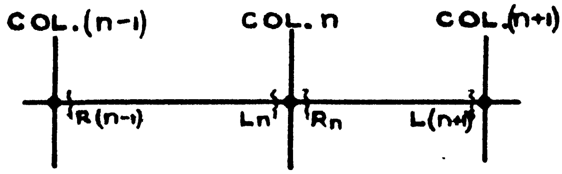


FIG. 375

With regard to the signs, bending moments should be called negative when producing tension at the top of a beam or at the right-hand edge of a column respectively, that is, on the hatched faces in Fig. 376 (see also Fig. 374).

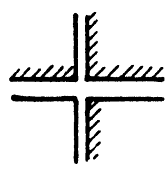


FIG. 376

With the notations set out in Table 68 the following formulæ obtain :

$$M'_{L_n} = M''_{L_n} \times \left(\frac{K_n - K_{L_n}}{K_n}\right) + M''_{R_n} \times \frac{K_{L_n}}{K_n} \quad . \quad . \quad . \quad . \quad . \quad . \quad (66)$$

$$M'_{R_n} = M''_{R_n} \times \left(\frac{K_n - K_{R_n}}{K_n}\right) + M''_{L_n} \times \frac{K_{R_n}}{K_n} \quad . \quad . \quad . \quad . \quad . \quad . \quad (67)$$

$$M_{L_n} = M'_{L_n} - \left(\frac{M'_{R(n-1)}}{3}\right) \times \left(\frac{K_n - K_{L_n}}{K_n}\right) - \left(\frac{M'_{L(n+1)}}{3}\right) \times \left(\frac{K_{L_n}}{K_n}\right) \quad . \quad (68)$$

$$M_{R_n} = M'_{R_n} - \left(\frac{M'_{L(n+1)}}{3}\right) \times \left(\frac{K_n - K_{R_n}}{K_n}\right) - \left(\frac{M'_{R(n-1)}}{3}\right) \times \left(\frac{K_{R_n}}{K_n}\right) \quad . \quad (69)$$

$$M_{u_n} = (M_{R_n} - M_{L_n}) \times \left(\frac{K_{u_n}}{K_{u_n} + K_{l_n}}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (70)$$

$$M_{l_n} = (M_{L_n} - M_{R_n}) \times \left(\frac{K_{l_n}}{K_{u_n} + K_{l_n}}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (71)$$

It will be seen that the above formulæ are extensions of those previously given for continuous beams, the only difference being that the primary moments M''_{L_n} and M''_{R_n} (which may again be taken from Table 64) are now distributed to all four members connected to the joint, in proportion to their relative stiffnesses, which, as pointed out before, are denoted by K_{L_n} (left-hand beam), K_{u_n} (upper column), K_{l_n} (lower column), and K_{R_n} (right-hand beam).

Thus the values of M'_{L_n} and M'_{R_n} are obtained, while the final expressions M_{L_n} and M_{R_n} take the influences of the two neighbouring joints into account.

JOINT n

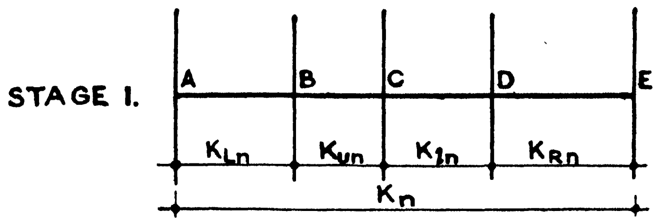


FIG. 377

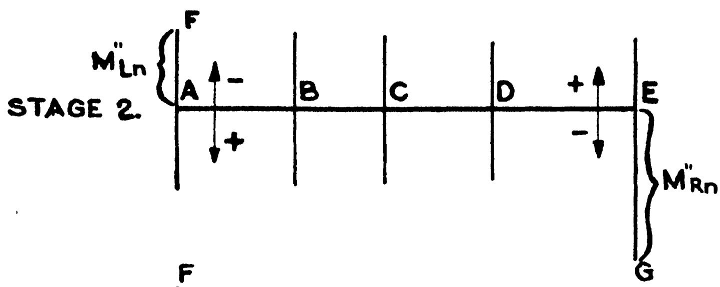


FIG. 378

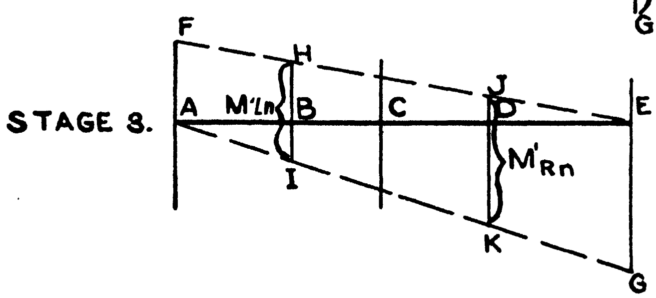


FIG. 379

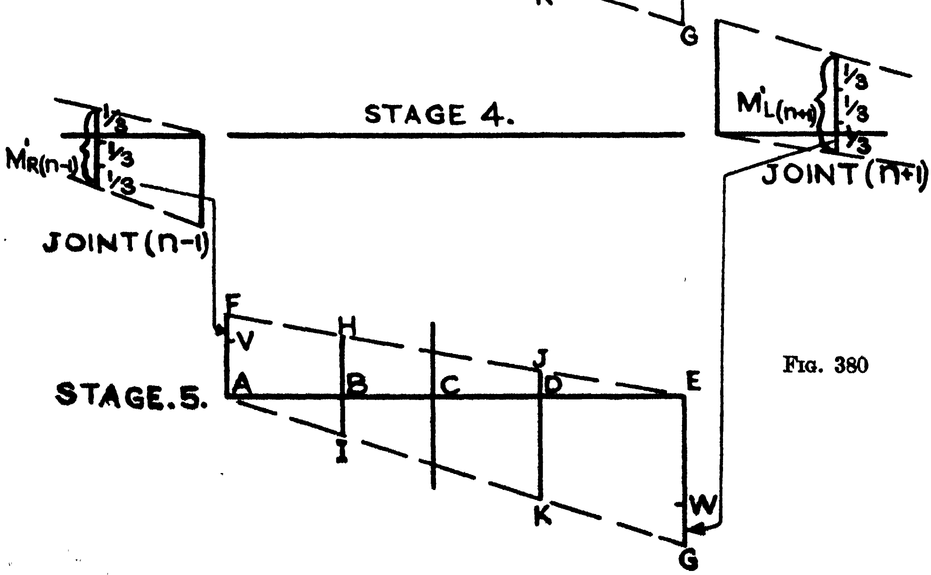


FIG. 380

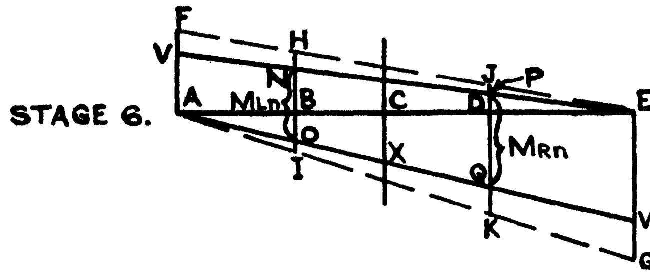


FIG. 381

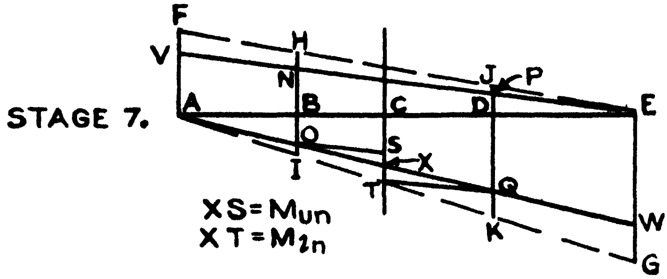


FIG. 382

Figs. 377 to 382 show a simple graphical method of determining M_{L_n}, M_{R_n}, M_{u_n}, and M_{l_n}. Draw, to any convenient scale, a horizontal line $AE = K_n = AB + BC + CD + DE = K_{L_n} + K_{u_n} + K_{l_n} + K_{R_n}$ and vertical lines through A, B, C, D, and E (Stage 1) (Fig. 377). Note that the order of the K values is important. It should not be changed from that given above. Make $AF = M''_{L_n}$ and $EG = M''_{R_n}$, where, for negative values of M''_{L_n} and M''_{R_n}, AF should point upward and EG downward (Stage 2) (Fig. 378).

The sloping lines FE and AG determine the values $HI = M'_{L_n}$ and $JK = M'_{R_n}$ (Stage 3) (Fig. 379).

Before proceeding any further, similar diagrams should be drawn for each of the joints at the adjoining supports (Stage 4) (Fig. 380).

We thus obtain values similar to HI and JK at the neighbouring joints, which represent $M'_{R(n-1)}$ and $M'_{L(n+1)}$ respectively.

Transfer $\frac{1}{3}$ of these latter moments to AF and EG respectively, thus fixing points V and W (Stage 5) (Fig. 380). The sloping lines VE and WA will then determine the final values $NO = M_{L_n}$ and $PQ = M_{R_n}$ (Stage 6) (Fig. 381). Proceed by drawing a line through O parallel to VE; this fixes the value $XS = M_{u_n}$ on the vertical line through C, and similarly $XT = M_{l_n}$ is obtained by drawing a line through Q parallel to VE (Stage 7) (Fig. 382). With $AW = M''_{L_n} - \frac{1}{3}M'_{R(n-1)}$ and $EV = M''_{R_n} - \frac{1}{3}M'_{L(n+1)}$, both being negative, which will normally be the case, the

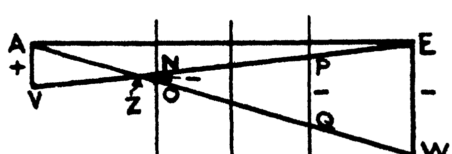


FIG. 383

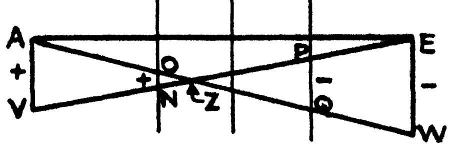


FIG. 384

resultant beam moments M_{L_n} and M_{R_n} are also negative; for positive values of AW and EV (including the case in which one of them is zero) the beam moments are of course also positive. If one of the above values is positive and the other negative, the two sloping lines AW and EV will intersect at Z, and the signs of the beam moments

will be determined by whichever side of point Z the ordinates of the beam moments lie. For instance in Fig. 383, ON is negative, whereas in Fig. 384 it is positive.

For the signs of the column moments M_{u_n} and M_{l_n}, that above the line AW is always negative and that below the line positive. In order that this rule may hold good for any case, care must be taken that the lines OS and QT, which determine the column moments, are always drawn through the correct points O and Q on AW. AW may be above or below the horizontal line or may even coincide with it. Some examples of possible positions of AW and the corresponding positions of OS and QT are shown in Fig. 385.

Care should be taken to determine the extreme positive *and* negative values of all bending moments. If in a reinforced concrete structure varying live loads

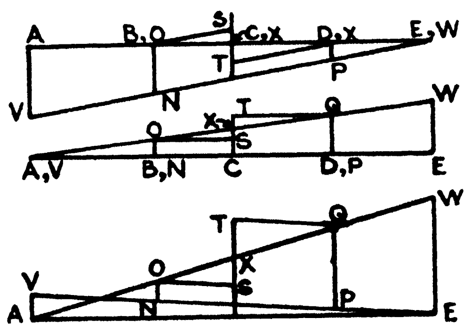


FIG. 385

produce bending moments of different signs, both moments must be investigated, even if the numerical value of one of them is much smaller than that of the other, for a small bending moment, of opposite sign, might well affect the steel in an asymmetrically reinforced section. Further, it should be noted that not always do the greatest values set up the worst conditions. In reinforced concrete columns, particularly, the smallest reactions combined with the maximum bending moments may call for the greatest amount of reinforcement.

The safest way of determining the worst cases is to apply the method explained in Figs. 377 *et seq.*, first for the dead load on the whole system and secondly for live loads separately on each panel; and then to tabulate all values, as shown in Table 69.

A diagrammatic example of this procedure, for a frame of three bays with a cantilever at one end, is given in Fig. 386.

In groups *a*, *b*, *c*, *d*, and *e* of Table 69 are given the correct signs of all moments to be obtained from Fig. 386; no numerical values, however, have been stated, as the example has been given purely to illustrate the principle, and it is not based on any actual loads and dimensions. The further groups of the table show how the numerical values *a* to *e* must be combined to give the worst conditions for all shear forces, reactions, and bending moments, and any possible combinations of the latter two. All these values are based on the loading diagrams given in Fig. 387.

The span moments from this method of calculation, are found by superimposing the appropriate end-moment diagram on the free diagram (Fig. 388). As an alterna-

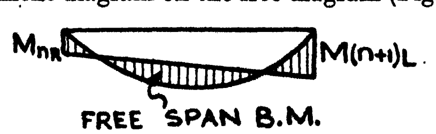


FIG. 388

tive to such a table, in cases of simple loading arrangements the bending moments, etc., may again be given as functions of the loads, thus:

$M = A_{w_1} + B_{w_2} + \ldots$ (see the numerical example at the end of this chapter).

The shear forces are determined by Formulæ 72 and 73. In these formulæ, if support n is under consideration

S_{L_n} = final total shear in the beam on the left side of the support.

S_{R_n} = final total shear in the beam of the right side of the support.

$S^o_{L_n}$ = the reaction on support n of the left-hand span n, if this span were simply supported.

$S^o_{R_n}$ = the reaction on support n of the right-hand span $(n + 1)$ if this span were simply supported.

$$S_{L_n} = S^o_{L_n} + \left(\frac{M_{L_n} - M_{R(n-1)}}{l_n}\right) \quad . \quad . \quad . \quad . \quad . \quad (72)$$

And:

$$S_{R_n} = S^o_{R_n} - \left(\frac{M_{R_n} - M_{L(n+1)}}{l_{(n+1)}}\right) \quad . \quad . \quad . \quad . \quad . \quad (73)$$

All the terms in these equations must be given their correct signs; $S^o_{L_n}$ and $S^o_{R_n}$ will normally be negative and positive respectively, and must be given their appropriate signs, as must also the M values, which, generally, all will be negative.

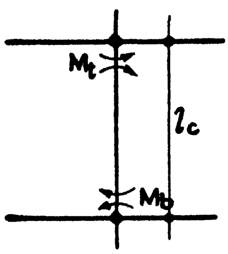


FIG. 389

The reactions may be found from:

$$R_n = S_{R_n} - S_{L_n}$$

(S_{R_n} and S_{L_n} being again given their proper signs), and it should be noted that the heaviest reaction, R_n, always occurs under the same conditions of loading which set up the two maximum shear forces S_{L_n} and S_{R_n}.

A word should be added with regard to the shear force in a column subject to bending. In short columns carrying heavy bending moments the shear force might be considerably high, and might require ties greatly in excess of those usually provided in a column under direct load only. It is, therefore, always advisable to check the shear force from the following formulæ (see Fig. 389).

$$S_c = \frac{M_t - M_b}{l_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (74)$$

where S_c = shear in column.

M_t = bending moment at the top of the column (with its proper sign).

M_b = bending moment at the bottom of the column (with its proper sign).

l_c = length of column, between centre lines of beams.

Where different live loads produce different moments, M_t and M_b, the values should be chosen so that the expression $M_t - M_b$ has its maximum value.

Example of a Beam Connected Rigidly to Centre Columns.—The properties of and the loads on the frame chosen as an example are shown in Figs. 390 and 391. The effects of wind loads are investigated for this same example on pp. 316 *et seq.*

(a) Stiffnesses:

Col. 1.

$$I_u = \frac{12 \times 8^3}{12} = 512 \text{ in.}^4; \; l = 9 \text{ ft. } 0 \text{ in.}; \; K_{u_1} = \frac{512}{9 \times 12} = 4{\cdot}7 \text{ in.}^3$$

$$I_l = \frac{15 \times 8^3}{12} = 640 \text{ in.}^4; \; l = 9 \text{ ft. } 0 \text{ in.}; \; K_{l_1} = \frac{640}{9 \times 12} = 5{\cdot}9 \text{ in.}^3$$

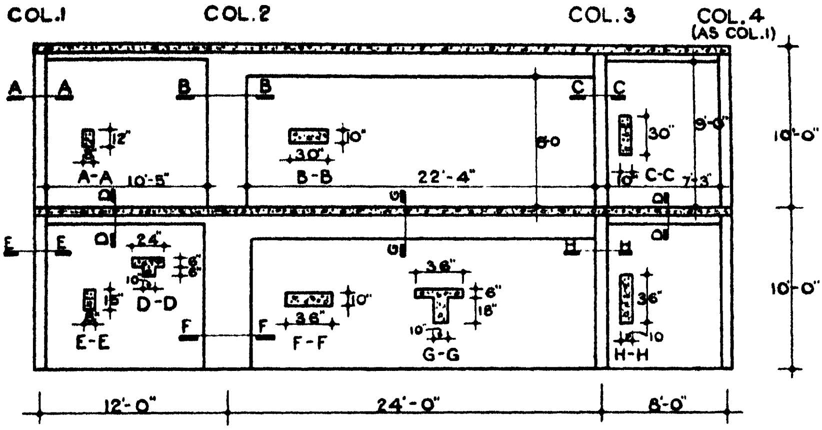


Fig. 390

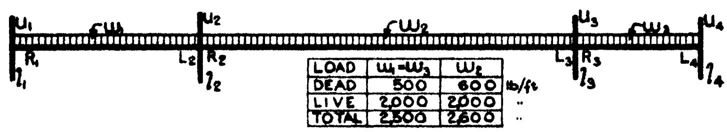


Fig. 391

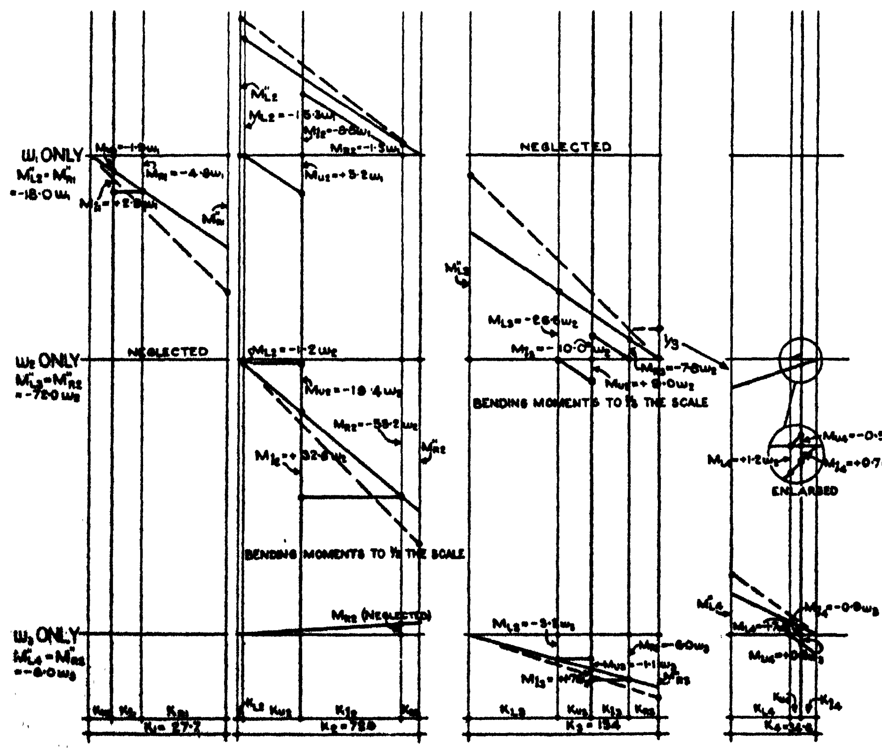
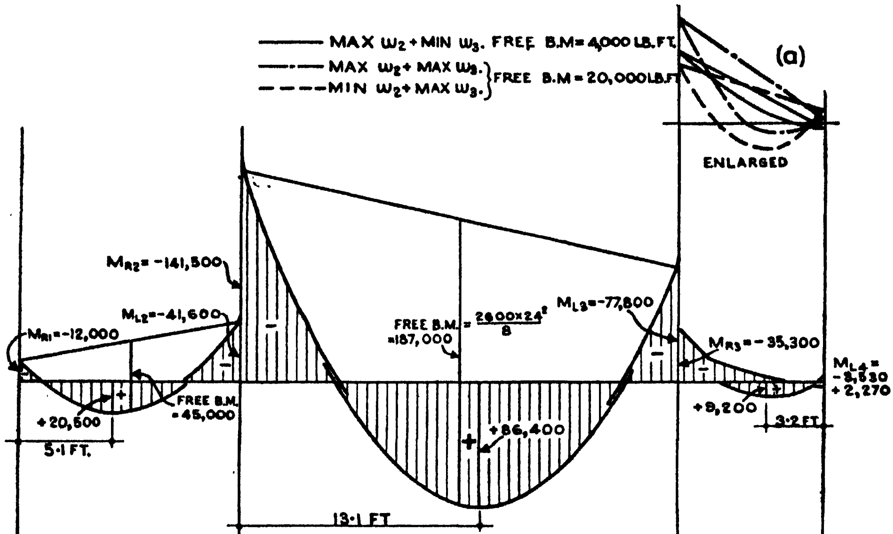


Fig. 392

Fig. 393

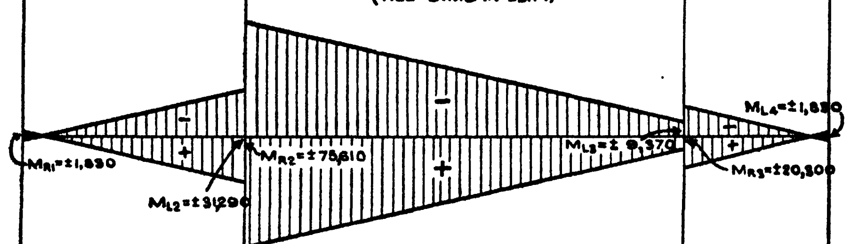
Fig. 394

GRAPHICAL METHOD FOR DETERMINING BENDING MOMENTS
FOR MEANINGS OF SYMBOLS SEE TABLE 68
FOR COEFFICIENTS OF M''_{L2}, M''_{R2} ETC. SEE TABLE 64



MAXIMUM BENDING MOMENTS DUE TO VERTICAL LOADS (ALL B.Ms. IN LB.FT)

Fig. 395



MAXIMUM BENDING MOMENTS DUE TO WIND

Fig. 396
See p. 316.

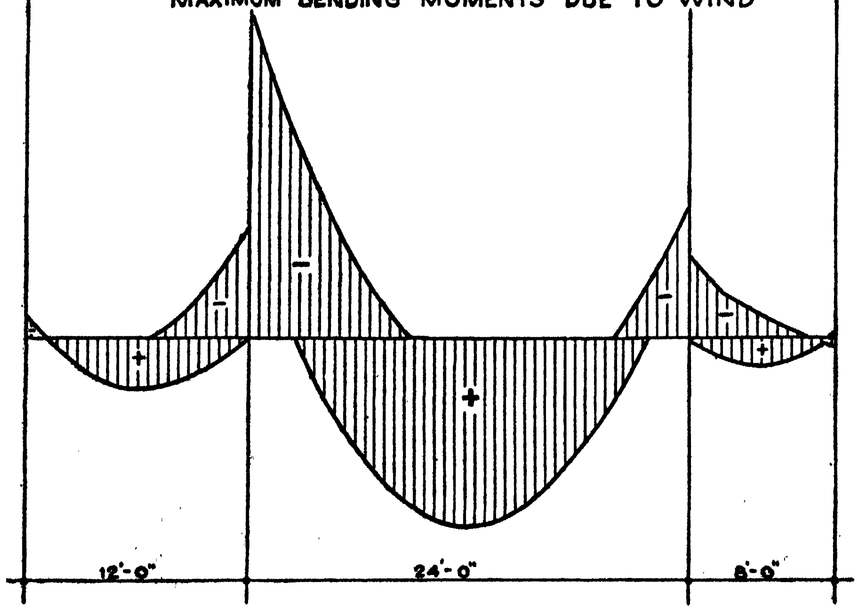


Fig. 397

MAXIMUM BENDING MOMENTS DUE TO VERTICAL LOADS+WIND

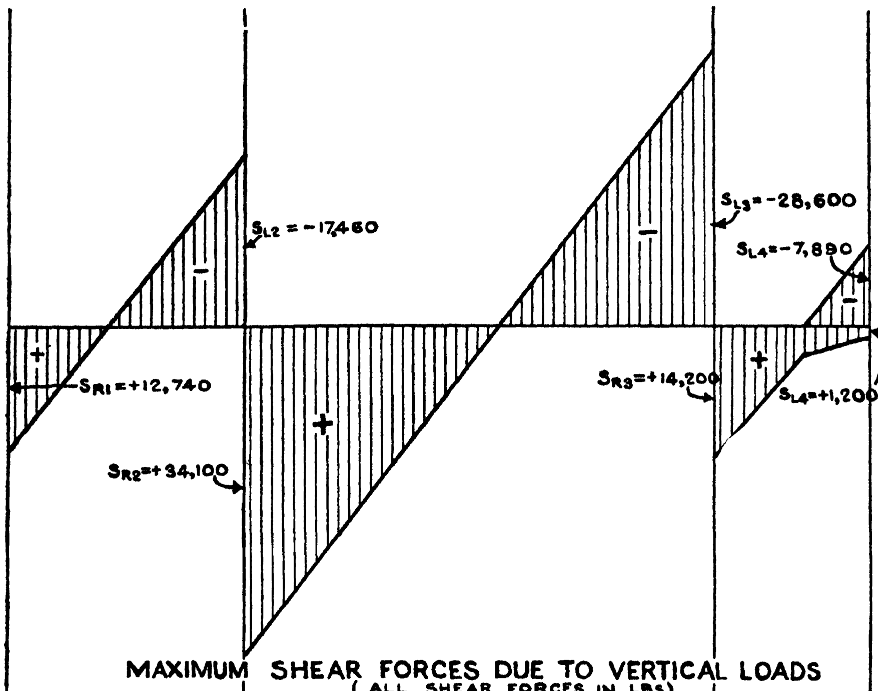


Fig. 398

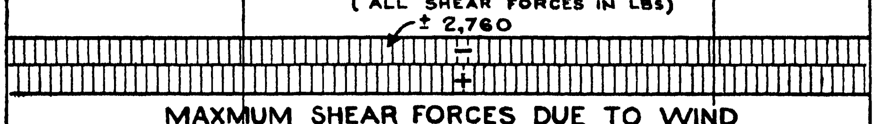


Fig. 399
See p. 316.

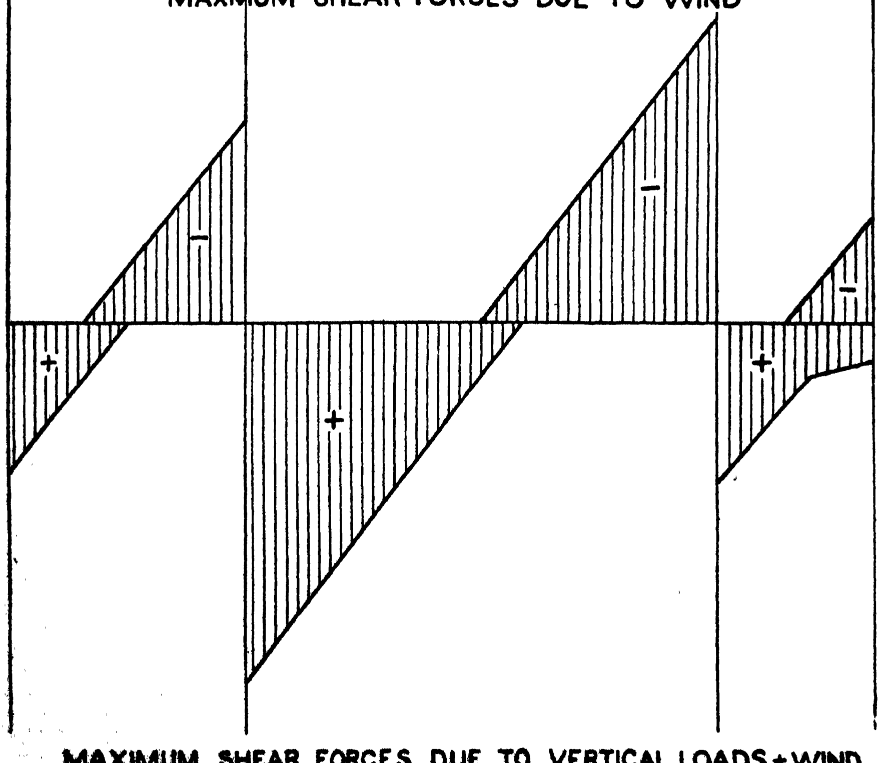


Fig. 400

Col. 2.

$$I_u = \frac{10 \times 30^3}{12} = 22{,}500 \text{ in.}^4 ; \; l = 8 \text{ ft. } 0 \text{ in.} ; \; K_{u_2} = \frac{22{,}500}{8 \times 12} = 234 \text{ in.}^3$$

$$I_l = \frac{10 \times 36^3}{12} = 38{,}880 \text{ in.}^4 ; \; l = 8 \text{ ft. } 0 \text{ in.} ; \; K_{l_2} = \frac{38{,}880}{8 \times 12} = 405 \text{ in.}^3$$

Col. 3.

$$I_u = \frac{30 \times 10^3}{12} = 2500 \text{ in.}^4 ; \; l = 8 \text{ ft. } 0 \text{ in.} ; \; K_{u_3} = \frac{2500}{8 \times 12} = 26 \text{ in.}^3$$

$$I_l = \frac{36 \times 10^3}{12} = 3000 \text{ in.}^4 ; \; l = 8 \text{ ft. } 0 \text{ in.} ; \; K_{l_3} = \frac{3000}{8 \times 12} = 31 \text{ in.}^3$$

Col. 4.

As Col. 1.

$$K_{u_4} = 4{\cdot}7 \text{ in.}^3$$
$$K_{l_4} = 5{\cdot}9 \text{ in.}^3$$

Beam 1–2.

$$I = \frac{10 \times 6^3}{12} + \frac{24 \times 6^3}{12} + \frac{10 \times 6 \times 24 \times 6 \times 6^2}{10 \times 6 + 24 \times 6} = 180 + 432 + 1525 = 2137 \text{ in.}^4$$

$$l = 10 \text{ ft. } 5 \text{ in.} = 125 \text{ in.} ; \; K_{R_1} = K_{L_2} = \frac{2137}{125} = 17{\cdot}1 \text{ in.}^3$$

Beam 2–3.

$$I = \frac{10 \times 18^3}{12} + \frac{36 \times 6^3}{12} + \frac{10 \times 18 \times 36 \times 6 \times 12^2}{10 \times 18 + 36 \times 6}$$

$$= 4860 + 648 + 14{,}138 = 19{,}646 \text{ in.}^4$$

$$l = 22 \text{ ft. } 4 \text{ in.} = 268 \text{ in.} ; \; K_{R_2} = K_{L_3} = \frac{19{,}646}{268} = 73 \text{ in.}^3$$

Beam 3–4.

$I = 2137$ in.^{4} (as for beam 1–2).

$$l = 7 \text{ ft. } 5 \text{ in.} = 89 \text{ in.} ; \; K_{R_3} = K_{L_4} = \frac{2137}{89} = 24{\cdot}0 \text{ in.}^3$$

$K_1 = 4{\cdot}7 + 5{\cdot}9 + 17{\cdot}1 = 27{\cdot}7$ in.^{3}
$K_2 = 17{\cdot}1 + 234 + 405 + 73 = 729$ in.^{3}
$K_3 = 73 + 26 + 31 + 24 = 154$ in.^{3}
$K_4 = 24{\cdot}0 + 4{\cdot}7 + 5{\cdot}9 = 34{\cdot}6$ in.^{3}

For the following calculations, the method of giving each bending moment, etc., as a function of the loads, has been adopted.

(b) Load on span 1–2 only :

$$M''_{R_1} = M''_{L_2} = -\frac{w_1 \times 12^2}{8} = -18{\cdot}0w_1.$$

From diagram (Fig. 392) :

$M_{u_1} = -1{\cdot}9w_1$; $M_{l_1} = +2{\cdot}9w_1$; $M_{R_1} = -4{\cdot}8w_1$;
$M_{L_2} = -15{\cdot}3w_1$; $M_{u_2} = +5{\cdot}0w_1$; $M_{l_2} = -8{\cdot}8w_1$; $M_{R_2} = -1{\cdot}3w_1$.

The influence of w_1 on joint 3 is, in this particular case, so small that it may be neglected.

(c) Load on span 2–3 only:

$$M''_{R_2} = M''_{L_3} = -\frac{w_2 \times 24^2}{8} = -72\cdot0w_2.$$

From diagram (Fig. 393):

$M_{L_2} = -1\cdot2w_2$; $M_{u_2} = -20\cdot4w_2$; $M_{l_2} = +32\cdot6w_2$; $M_{R_2} = -53\cdot2w_2$;
$M_{L_3} = -26\cdot8w_2$; $M_{u_3} = +9\cdot0w_2$; $M_{l_3} = -10\cdot0w_2$; $M_{R_3} = -7\cdot8w_2$;
$M_{L_4} = +1\cdot2w_2$; $M_{u_4} = -0\cdot5w_2$; $M_{l_4} = +0\cdot7w_2$.

The influence of w_2 on joint 1 is so small that it may be neglected.

(d) Load on span 3–4 only.

$$M''_{L_3} = M''_{R_4} = -\frac{w_3 \times 8^2}{8} = -8\cdot0w_3.$$

From diagram (Fig. 394):

$M_{L_3} = -3\cdot2w_3$; $M_{u_3} = -1\cdot1w_3$; $M_{l_3} = +1\cdot7w_3$; $M_{R_3} = -6\cdot0w_3$;
$M_{L_4} = -1\cdot7w_3$; $M_{u_4} = +0\cdot8w_3$; $M_{l_4} = -0\cdot9w_3$.

The influence of w_3 on joint 2 is again so small that it may be neglected.

(e) Bending moments, shear forces and reactions.

(Bending Moments have been scaled from the diagrams in Figs. 392, 393, and Fig. 394.)

$$\begin{aligned}
M_{u_1} &= -\ 1\cdot9w_1.\\
M_{l_1} &= +\ 2\cdot9w_1.\\
M_{R_1} &= -\ 4\cdot8w_1.\\
M_{L_2} &= -15\cdot3w_1 - 1\cdot2w_2.\\
M_{u_2} &= +\ 5\cdot0w_1 - 20\cdot4w_2.\\
M_{l_2} &= -\ 8\cdot8w_1 + 32\cdot6w_2.\\
M_{R_2} &= -\ 1\cdot3w_1 - 53\cdot2w_2.\\
M_{L_3} &= \qquad -26\cdot8w_2 - 3\cdot2w_3.\\
M_{u_3} &= \qquad +\ 9\cdot0w_2 - 1\cdot1w_3.\\
M_{l_3} &= \qquad -10\cdot0w_2 + 1\cdot7w_3.\\
M_{R_3} &= \qquad -\ 7\cdot8w_2 - 6\cdot0w_3.\\
M_{L_4} &= \qquad +\ 1\cdot2w_2 - 1\cdot7w_3.\\
M_{u_4} &= \qquad -\ 0\cdot5w_2 + 0\cdot8w_3.\\
M_{l_4} &= \qquad +\ 0\cdot7w_2 - 0\cdot9w_3.
\end{aligned}$$

$$\begin{aligned}
M_{L_2} - M_{R_1} &= -10\cdot5w_1 - 1\cdot2w_2.\\
M_{L_3} - M_{R_2} &= +\ 1\cdot3w_1 + 26\cdot4w_2 - 3\cdot2w_3.\\
M_{L_4} - M_{R_3} &= \qquad +\ 9\cdot0w_2 + 4\cdot3w_3.
\end{aligned}$$

$$R_1 = S_{R_1} = \frac{12\cdot0}{2}w_1 - \frac{10\cdot5}{12\cdot0}w_1 - \frac{1\cdot2}{12\cdot0}w_2 = 5\cdot12w_1 - 0\cdot10w_2.$$

$$S_{L_2} = -\frac{12\cdot0}{2}w_1 - \frac{10\cdot5}{12\cdot0}w_1 - \frac{1\cdot2}{12\cdot0}w_2 = -6\cdot88w_1 - 0\cdot10w_2.$$

$$S_{R_2} = \frac{24\cdot0}{2}w_2 + \frac{1\cdot3}{24\cdot0}w_1 + \frac{26\cdot4}{24\cdot0}w_2 - \frac{3\cdot2}{24\cdot0}w_3 = 0\cdot05w_1 + 13\cdot1w_2 - 0\cdot13w_3.$$

$$R_2 = S_{R_2} - S_{L_2} = 6\cdot93w_1 + 13\cdot20w_2 - 0\cdot13w_3.$$

$$S_{L_3} = -\frac{24\cdot0}{2}w_2 + \frac{1\cdot3}{24\cdot0}w_1 + \frac{26\cdot4}{24\cdot0}w_2 - \frac{3\cdot2}{24\cdot0}w_3 = 0\cdot05w_1 - 10\cdot90w_2 - 0\cdot13w_3.$$

$$S_{R_3} = \frac{8\cdot0}{2}w_3 + \frac{9\cdot0}{8\cdot0}w_2 + \frac{4\cdot3}{8\cdot0}w_3 = 1\cdot12w_2 + 4\cdot54w_3.$$

$$R_3 = S_{R3} - S_{L_2} = -0\cdot05w_1 + 12\cdot02w_2 + 4\cdot54w_3.$$

$$S_{L_4} = -\frac{8\cdot0}{2}w_3 + \frac{9\cdot0}{8\cdot0}w_2 + \frac{4\cdot3}{8\cdot0}w_3 = 1\cdot12w_2 - 3\cdot46w_3.$$

$$R_4 = -S_{L_4} = -1\cdot12w_2 + 3\cdot46w_3.$$

(f) Numerical values :

Before determining the numerical values some remarks are necessary on the manner of combining the bending moments and the reactions for the columns.

For centre columns the following four cases are to be distinguished and investigated :

Case (*a*).—Greatest bending moments in one direction, with appropriate reaction.

Case (*b*).—Greatest reaction, with appropriate bending moments.

Case (*c*).—Smallest reaction, with appropriate bending moments.

Case (*d*).—Greatest bending moments in other direction, with appropriate reaction.

The same four cases apply to end columns with cantilevers, but for end columns without cantilevers, case (*a*) = case (*b*) and case (*c*) = case (*d*).

Col. 1. *Case* (*a*) :

$M_{u_1} = -1\cdot9 \times 2500 = -4800$ lb.-ft.
$M_{l_1} = +2\cdot9 \times 2500 = +7300$ lb.-ft.
$R_1 = 5\cdot12 \times 2500 - 0\cdot10 \times 600 = 12{,}740$ lb.

Case (*c*) :

$R_1 = 5\cdot12 \times 500 - 0\cdot10 \times 2600 = 2300$ lb.
$M_{u_1} = -1\cdot9 \times 500 = -950$ lb.-ft.
$M_{l_1} = +2\cdot9 \times 500 = +1450$ lb.-ft.

Col. 2. *Case* (*a*) :

$M_{u_2} = +5\cdot0 \times 500 - 20\cdot4 \times 2600 = -50{,}500$ lb.-ft.
$M_{l_2} = -8\cdot8 \times 500 + 32\cdot6 \times 2600 = +80{,}400$ lb.-ft.
$R_2 = 6\cdot93 \times 500 + 13\cdot20 \times 2600 - 0\cdot13 \times 500 = 37{,}700$ lb.

Case (*b*) :

$R_2 = 6\cdot93 \times 2500 + 13\cdot20 \times 2600 - 0\cdot13 \times 500 = 51{,}600$ lb.
$M_{u_2} = +5\cdot0 \times 2500 - 20\cdot4 \times 2600 \times -40{,}500$ lb.-ft.
$M_{l_2} = -8\cdot8 \times 2500 + 32\cdot6 \times 2600 = +63{,}000$ lb.-ft.

Case (*c*) :

$R_2 = 6\cdot93 \times 500 + 13\cdot20 \times 600 - 0\cdot13 \times 2500 = 11{,}050$ lb.
$M_{u_2} = +5\cdot0 \times 500 - 20\cdot4 \times 600 = -9700$ lb.-ft.
$M_{l_2} = -8\cdot8 \times 500 + 32\cdot6 \times 600 = +15{,}200$ lb.-ft.

Case (*d*) :

$M_{u_2} = +5\cdot0 \times 2500 - 20\cdot4 \times 600 = +260$ lb.-ft.
$M_{l_2} = -8\cdot8 \times 2500 + 32\cdot6 \times 600 = -2440$ lb.-ft.
$R_2 = 6\cdot93 \times 2500 + 13\cdot20 \times 600 - 0\cdot13 \times 500 = 25{,}200$ lb.

Col. 3. *Case* (*a*) :

$M_{u_3} = +\ 9{\cdot}0 \times 2600 - 1{\cdot}1 \times 500 = +\ 22{,}900$ lb.-ft.

$M_{l_3} = -\ 10{\cdot}0 \times 2600 + 1{\cdot}7 \times 500 = -\ 25{,}200$ lb.-ft.

$R_3 = -\ 0{\cdot}05 \times 500 + 12{\cdot}02 \times 2600 + 4{\cdot}54 \times 500 = 33{,}500$ lb.

Case (*b*) :

$R_3 = -\ 0{\cdot}05 \times 500 + 12{\cdot}02 \times 2600 + 4{\cdot}54 \times 2500 = 42{,}600$ lb.

$M_{u_3} = +\ 9{\cdot}0 \times 2600 - 1{\cdot}1 \times 2500 = +\ 20{,}900$ lb.-ft.

$M_{l_3} = -\ 10{\cdot}0 \times 2600 + 1{\cdot}7 \times 2500 = -\ 21{,}800$ lb.-ft.

Case (*c*) :

$R_3 = -\ 0{\cdot}05 \times 2500 + 12{\cdot}02 \times 600 + 4{\cdot}54 \times 500 = 9400$ lb.

$M_{u_3} = +\ 9{\cdot}0 \times 600 - 1{\cdot}1 \times 500 = +\ 4850$ lb.-ft.

$M_{l_3} = -\ 10{\cdot}0 \times 600 + 1{\cdot}7 \times 500 = -\ 5150$ lb.-ft.

Case (*d*) :

$M_{u_3} = +\ 9{\cdot}0 \times 600 - 1{\cdot}1 \times 2500 = +\ 2650$ lb.-ft.

$M_{l_3} = -\ 10{\cdot}0 \times 600 + 1{\cdot}7 \times 2500 = -\ 1750$ lb.-ft.

$R_3 = -\ 0{\cdot}05 \times 500 + 12{\cdot}02 \times 600 + 4{\cdot}54 \times 2500 = 18{,}500$ lb.

Col. 4. *Case* (*a*) :

$M_{u_4} = -\ 0{\cdot}5 \times 600 + 0{\cdot}8 \times 2500 = +\ 1700$ lb.-ft.

$M_{l_4} = +\ 0{\cdot}7 \times 600 = 0{\cdot}9 + 2500 = -\ 1830$ lb.-ft.

$R_4 = -\ 1{\cdot}12 \times 600 + 3{\cdot}46 \times 2500 = 7980$ lb.

Case (*c*) :

$R_4 = -\ 1{\cdot}12 \times 2600 + 3{\cdot}46 \times 500 = -\ 1200$ lb. (uplift).

$M_{u_4} = -\ 0{\cdot}5 \times 2600 + 0{\cdot}8 \times 500 = -\ 900$ lb.-ft.

$M_{l_4} = +\ 0{\cdot}7 \times 2600 - 0{\cdot}9 \times 500 = +\ 1370$ lb.-ft.

Note.—Diagrams of the following beam moments and shear forces are shown in Figs. 395 and 398.

Beam 1–2 :

Max. $M_{R_1} = -\ 4{\cdot}8 \times 2500 = -\ 12{,}000$ lb.-ft.

Max. $M_{L_2} = -\ 15{\cdot}3 \times 2500 - 1{\cdot}3 \times 2600 = -\ 41{,}600$ lb.-ft.

Max. $S_{R_1} = 5{\cdot}12 \times 2500 - 0{\cdot}10 \times 600 = 12{,}740$ lb.

Max. $S_{L_2} = -\ 6{\cdot}88 \times 2500 - 0{\cdot}10 \times 2600 = -\ 17{,}460$ lb.

Max. $M_{(\text{span})} = \dfrac{12{,}740^2}{2 \times 2500} - 12{,}000 = +\ 20{,}500$ lb.-ft. at point $x = \dfrac{12{,}740}{2500} = 5{\cdot}1$ ft. from Col. 1.

Beam 2–3 :

Max. $M_{R_2} = -\ 1{\cdot}3 \times 2500 - 53{\cdot}2 \times 2600 = -\ 141{,}500$ lb.-ft.

Max. $M_{L_3} = -\ 26{\cdot}8 \times 2600 - 3{\cdot}2 \times 2500 = -\ 77{,}800$ lb.-ft.

Max. $S_{R_2} = 0{\cdot}05 \times 2500 + 13{\cdot}10 \times 2600 - 0{\cdot}13 \times 500 = 34{,}100$ lb.

Max. $S_{L_3} = 0{\cdot}05 \times 500 - 10{\cdot}90 \times 2600 - 0{\cdot}13 \times 2500 = -\ 28{,}600$ lb.

To find Max. $M_{(\text{span})}$ (live load on 2–3 only) :

$M_{R_2} = -\ 1{\cdot}3 \times 500 - 53{\cdot}2 \times 2600 = -\ 137{,}700$ lb.-ft.

$S_{R_2} = 0{\cdot}05 \times 500 + 13{\cdot}10 \times 2600 - 0{\cdot}13 \times 500 = 34{,}000$ lb.

Max. $M_{(\text{span})} = \dfrac{34{,}000^2}{2 \times 2600} - 137{,}700 = 84{,}600$ lb.-ft. at point $x = \dfrac{34{,}000}{2600} = 13{\cdot}1$ ft. from Col. 2.

Beam 3–4 :

Max. $M_{R_3} = -7{\cdot}8 \times 2600 - 6{\cdot}0 \times 2500 = -35{,}300$ lb.-ft.

Max. $M_{L_4} = +1{\cdot}2 \times 600 - 1{\cdot}7 \times 2500 = -3530$ lb.-ft.

Max. $S_{R_3} = 1{\cdot}12 \times 2600 + 4{\cdot}54 \times 2500 = 14{,}200$ lb.

Max. $S_{L_4} = 1{\cdot}12 \times 600 - 3{\cdot}46 \times 2500 = -7980$ lb.

Max. $M_{l(\text{span})} = \dfrac{7980^2}{2 \times 2500} - 3530 = 9200$ lb.-ft. at point $x = \dfrac{7980}{2500} = 3{\cdot}2$ ft. from Col. 4.

Alternatively :

Max. positive $M_{L_4} = 1{\cdot}2 \times 2600 - 1{\cdot}7 \times 500 = +2270$ lb.-ft.

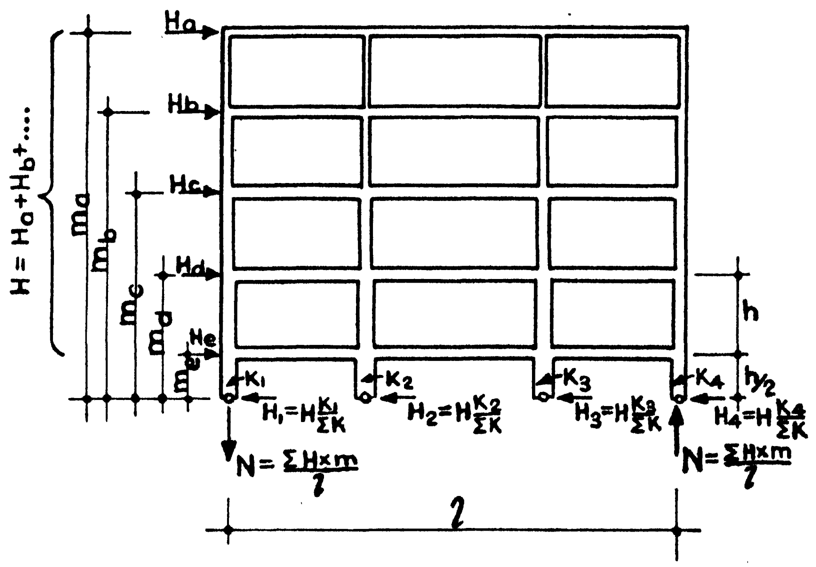
And for the same condition of loading :

$$M_{R_3} = -7{\cdot}8 \times 2600 - 6{\cdot}0 \times 500 = -23{,}300 \text{ lb.-ft.}$$

Note.—The bending moments in this latter span vary considerably. Therefore, complete diagrams were drawn for three different conditions (see Fig. 395 at (*a*)), and from these the curves of the maximum positive and negative moments were obtained.

LATERAL FORCES ON STRUCTURES

Most building are subject to horizontal forces in addition to the vertical loads which they must sustain, and this horizontal loading, for most multi-storey buildings, will be due principally to wind. Power stations and similar buildings are sometimes



Note : H_a, H_b, etc., represent the wind loads applied on the appropriate floors and not the total horizontal forces at any level.

FIG. 401

subjected to the pull of transmission cables; and a horizontal force may also be exerted on a structure by tapered chutes of bunkers. The internal pressures of the contents of bunkers are generally balanced, and thus do not affect the stability of the building as a whole. Buildings which are constructed against a hill may experience considerable one-sided earth pressure, which must be balanced by the resistance of the internal framework; and on structures which carry moving traffic horizontal forces are exerted particularly when a vehicle brakes or accelerates.

To deal with wind pressure, and other occasional horizontal forces which obey the same rules, approximate methods of calculation may be applied in the same manner as for the determination of moments due to vertical loads.

In By-law 6 the conditions under which allowance must be made for wind loads are set out, as are also the intensities of the loads and the areas over which they must be applied.

The following six rules are to be observed :

(1) The horizontal loads on a frame are to be split into as many parts as there are columns to resist them, these parts are to be proportional to the stiffnesses of the columns. If in one frame there are n columns assisting in the resistance to a horizontal force H, and if their stiffnesses are K_1, K_2, etc., to K_n, the first column takes the force $H \cdot \dfrac{\frac{K_1}{h_1}}{\sum \frac{K}{h}}$; the second $H \cdot \dfrac{\frac{K_2}{h_2}}{\sum \frac{K}{h}}$ and so on.

If the height of all columns is equal, as it will be generally, these formulæ can be simplified so that the first column takes the part $H \cdot \frac{K_1}{\Sigma K}$ and the second $H \cdot \frac{K_2}{\Sigma K}$, etc. Under ordinary circumstances it is unnecessary to determine these proportions anew for every floor, because it is sufficiently accurate to keep to average stiffnesses. There are, however, exceptions—for example, the case in which one or several of

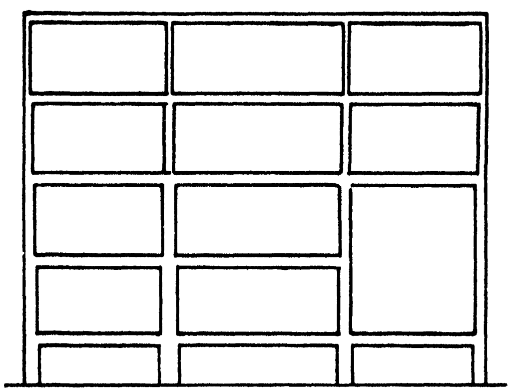


FIG. 402

the columns is made exceptionally small in one or several floors, so that the proportion of its stiffnesses to those of the remaining columns is much smaller than the corresponding proportions in the floor above or below; or, as another example, if one column is not stiffened by a horizontal beam in one floor, as in Fig. 402.

(2) The horizontal force acting on any floor is equal to the sum of all horizontal forces acting above and at the level of this floor.

(3) Where regular structures are considered, that is where the beams in different floors are approximately equal, and where the columns increase in section gradually from roof to base, the points of contraflexure can be assumed to be at the mid-height between any two floors.

For a special case, however, the distance of the point of contraflexure from the upper end can be assumed as :

$$a' = h' \times \left(\frac{R_T}{R_T + R_B}\right) \quad . \quad . \quad . \quad . \quad . \quad (75)$$

in which h' is the clear height of the column and R_B and R_T are the rigidities at the bottom and top, as explained before, namely:

$$R_B = \frac{K_B}{K_B + K}$$

$$R_T = \frac{K_T}{K_T + K}$$

where K is the stiffness of the column and K_T and K_B are the sums of the stiffnesses of the other members keeping the column rigid at the upper and the lower end respectively (see Fig. 403). The rigidity of a foundation should be taken either

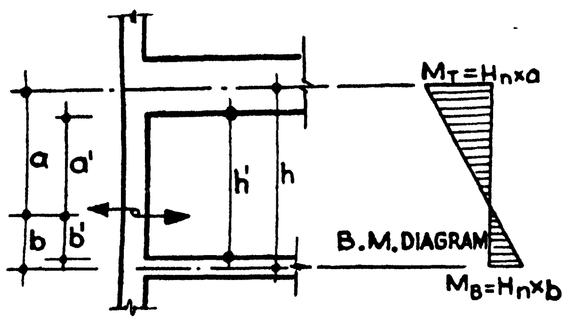


FIG. 403

as zero or unity, depending on whether the column is rigidly fixed or hinged, but not as any intermediate value. This means that a foundation should either be sufficiently strong to take the overturning moment resulting from full rigidity or should be considered as a pin-joint.

(4) The bending moment at the upper end of each column will be: $M_T = H_n a$, where H_n is the portion of the total horizontal force which is taken by the column n under consideration, and a the distance of the point of contraflexure from the upper end—which, as explained in (3) will be, in many cases, half the height between the floors. The bending moment at the lower end of a column will be: $M_B = H_n b$, where b is the distance of the point of contraflexure from the lower end (see Fig. 403).

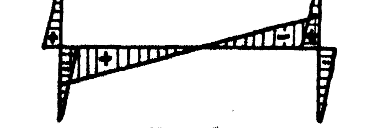


FIG. 404
(See paragraph 6)

(5) There are also vertical forces in the columns due to the overturning moment on the structure as a whole. The centre columns take such a small part of this overturning moment that it can generally be assumed that the whole of the overturning moment is taken by the end columns of a frame.

Therefore, $N = \pm \frac{OM}{l}$, N being the normal or vertical force due to the overturning moment in an end column and OM being the overturning moment due to all the wind load on the portion of the building above, such that $OM = \Sigma H \times m$ (m is the height of the line of application of the wind force H (at any floor) above the point of contraflexure of the column under consideration and l is the distance between the centre-lines of the two end columns of the particular frame) (see Fig. 401).

(6) The beams also, will receive bending moments, due to the horizontal loads, by virtue of their connections to the columns. If there is a frame of only one bay, the bending moments at the ends of the beams are simply $M_{R1} = \pm (M_{u_1} - M_{l_1})$;

$M_{L_n} = \pm (M_{u_n} - M_{l_n})$ (one of the column moments is negative, so that the numerical values of the beam moments are actually equal to the sums of the numerical values of the column moments) (see Fig. 404).

If the frame is of several bays (Fig. 405) the bending moments in the beams at the end columns will also be determined by the formulæ: $M_{R_1} = \pm (M_{u_1} - M_{l_1})$; $M_{L_n} = \pm (M_{u_n} - M_{l_n})$. We can then determine the bending moment for the end span, near the centre column from: $M_{L_2} = M_{R_1} - S_{R_1} \times l_1$; where S_{R_1} is the vertical shear in the beam equal to the difference of the values of N for the columns above and below the beam. If we consider a portion of frame between the points of contraflexure of two adjacent floors, with the horizontal forces H_u and H_l acting on them (where H_u is equal to the sum of all horizontal forces above and H_l is equal to H_u

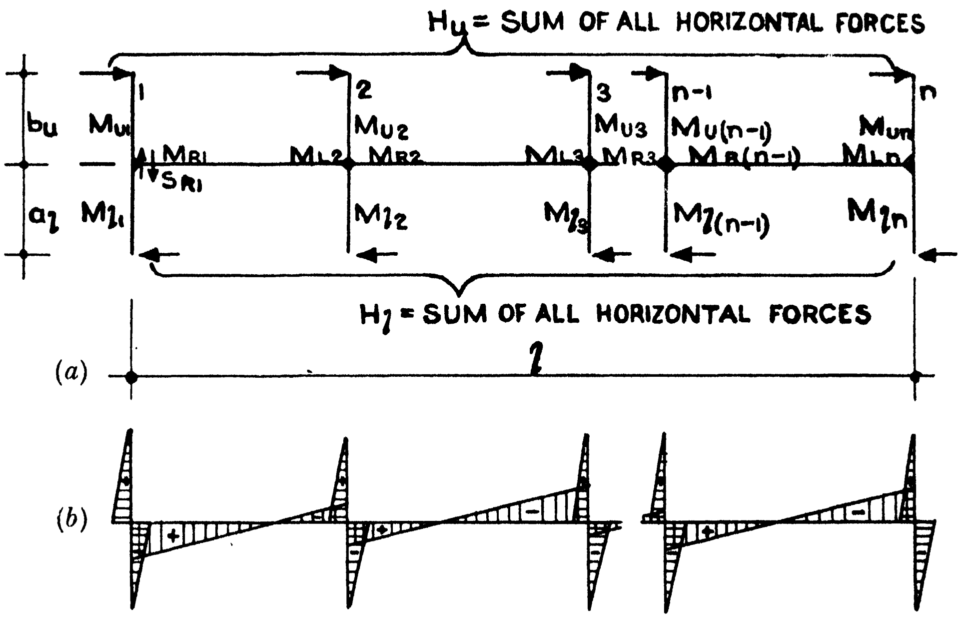


FIG. 405

increased by whatever horizontal force acts at the level of the floor which is under consideration), then also $S_{R_1} = \dfrac{H_u b_u + H_l a_l}{l} = S_{L_2}$ where l is the total length of the frame, and $M_{R_2} = M_{L_2} + (M_{u_2} - M_{l_2})$, in which M_{l_2} and M_{u_2} are the bending moments in the lower and upper part of column 2.

We then find

$$M_{L_3} = M_{R_2} - S_{R_2} \times l_2$$
$$M_{R_3} = M_{L_3} + (M_{u_3} - M_{l_3})$$

and so on.

The bending moments are shown in Fig. 405 (*b*), but as the shear force is constant throughout the length of the beam [i.e. as $S_{R_1} = S_{L_2} = S_{R_2}$ etc. $= S$] a separate shear diagram has not been given.

Finally we come to the other end of the frame, and have here a check on the calculation, as the last bending moment, M_{L_n}, which equals $M_{R_{(n-1)}} - S \times l_n$, must also be equal to $M_{u_n} - M_{l_n}$.

To simplify these calculations, it is strongly recommended that whenever a frame is designed and the preliminary sizes of columns and beams fixed, a sketch of the frames be made showing the stiffness of each member. This will enable the designer to see at once whether an ordinary or regular case obtains, and, therefore, whether the preceding simplifications (average stiffness of a whole column from top to bottom, and assumption of point of contraflexure in centre of column span) are applicable or not. With regard to wind, the fact should be held in mind that working stresses may be increased by $33\frac{1}{3}$ per cent. under the conditions of By-law 90. Very often a preliminary survey shows that even at the worst point stresses due to wind are less than a third of the remaining stresses. In such a case it is permissible not to pursue the investigation of wind effects any further.

Local bending moments caused by wind against the column between two floors can be generally neglected; but earth pressure may have considerable local effects on the columns. In this case a column can be considered as a continuous beam supported by the beams or floor-slabs acting as columns, and reactions upon these supports may be found. These reactions, then, are horizontal forces acting on the frame as a whole, and the moment diagram for the continuous column is to be superimposed on whatever moments the same column obtains as part of the frame from the vertical forces.

A word is to be added about the superimposing of moment diagrams. It is, of course, always permissible to add the maximum bending moment due to one kind of load to the maximum bending moment of another, because one is on the safe side in doing so. However, as these maximum bending moments due to different loads usually occur at different points, it is advisable for economy to add the diagrams to each other and to check where the resultant maximum moment occurs. This is comparatively easy if only concentrated loads occur, for the moment diagrams then consist only of straight lines. (See Fig. 406.)

In this case, the addition of the diagrams may be done by adding the calculated bending moments for each system of loading at points where there is a change of direction in the diagram for either (or if there are more than two systems of loading, for any) of the systems. The moments at each of these points have then to be added algebraically. Where distributed loads occur, it is advisable to find the end reactions for each system of loading and end bending moments first, and from these the equations of the span moment curves, which equations may be differentiated to find the maximum moments. Where uniformly distributed loads and concentrated loads occur together, it is best to superimpose the shear diagrams and thus, where the resultant shear is zero, to find the maximum moment.

It is recommended generally that the designer treat shear diagrams as of equal importance as moment diagrams, because particularly in reinforced concrete, the providing of reinforcement against shear frequently offers a greater problem than it does against bending moments, and thus the more the designer knows about the distribution of shear in any structural member, the easier it is for him to cope with such problems. Shear force diagrams added, as are moment diagrams, are shown in Fig. 407.

Numerical Example.—The foregoing theory will now be applied to the frame shown in Fig. 390. The bending moments due to the vertical loads are given in Fig. 395, p. 306, and the wind forces are shown in Fig. 408.

Let the spacing of frames equal 18 ft. 0 in. Then:

$$H = 18 \times 10 \times 15 = 2700 \text{ lb. per floor.}$$

Take same force for roof, to allow for parapet, roof houses, etc. Assume four floors above the one under consideration. Then:

$$H_u = 4 \times 2700 = 10,800 \text{ lb.}$$

$$H_l = 5 \times 2700 = 13,500 \text{ lb.}$$

All points of contraflexure can be assumed to be halfway between the floors.

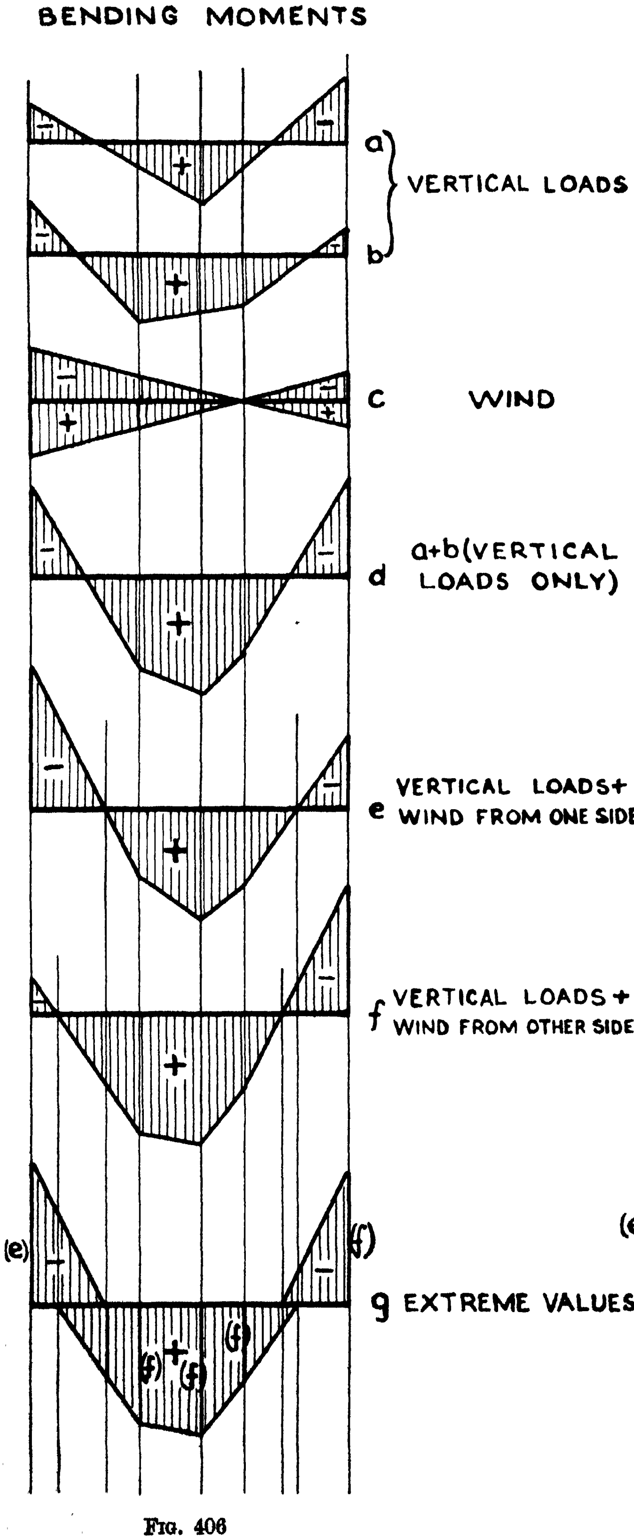


Fig. 406

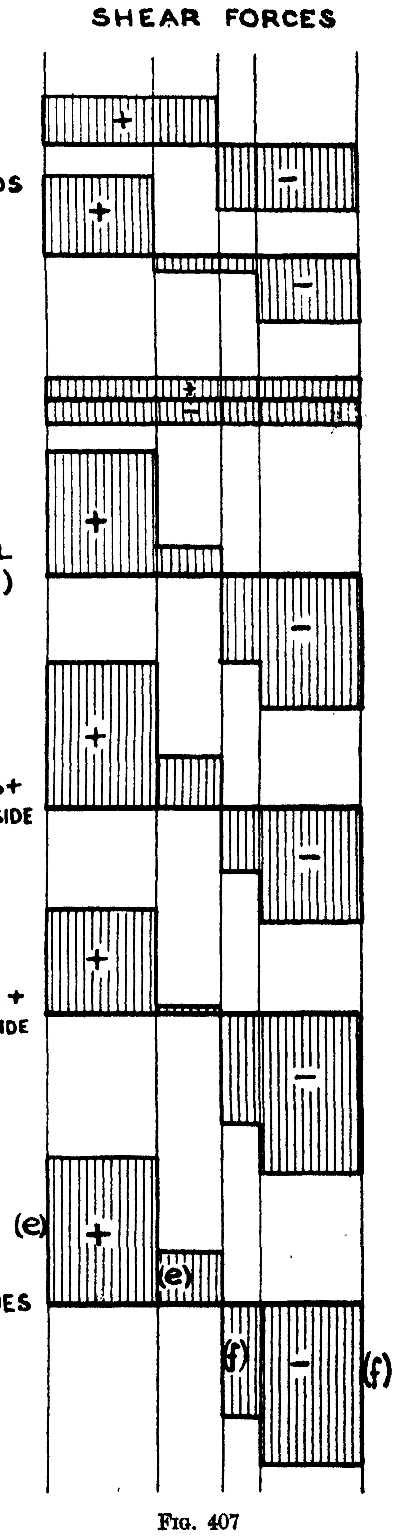


Fig. 407

From the stiffnesses of the columns, as computed on pp. 304 and 308:
$K_{u_1} = 4{\cdot}7$ in.^{3}; $K_{u_2} = 234$ in.^{3}; $K_{u_3} = 26$ in.^{3}; $K_{u_4} = 4{\cdot}7$ in.^{3}; $K_{l_1} = 5{\cdot}9$ in.^{3}; $K_{l_2} = 405$ in.^{3}; $K_{l_3} = 31$ in.^{3}; $K_{l_4} = 5{\cdot}9$ in.^{3}.
we get the following average percentages of the total wind load as being sustained by each column:
Col. 1: 1·5 per cent.; Col. 2: 88 per cent.; Col. 3: 9 per cent.; Col. 4: 1·5 per cent.
Therefore:

$$
\begin{aligned}
M_{u_1} &= 0{\cdot}015 \times 10{,}800 \times 5{\cdot}0 = 810 \text{ lb.-ft.}\\
M_{l_1} &= 0{\cdot}015 \times 13{,}500 \times 5{\cdot}0 = 1020 \text{ lb.-ft.}\\
M_{u_2} &= 0{\cdot}88 \times 10{,}800 \times 5{\cdot}0 = 47{,}600 \text{ lb.-ft.}\\
M_{l_2} &= 0{\cdot}88 \times 13{,}500 \times 5{\cdot}0 = 59{,}300 \text{ lb.-ft.}\\
M_{u_3} &= 0{\cdot}09 \times 10{,}800 \times 5{\cdot}0 = 4860 \text{ lb.-ft.}\\
M_{l_3} &= 0{\cdot}09 \times 13{,}500 \times 5{\cdot}0 = 6070 \text{ lb.-ft.}\\
M_{u_4} &= M_{u_1} = 810 \text{ lb.-ft.}\\
M_{l_4} &= M_{l_1} = 1020 \text{ lb.-ft.}
\end{aligned}
$$

All these bending moments may be either positive or negative; therefore their

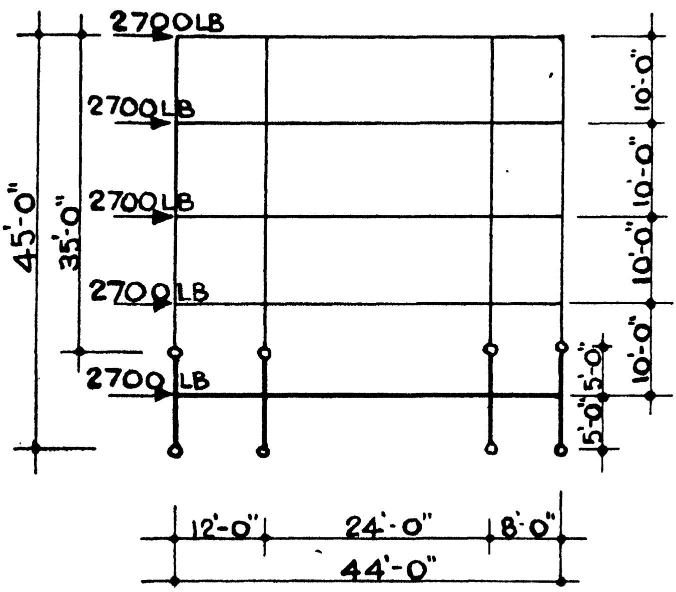


FIG. 408

numerical values will always have to be added to the bending moments due to vertical loads.

Vertical Reactions:

Overturning moment for upper columns (i.e. immediately above floor level):

$$OM_{u(total)} = 2700\ (5 + 15 + 25 + 35) = 216{,}000 \text{ lb.-ft.}$$

$$N_u = \frac{216{,}000}{44{\cdot}0} = 4{,}910 \text{ lb.}$$

Overturning moment for lower columns (i.e. immediately below floor level:

$$OM_{l(total)} = 216{,}000 + 2700 \times 45 = 337{,}500 \text{ lb.-ft.}$$

$$N_l = \frac{337{,}500}{44{\cdot}0} = 7670 \text{ lb.}$$

These forces, too, may be either positive or negative; but their signs are related to those of the bending moments in the following way:

(*a*) Wind from the left:

Bending moments in upper columns are positive.
" " " lower " " negative.

N is negative for col. 1; positive for col. 4.

(*b*) Wind from the right:

Bending moments in upper columns are negative.
" " " lower " " positive.

N is positive for col. 1; negative for col. 4.

These rules have been observed in combining the influence of wind on the columns with that of the vertical loads in the following paragraphs. (See note on page 310 for an explanation of the cases.)

Col. 1. *Case* (*a*):

$M_{u_1} = -4800 - 810 = -5610$ lb.-ft.
$M_{l_1} = +7300 + 1020 = +8320$ lb.-ft.
$R_1 = 12{,}740 + 7670 = +20{,}410$ lb.

Case (*c*):

Min. $R_1 = 12{,}740 - 7670 = +5070$ lb.
Min. $M_{u_1} = -950 + 810 = -140$ lb.-ft.
Min. $M_{l_1} = +1450 - 1020 = +430$ lb.-ft.

Col. 2. *Case* (*a*):

$M_{u_2} = -50{,}500 - 47{,}600 = -98{,}100$ lb.-ft.
$M_{l_2} = +80{,}400 + 59{,}300 = +139{,}700$ lb.-ft.
$R_2 = 37{,}700$ lb.

Case (*b*):

$R_2 = 51{,}600$ lb.
$M_{u_2} = -40{,}500 - 47{,}600 = -88{,}100$ lb.-ft.
$M_{l_2} = +63{,}000 + 59{,}300 = +122{,}300$ lb.-ft.

Case (*c*):

$R_2 = 11{,}050$ lb.
$M_{u_2} = -9700 - 47{,}600 = -57{,}300$ lb.-ft.
$M_{l_2} = +15{,}200 + 59{,}300 = +74{,}500$ lb.-ft.

Case (*d*):

$M_{u_2} = +260 + 47{,}600 = 47{,}860$ lb.-ft.
$M_{l_2} = -2440 - 59{,}300 = -61{,}740$ lb.-ft.
$R_2 = 25{,}200$ lb.

Col. 3. *Case* (*a*):

$M_{u_3} = +22{,}900 + 4860 = +27{,}760$ lb.-ft.
$M_{l_3} = -25{,}200 - 6070 = -31{,}270$ lb.-ft.
$R_3 = 33{,}500$ lb.

Case (b):

$R_3 = 42{,}600$ lb.

$M_{u_3} = +20{,}900 + 4860 = +25{,}460$ lb.-ft.

$M_{l_3} = -21{,}800 - 6070 = -27{,}870$ lb.-ft.

Case (c):

$R_3 = 9400$ lb.

$M_{u_3} = +4850 + 4860 = +9710$ lb.-ft.

$M_{l_3} = -5150 - 6070 = -11{,}220$ lb.-ft.

Case (d):

$M_{u_3} = +2650 - 4860 = -2210$ lb.-ft.

$M_{l_3} = -1750 + 6070 = +4320$ lb.-ft.

$R_3 = 18{,}500$ lb.

Col. 4. *Case (a):*

$M_{u_4} = +1700 + 810 = +2510$ lb.-ft.

$M_{l_4} = -1830 - 1020 = -2850$ lb.-ft.

$R_4 = 7980 + 7670 = 15{,}650$ lb.

Case (c):

$R_4 = -1200 - 7670 = -8870$ lb. (uplift).

$M_{u_4} = -900 - 810 = -1710$ lb.-ft.

$M_{l_4} = +1370 + 1020 = +2390$ lb.-ft.

Beam moments due to wind:

$S = N_l - N_u = 7670 - 4910 = 2760$ lb.

$M_{R_1} = M_{u_1} - M_{l_1} = 810 + 1020 = 1830$ lb.-ft.

$M_{L_2} = 1830 - 2760 \times 12{\cdot}0 = -31{,}290$ lb.-ft.

$M_{R_2} = -31{,}290 + (47{,}600 + 59{,}300) = +75{,}610$ lb.-ft.

$M_{L_3} = 75{,}610 - 2760 \times 24{\cdot}0 = +9370$ lb.-ft.

$M_{R_3} = +9370 + (4860 + 6070) = +20{,}300$ lb.-ft.

$M_{L_4} = 20{,}300 - 2760 \times 8 = -1780$ lb.-ft.

The last value is nearly enough equal to $-(M_{u_4} - M_{l_4}) = -1830$ lb.-ft., (page 318) and it is a check on the correctness of the numerical calculation.

The above bending moments in the beams, again, are either positive or negative; only for the sake of simplicity has one direction of wind been given preference whilst determining the numerical values.

In Figs. 396 and 399 the wind load bending moment and shear distributions are given and in Figs. 397 and 400 these have been combined with the vertical load bending moments and shears.

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