Structural appraisal of iron-framed textile mills

ICE design and practice guides

One of the major aims of the Institution of Civil Engineers is to provide its members with opportunities for continuing professional development. One method by which the Institution is achieving this is the production of design and practice guides on topics relevant to the professional activities of its members. The purpose of the guides is to provide an introduction to the main principles and important aspects of the particular subject, and to offer guidance as to appropriate sources of more detailed information.

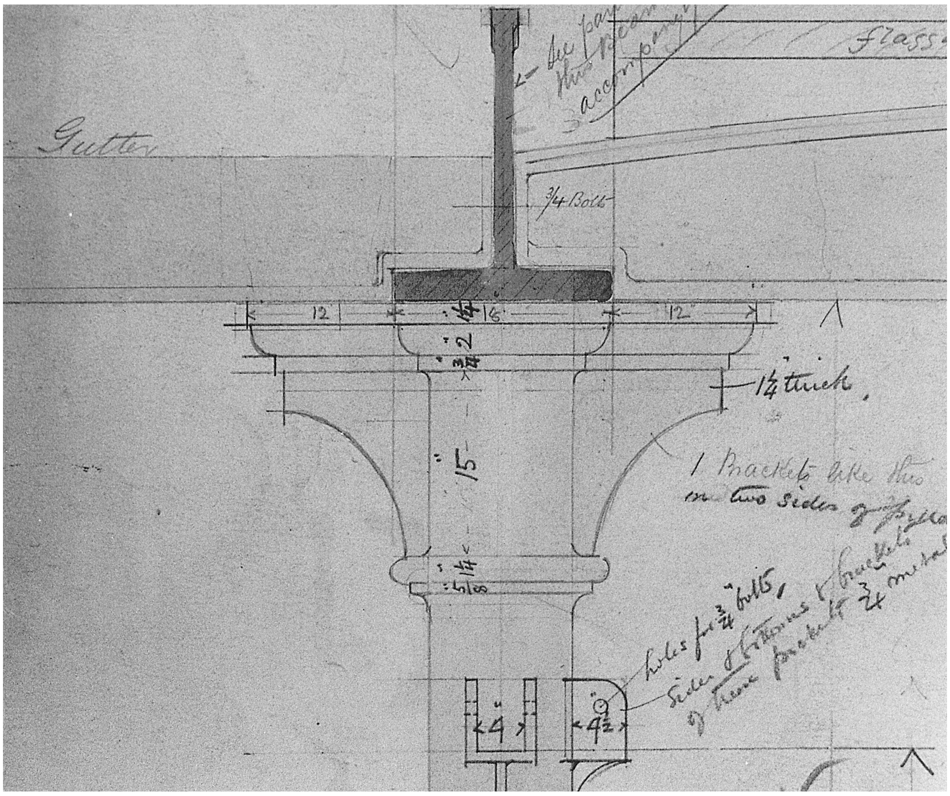
The Institution has targeted as its principal audience practising civil engineers who are not expert in or familiar with the subject matter. This group includes recently graduated engineers who are undergoing their professional training and more experienced engineers whose work experience has not previously led them into the subject area in any detail. Those professionals who are more familiar with the subject may also find the guides of value as a handy overview or summary of the principal issues.

Where appropriate, the guides will feature checklists to be used as an *aide-mémoire* on major aspects of the subject and will provide, through references and bibliographies, guidance on authoritative, relevant and up-to-date published documents to which reference should be made for reliable and more detailed guidance.

ICE design and practice guide

Structural appraisal of iron-framed textile mills

Tom Swailes and Joe Marsh



DETR
Environment
Transport
Regions

UMIST HISTORIC STRUCTURES GROUP

Thomas Telford

Published by Thomas Telford Publishing, Thomas Telford Ltd, 1 Heron Quay, London E14 4JD
http://www.t-telford.co.uk

First published 1998

Distributors for Thomas Telford books are
USA: ASCE Press, 1801 Alexander Bell Drive, Reston, VA 20191-4400, USA
Japan: Maruzen Co. Ltd, Book Department, 3–10 Nihonbashi 2-chome, Chuo-ku, Tokyo 103
Australia: DA Books and Journals, 648 Whitehorse Road, Mitcham 3132, Victoria

A catalogue record for this book is available from the British Library

Front cover: Manningham Mills, Bradford, c. 1871. From Keighley, M. 'A Fabric Huge—The Story of the Listers'. James & James. © Lister & co. 1989, reproduced with permission.
Frontispiece: Manningham Mills [detail]. Drawings for ironwork to extensions in the 1880s, with comments added by the ironfounder. Drawings in the possession of M. Robinson, Ove Arup & Partners. Reproduced with permission.
Illustrations are by the authors except where otherwise indicated.

Classification
Availability: Unrestricted
Content: Recommendations based on current practice
Status: Refereed
User: Practising civil engineers and designers

ISBN: 0 7277 2713 3

Typeset by Gray Publishing, Tunbridge Wells, Kent
Printed in Great Britain by the Cromwell Press, Trowbridge, Wiltshire

Acknowledgements

This design and practice guide has been prepared for the Structural and Building Board of the Institution of Civil Engineers and for the Department of the Environment, Transport and the Regions (DETR) by members of the UMIST Historic Structures Group. Joe Marsh is the main author of Chapter 2 of the guide and Tom Swailes is the main author of the remainder. Stuart Millns of AA Associates has written parts of Chapter 4 and, like Ron Fitzgerald of Structural Perspectives, has provided many captioned illustrations and has been a constant source of information and comment.

The guide draws upon the results of site-based research work supported by DETR, English Heritage and Dean Clough Industrial Park Ltd, through the DETR Partners in Technology programme. Research at UMIST on the strength of nineteenth-century cast iron has been supported by the Engineering and Physical Sciences Research Council and by English Heritage. The Institution of Civil Engineers Research and Development Enabling Fund has provided support towards archive research and towards one of the site testing projects. Thanks are due to the following individuals for comments on drafts of the guide that have enabled inaccuracies to be corrected and many improvements to be made:

Adrian Bell	Department of Civil and Structural Engineering, UMIST
Michael Bussell	Consulting Engineer, formerly with Ove Arup & Partners
Mike Chrimes	The Institution of Civil Engineers
Brian Clancy	Brian Clancy Partnership
Geoff Harding	DETR
Ian Hume	Consulting Engineer, formerly with English Heritage
Lawrance Hurst	Hurst, Peirce & Malcolm
Frank Kelsall	English Heritage
Clive Melbourne	The Institution of Civil Engineers
John Menzies	Consulting Engineer
John Roberts	Allott & Lomax
James Sutherland	Consultant, Harris & Sutherland
Bob de Vekey	BRE
Mark Watson	Historic Scotland
Bruce Walker	Historic Scotland/University of Dundee
Lothian Webster	Historic Scotland

Contents

1. Introduction

After 1926, Britain's textile industry suffered a rapid decline and thousands of industrial buildings became redundant. Often large and sometimes huge, the size of many textile mills has always been a significant obstacle to their successful conversion for new uses, and many have been demolished. In Oldham in Lancashire, for example, 180 mills were demolished between 1926 and 1976, leaving behind 140 [1]. English Heritage reported that mills were being destroyed in the Greater Manchester area at a rate of more than two per week during the property boom of the 1980s [2]. In the face of this trend, a few developments have shown what can be done, with imagination, to make very successful conversions of individual mills and groups of mills [3]. In some cases, structural engineering uncertainties have been a barrier to the reuse of mill buildings. The aim of the research that has led to this design and practice guide has been to remove some of these uncertainties. The guide is intended primarily for civil or structural engineers who may be concerned with the appraisal or structural alteration of textile mills. However, it is hoped that it may contain something for others who are interested in these important buildings.

For 150 years, Britain's textile industry was the cornerstone of the nation's economy and literature on the subject is very considerable. Chapter 2 of this design and practice guide gives a broad overview of mill building structures and their development, but the engineer will find useful material elsewhere. Modern texts include chapters on textiles from the perspectives of 'the history of technology', 'industrial archaeology' and 'industrial England' [4–6]. Many mills have been surveyed and recorded by industrial archaeologists and two issues of the journal of the Association for Industrial Archaeology in 1988 and 1993 have been devoted to textile mills [7].

For regional accounts of the textile industry in West Yorkshire, East Cheshire and Greater Manchester, and for detailed accounts of individual mills and mill groupings, the reader is referred to three volumes published in 1992 and 1993 by the Royal Commission on the Historical Monuments of England [8–10]. In these regions, the predominant manufacture was of wool (West Yorkshire), silk (East Cheshire) and cotton (Greater Manchester) and concise accounts of these three sectors of the industry are to be found in recent *Shire Albums* [11]. Several other regions, towns and cities that owed their growth and prosperity to textiles have their own histories [12, 13], as have the works of inventors and textile entrepreneurs such as Richard Arkwright.

Progress in mill building construction is presented in Figure 1.1 alongside the principal developments in textile machinery, in power systems and in the iron and steel industries [14, 15]. Illustrations of many examples of mill construction details are to be found in Chapter 2 of this design and practice guide.

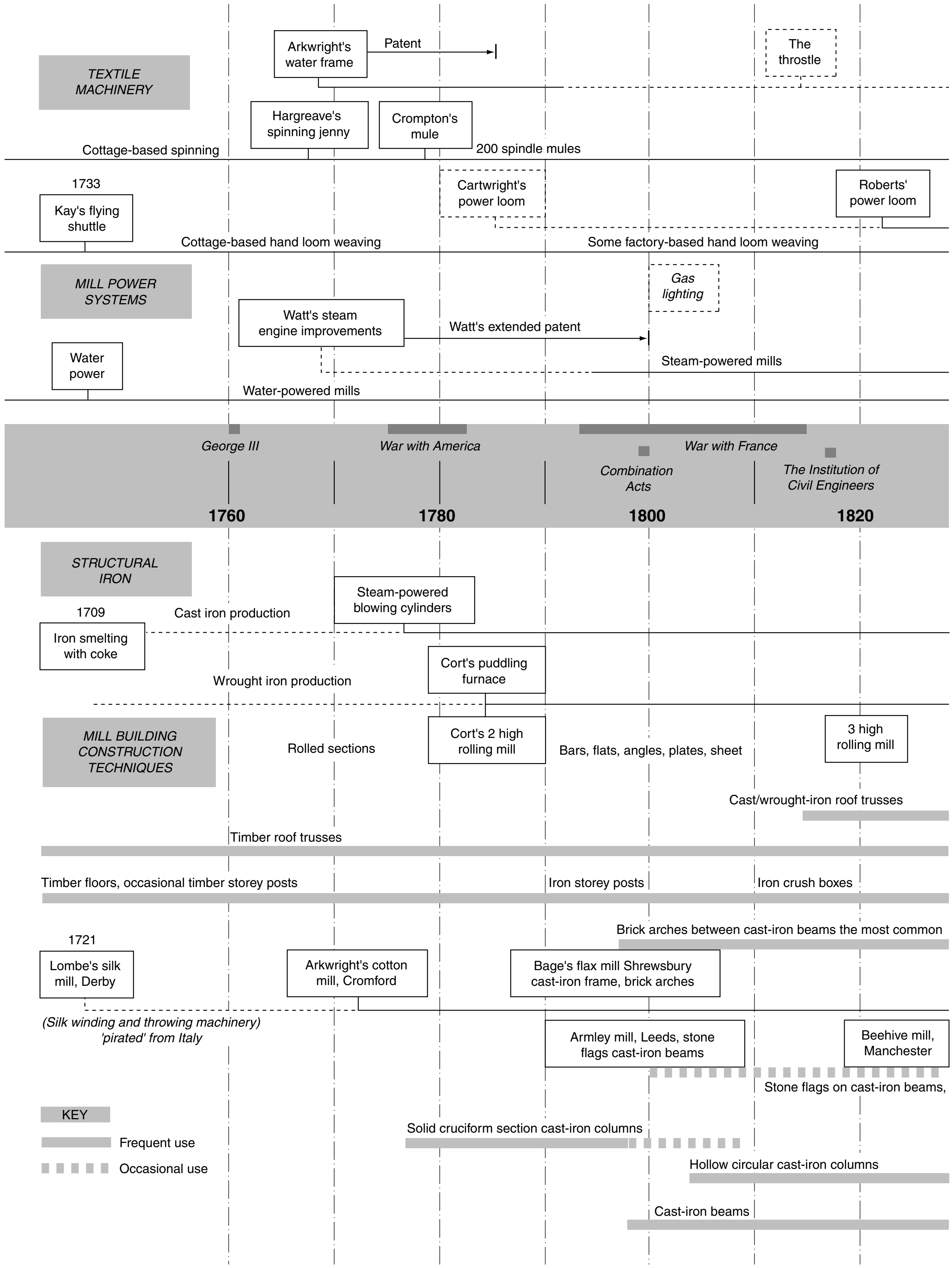
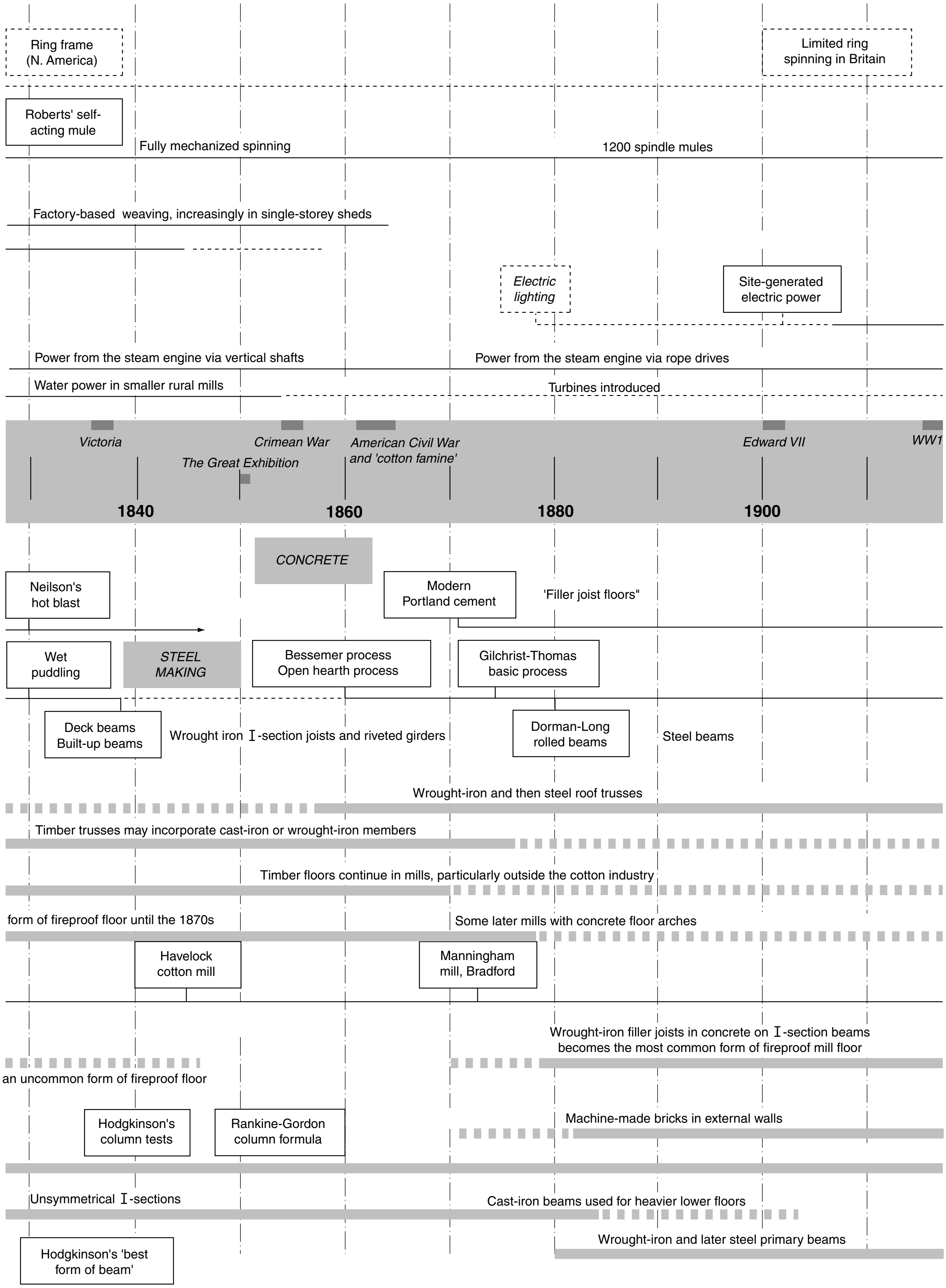

TEXTILE MACHINERY
Arkwright's water frame
Patent
The throstle
Hargreave's spinning jenny
Crompton's mule
Cottage-based spinning
200 spindle mules
1733
Kay's flying shuttle
Cartwright's power loom
Roberts' power loom
Cottage-based hand loom weaving
Some factory-based hand loom weaving
MILL POWER SYSTEMS
Gas lighting
Watt's steam engine improvements
Watt's extended patent
Water power
Steam-powered mills
Water-powered mills
George III
War with America
War with France
Combination Acts
The Institution of Civil Engineers
1760
1780
1800
1820
STRUCTURAL IRON
Steam-powered blowing cylinders
1709
Cast iron production
Iron smelting with coke
Cort's puddling furnace
Wrought iron production
MILL BUILDING CONSTRUCTION TECHNIQUES
Rolled sections
Cort's 2 high rolling mill
Bars, flats, angles, plates, sheet
3 high rolling mill
Cast/wrought-iron roof trusses
Timber roof trusses
Timber floors, occasional timber storey posts
Iron storey posts
Iron crush boxes
Brick arches between cast-iron beams the most common
1721
Lombe's silk mill, Derby
Arkwright's cotton mill, Cromford
Bage's flax mill Shrewsbury cast-iron frame, brick arches
(Silk winding and throwing machinery) 'pirated' from Italy
Armley mill, Leeds, stone flags cast-iron beams
Beehive mill, Manchester
Stone flags on cast-iron beams,
KEY
Frequent use
Occasional use
Solid cruciform section cast-iron columns
Hollow circular cast-iron columns
Cast-iron beams

Figure 1.1


Ring frame (N. America)
Limited ring spinning in Britain
Roberts' self-acting mule
Fully mechanized spinning
1200 spindle mules
Factory-based weaving, increasingly in single-storey sheds
Electric lighting
Site-generated electric power
Power from the steam engine via vertical shafts
Power from the steam engine via rope drives
Water power in smaller rural mills
Turbines introduced
Victoria
Crimean War
American Civil War and 'cotton famine'
Edward VII
WW1
The Great Exhibition
1840
1860
1880
1900
CONCRETE
Neilson's hot blast
Modern Portland cement
'Filler joist floors"
Wet puddling
STEEL MAKING
Bessemer process Open hearth process
Gilchrist-Thomas basic process
Deck beams Built-up beams
Wrought iron I-section joists and riveted girders
Dorman-Long rolled beams
Steel beams
Wrought-iron and then steel roof trusses
Timber trusses may incorporate cast-iron or wrought-iron members
Timber floors continue in mills, particularly outside the cotton industry
form of fireproof floor until the 1870s
Some later mills with concrete floor arches
Havelock cotton mill
Manningham mill, Bradford
Wrought-iron filler joists in concrete on I-section beams becomes the most common form of fireproof mill floor
an uncommon form of fireproof floor
Hodgkinson's column tests
Rankine-Gordon column formula
Machine-made bricks in external walls
Unsymmetrical I-sections
Cast-iron beams used for heavier lower floors
Wrought-iron and later steel primary beams
Hodgkinson's 'best form of beam'

The simplified presentation of Figure 1.1 ignores the considerable variations both between and within regions. In the cotton industry of late nineteenth-century Lancashire for example, Oldham and Bolton specialized in spinning. In the weaving centres of Blackburn and Burnley to the north, single-storey weaving sheds, without the characteristic multi-storey mill alongside, were common.

Nineteenth-century mill buildings invariably have internal framing only—of timber, iron or steel—within fairly massive stone or brickwork masonry external walls. However, they are often tall, multi-storey buildings. Engineers involved in the structural appraisal of mills from this era have had difficulty in demonstrating that these partially framed buildings can satisfy modern structural requirements for tall buildings. Chapter 3 of this guide provides the background to the modern Building Regulations requirements for buildings of five storeys or more [16]. The ability to comply with these requirements is discussed.

The Building Regulations are likely to be applicable when an old building is to be converted for a new use. Historical and modern evidence is presented in Chapter 3 to show that the robustness of these old buildings must be assessed carefully. Guidance is given on simple measures that can be taken to improve robustness, where necessary.

Good general guidance is already available in a number of existing publications on the survey, inspection, structural appraisal and renovation of existing buildings [17–20]. Reference must also be made to the British Standard codes of practice for loadings for buildings and for the structural use of unreinforced masonry [21, 22]. Chapter 4 of this design and practice guide deals specifically with the constructional features and structural problems found in textile mills. Cast iron as a structural material is dealt with in some detail in Chapter 4 as it was used very extensively for beams and columns in mills until the latter part of the nineteenth century. Unlike masonry and timber, cast iron is an obsolete structural material for which no modern code of practice exists and with which very few engineers are familiar. In the structural appraisal of an iron-framed mill, the weakest element often appears to be the cast-iron floor beams. Research has shown that permissible stresses, which in recent years have been in common use for cast iron in building structures, are rather conservative [23]. Further evidence of this is presented in Chapter 4 of this design and practice guide, in which it is recommended that the structural assessment of cast-iron beams and columns in mill buildings should follow the procedures normally adopted for highway bridges [24].

A summary diagram at the beginning of Chapter 5 shows several measures that may be needed to ensure adequate strength and robustness in an iron-framed mill. This is followed by a worked example, with a commentary, that includes details of a proven method of strengthening weak cast-iron beams supporting a brickwork arch floor. The purpose of the example is to illustrate how some of the principles described in the preceding sections of the guide may be applied. Outline calculations are included for the capacity of the main structural elements under normal and accidental loadings. The calculations are not exhaustive and do not deal with connections or detailed methods of strengthening, for example. Further guidance on the structural appraisal, repair and strengthening of cast iron, wrought iron and steel is given in a recent Steel Construction Institute publication [25].

Imperial units are widely used in parts of this guide because the iron framing of mills is generally modular and repetitive, very often with bay sizes in multiples of a foot (ft). Member section sizes may similarly be in inches (in.) and fractions of an inch. There are 12 in. per ft and 25.4 mm per in.

2. Structural forms

The origins of the fireproof building

Prior to the industrial revolution, most manufacturing took place either in the home or out in the open. Almost the only large scale industrial buildings were storage warehouses, malthouses and breweries. Brick or stone load-bearing walls supported timber floors with intermediate timber stanchions supporting the floor joists in the wider buildings. Warehouses could be subject to very high floor loads and timbers of up to 18-in. square section were not uncommon (Figure 2.1).

The central development during the industrial revolution was that of the factory system. The invention of spinning machinery led to the movement of the textile industry out of the home into mass production factories that became known as textile mills (Figures 2.2 and 2.3). In the early multi-storey spinning mills, power generated by a waterwheel was distributed to large numbers of relatively lightweight machines (Figure 2.4, [26]). Mechanization of the several stages in the conversion of the raw material to yarn came early but efficient machines for weaving the yarn came much later, in the 1820s. The classic handweaver's cottage had a third storey lit by long windows but sometimes weaving would be carried out under supervision in purpose-built loomshops on several floors. The later heavy power looms were often housed in separate single-storey weaving sheds lit via roof glazing.

The lightweight spinning machines resulted in low floor loads compared with those in warehouses. Therefore, timbers of smaller section could be used for the structural

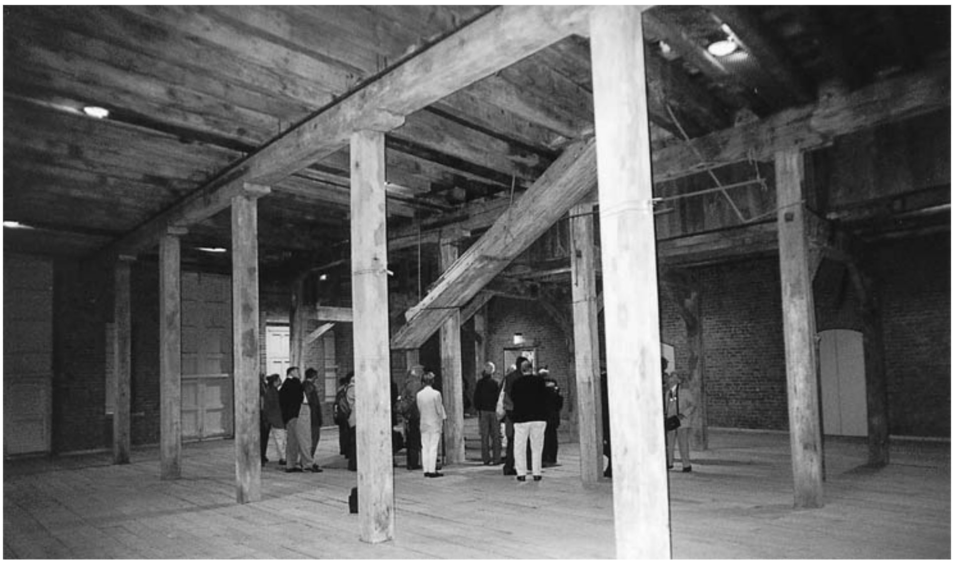


Figure 2.1 Liverpool Road Railway Station warehouse, Manchester. Built in 1830 and now a grade I listed building.

Figure 2.2 Bell Mill, Stanley, near Perth. Built c. 1787, a brick-built mill that exemplifies the long narrow floor plan of early Arkwright-type mills.

Figure 2.3 Higher Mill, Helmshore. Although built in 1790, the stone exterior, small windows and low rise of this mill are typical of pre industrial revolution water-powered mills.

framing. However, processing of cotton and flax by machine led to serious fire risks and the early entrepreneurs and investors soon began to look for 'fireproof' building materials and structures.

Cast iron, which was beginning to be available in large quantities in the 1780s, appeared to answer the problem. Cast-iron columns soon replaced timber stanchions even where there was little fire risk (such as in churches). Although cast iron is very strong in compression, as a brittle material that may be liable to fracture under physical or thermal shock, it was not universally accepted for structural floor components. Warehouses continued to be constructed with wooden floors well into the nineteenth century and woollen mills, where the fire risk is not great, were constructed with wooden floors until the beginning of this century. However, in cotton and flax mills, and in many other industrial buildings, floors using cast-iron beams did become

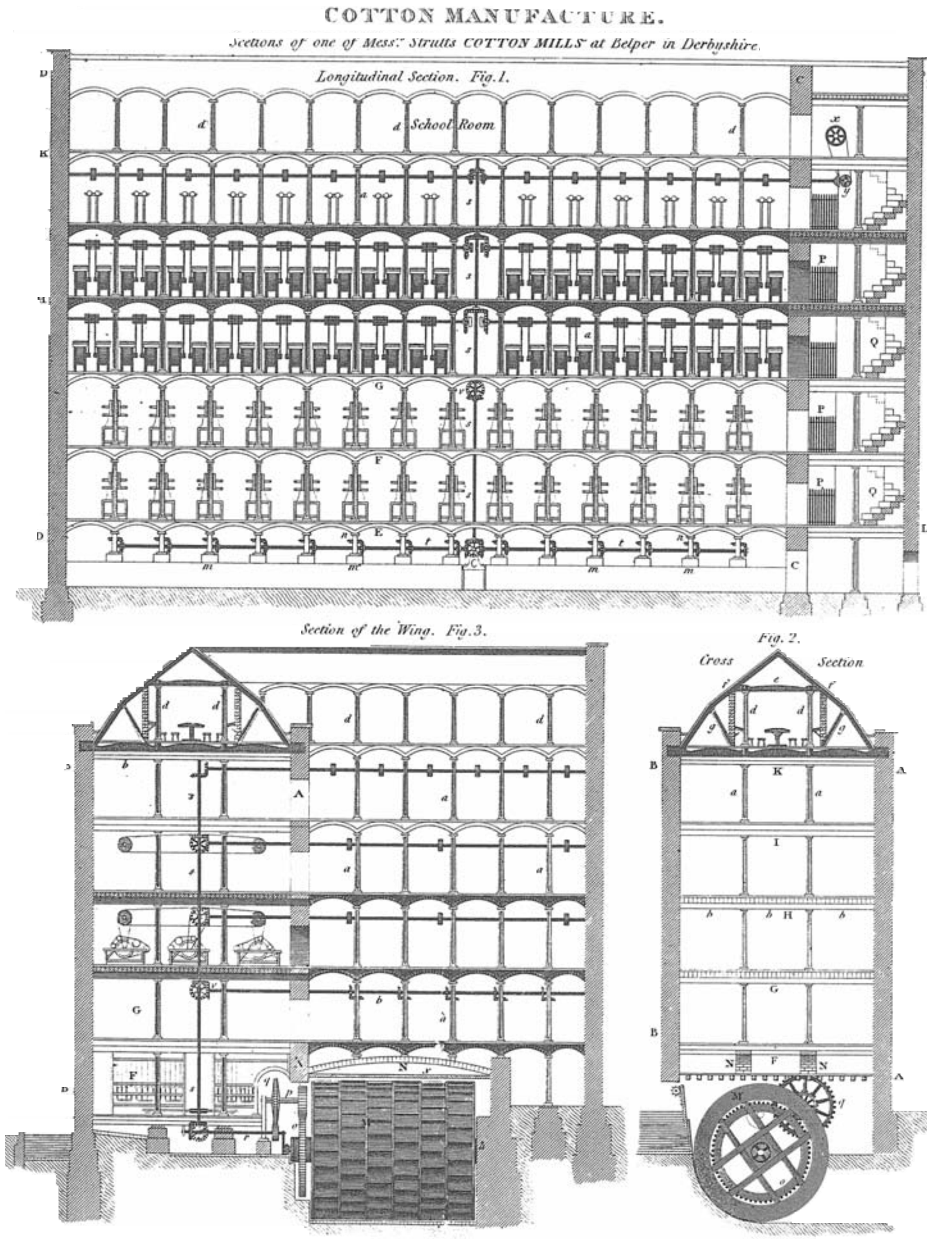


Figure 2.4 North Mill, Belper, built in 1803–4. This illustration accompanies the article on 'Cotton manufacture' in A. Rees's Cyclopaedia *of 1820.*

common. Despite some widely reported building and bridge failures in the 1840s, movement towards the use of fabricated wrought-iron beams was slow. Wrought iron, in its turn, was replaced by rolled steel towards the end of the century.

Power systems

Power for the first generation of industrial revolution textile mills was derived from the fall of water. The location of these mills depended on the availability of a suitable site. In some places, such as Styal on the River Bollin in Cheshire, hundreds of yards of engineering and mining work were necessary to harness the fall needed to enable the installation of waterwheels (Figure 2.5). Power was taken from the waterwheels through cast-iron gear wheels and shafts to each storey of the building. These vertical drive shafts drove overhead line shafts through bevel gears and the line shafts powered each machine through flat belts (Figure 2.6). In the 1780s, Newcomen-type

Figure 2.5 Quarry Bank Mill, Styal, Cheshire. Built in stages from 1784 onwards by the Greg family.

steam powered pumps were recycling water over waterwheels. Soon afterwards, James Watt's direct rotative, separate condensing steam engine began to replace waterwheels altogether as the source of power (Figure 2.7, [26]). Mills, released from their dependence on rivers and streams, were constructed in cities from the beginning of the last century and many surviving buildings show evidence of the early steam

Figure 2.6 Line shafting and bevel gears. James Walker Blanket Mill, Mirfield, Yorkshire. (Ron Fitzgerald)

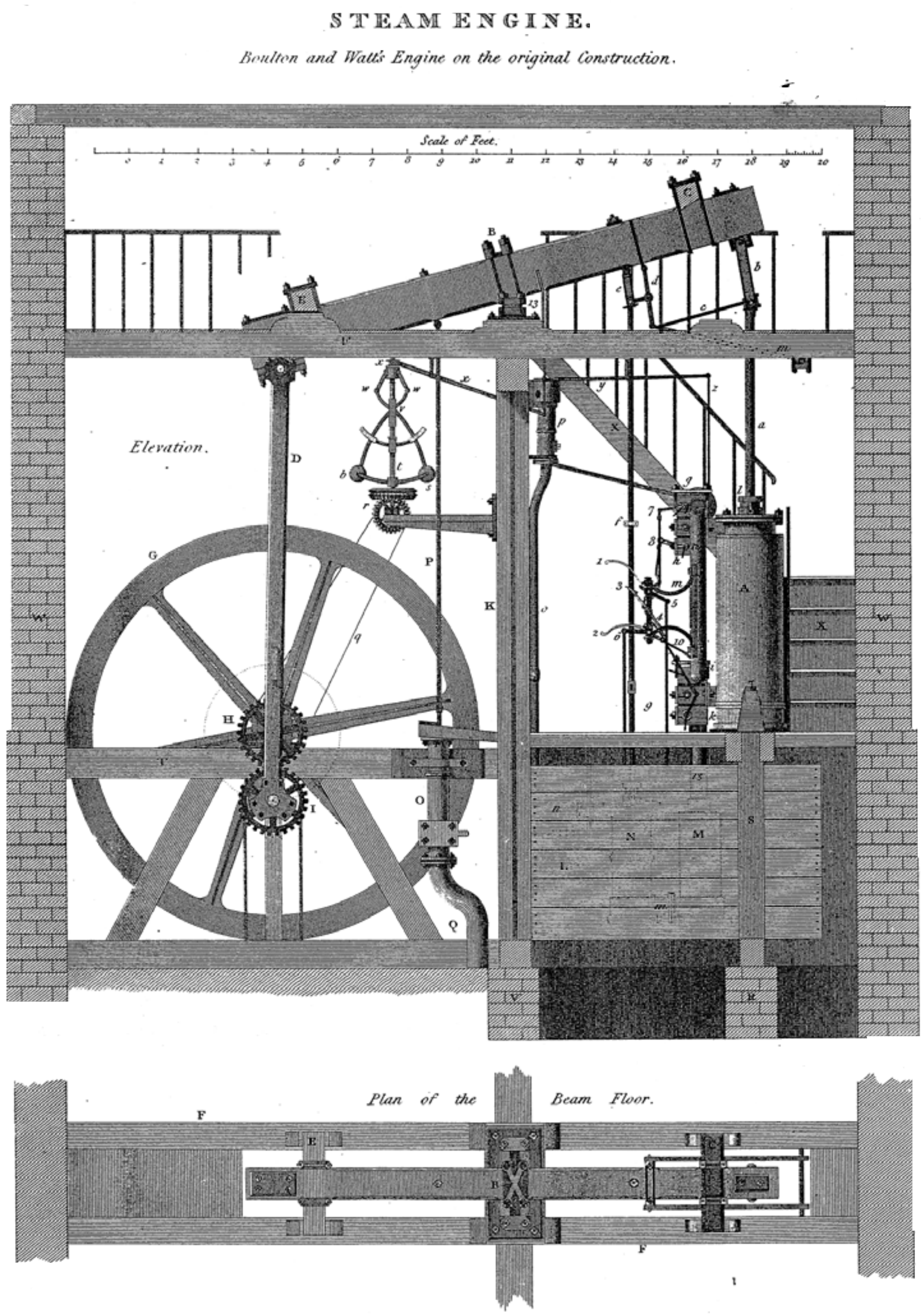


Figure 2.7 Boulton & Watt steam engine. This illustration of the pre-1800 form of James Watt's direct rotative, separate condensing steam engine accompanies the article on the 'Steam engine' in A. Rees's Cyclopaedia *of 1820.*

engines that provided their power. Tall, semicircular arch-headed windows, rising through two storeys, provide external evidence of an early engine house (Figure 2.8). Cast-iron and brick floors, in an otherwise timber-framed mill, are further evidence of the use of early steam engines.

In the second half of the nineteenth century, the Watt-style engine with vertical cylinders gave way to engines with horizontal cylinders (Figure 2.9). Vertical drive shafts were replaced by rope drives but overhead line shafts continued to distribute power to each machine through belts. In many mills that had originally been powered

Figure 2.8 Wren's Nest Mill, Glossop. Development of the site began in 1815 or earlier and the first steam engine was installed in 1829.

by a waterwheel a turbine was installed and, in a few cases, electric power was generated and distributed through cables and electric motor drives instead of the mechanical shaft and belt system.

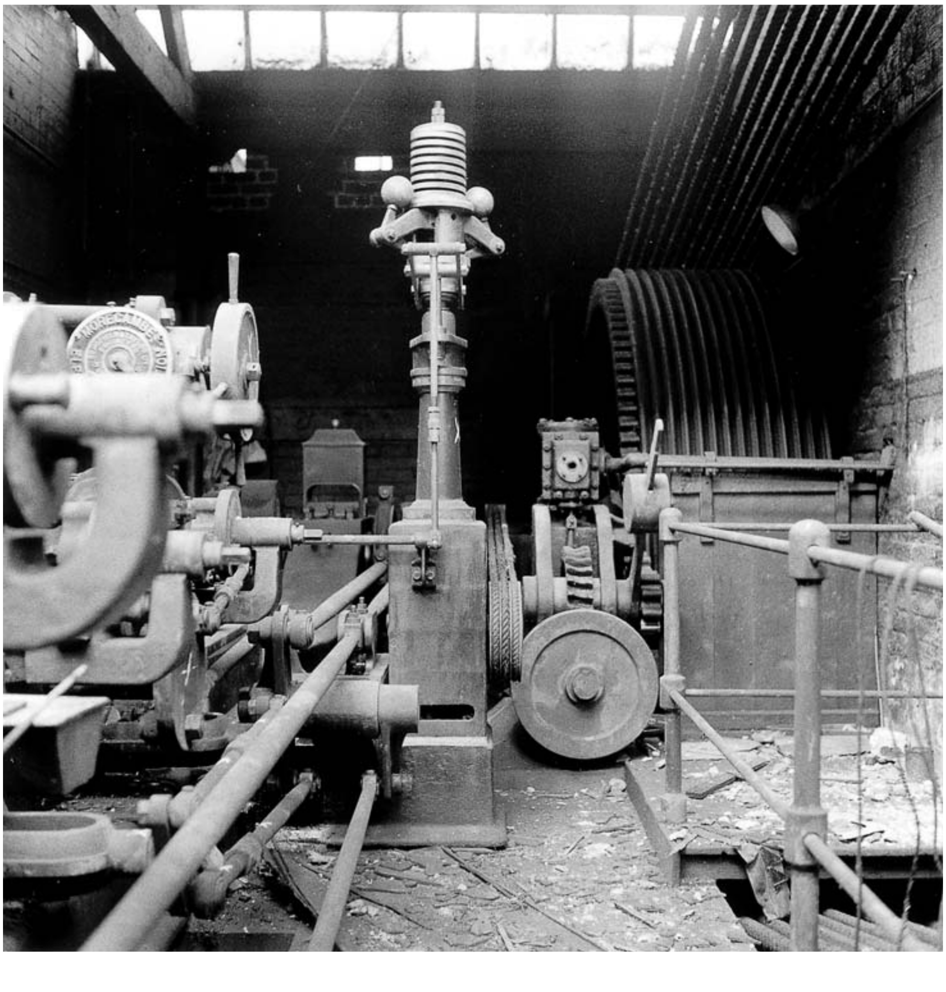


Figure 2.9 Pollit and Wigzell tandem compound steam engine and grooved flywheel for rope drive. James Walker blanket mill, Mirfield. (Ron Fitzgerald)

Figure 2.10 *(a) Stone wall section, built c. 1791. Greenup's Worsted Mill, Sowerby Bridge Mills, is five storeys high with internal timber framing. (b) Brick wall section. (Stuart Millns)*

Masonry

Foundations and walls followed previous local practice in warehouse construction. The use of stone was common in the Yorkshire Pennines, for example, whereas brick was common elsewhere. In the brick-built mills, handmade bricks were gradually replaced by bricks made by machine in the second half of the nineteenth century. The best stone-built mills used good quality stone for both inner and outer leaves and for the through-stones joining them together, with stone offcuts and poorer quality stone for the infill (Figure 2.10). In stone-built mills constructed with a low budget, the stone inner leaf was replaced by brick and the built infill replaced by rubble (Figure 2.11).

Figure 2.11 *Cavity wall with rubble infill. Greenup's Worsted Mill, as previous figure. Cavity wall construction and window elongation. Note through-stones and part throughs with loose cavity rubble. Single leaf internal brickwork returns to stone jambs at window reveals. (Stuart Millns)*

Figure 2.12 A Yorkshire industrial landscape – Dean Clough, Halifax. In the foreground is Bowling Mill, built c. 1840. The external stair and toilet towers maximize the floor space available for spinning machinery. Note the double stone lintels and good quality Yorkshire sandstone. (Stuart Millns)

Foundations for walls were often no more than corbelled brick or stone on to good ground. Foundations for columns allowed for preparation of stone seating blocks away from the construction site. Piers of brickwork were constructed on site to carry these prefabricated blocks.

External walls in textile mills were always load bearing. Initially, the window openings were relatively small. The decrease in load on the higher storeys was recognized by a progressive reduction in thickness of the wall towards the upper storeys. Typically, a 3-ft thick wall at basement level might be reduced to half that at eighth storey roof level. In the second half of the nineteenth century, mills were built wider and longer. The resulting increasing demands for light led to an increase in the height and width of windows and the use of deeper but narrower piers.

Internal framing

The first textile mills were similar to warehouse buildings with load-bearing walls supporting timber floors. With typical internal widths of 24–28 ft, large timber beams could span from wall to wall. Solid cruciform-section cast-iron 'storey posts' provided intermediate support in the wider buildings. These replaced the large timber stanchions of heavier warehouse construction. Initially, these iron posts were relatively slender and the same size from top to bottom of the building (Figure 2.13). Sometimes solid or thick walled circular-section columns, and even solid hexagonal-section columns, were used in place of the cruciform sections.

Hollow circular columns were being used instead of the earlier solid sections from around 1800. The reduced loading on upper storey columns was allowed for, either by the provision of narrower columns or by the provision of progressively thinner walls to columns of constant outside diameter. The top of each column was often cast as a socket to receive the lower end of the column above. In the earlier buildings, lead packing was used to ensure an even seating for column bases in the sockets. By the middle of the century, the column ends could be machined but lead packing was still being used. Flat-bolting faces and local thickenings for line shafting support brackets

Figure 2.13 Cruciform column. in the 1797 Marshall, Benyon & Bage Flax Mill, Shrewsbury.

are a characteristic feature of hollow circular textile mill columns. Occasionally, the local framing arrangements for shaft bearing supports on the column are more elaborate (Figure 2.14). In some cases, line shafting was suspended from the beam soffits via brackets, either bolted or clamped to the bottom flange.

Where timber beams were used, the connection system was modified to allow for continuity of the beams over the top of the columns to avoid crushing of the timber. At first, a simple double bracket was added to column capitals to increase the area of support for the beam. Sides added to the brackets could be extended to the tops of the

Figure 2.14 (a) Provisions for line shaft support – modified cruciform columns in the 1797 Marshall, Benyon & Bage Flax Mill, Shrewsbury. (b) Line shaft support brackets fixed to bolting faces on a hollow circular cast-iron column. James Walker blanket mill, Mirfield. (Ron Fitzgerald)

supported beams at which point a cast-iron plate would complete the 'boxing in' of the beam and act as a load-spreading support for the base of the next column (Figure 2.15). The most common version of this system comprised column capitals with flat-topped double brackets which supported both the beam and an inverted U-shaped cast-iron saddle or 'crush box'. The crush box was provided with small studs so that it could be located in sockets on the double bracket and with a short spigot on the top surface to locate and restrain the base of the next column.

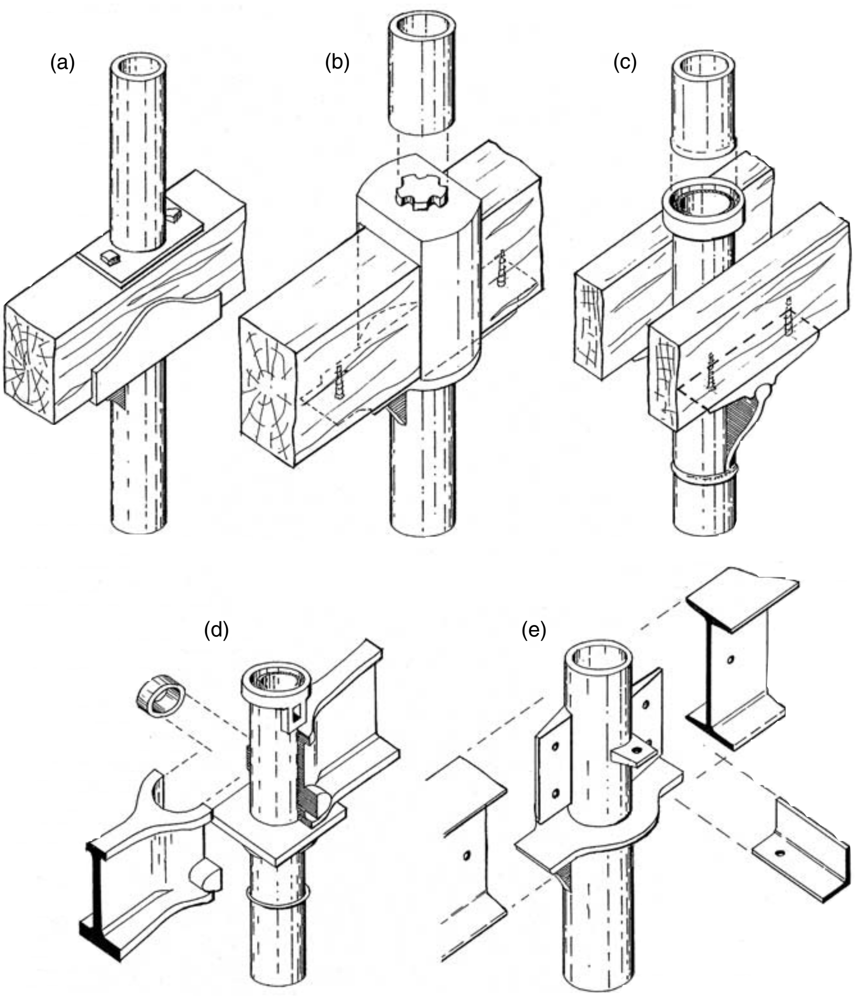


Figure 2.15 Beam to cast-iron column connections. (a) Column load transferred through timber beam. (b) Wren's Nest Mill, Glossop, timber beam through cast-iron saddle (c. 1850s); (c) Stanley Mills, Perthshire, twin timber beams on loose rockers on brackets cast either side of the column (c. 1850s); (d) Havelock Cotton Mill, Manchester (1845), cast-iron beam ends wrapped around the column. (e) Butterworth Hall Mill (c. 1900) steel beams on projecting column end plate.

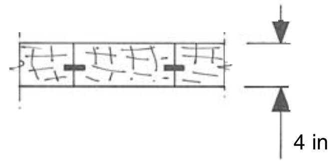
In the last third of the nineteenth century, fabricated wrought-iron stanchions were used for the support of heavy loads as an alternative to large hollow-section circular cast-iron columns, particularly in warehouses. Large section cruciform or I-section cast-iron stanchions were occasionally used in similar circumstances. In the 1880s fabricated steel stanchions began to replace wrought iron and, after the turn of the century, rolled steel I-section stanchions were becoming common in buildings other than textile mills. However, the specialist mill designers and builders were slow to embrace the steel frame and retained a preference for the hollow circular cast-iron column into the 1920s.

Floor systems up to 1850

Although the hollow circular cast-iron column had become the standard system for intermediate support, there was no one standard floor system. In fact, three main systems existed side by side for the first half of the century and were then in competition with several 'patent' and non-patent systems in the second half of the century. Timber floors with various forms of fire protection continued to be used throughout the century. Alongside timber floors, two forms of cast-iron-framed floor were developed during the 1790s and one of these remained dominant until the 1870s.

Fireproofing of timber floors took a variety of forms. Early on, while large section timber was still relatively cheap, 4-in. thick floorboards were fixed directly over the principal timber beams. The thickness of the boards themselves provided some delay to the spread of fire and this delay was increased by the insertion of continuous wrought-iron tongues between the boards (Figure 2.16a). A domestic-style lath and plaster ceiling fixed to ceiling joists at the soffit of the main beams further improved fire resistance but at high cost. As an alternative to lath and plaster, rolled sheet wrought iron was used to cover and protect beams and the underside of floors (Figure 2.16b). This system continued to be used well into the twentieth century.

(a)



(b)

Figure 2.16 (a) Wrought-iron tongues between thick floorboards (e.g. Havelock Silk Mill, Manchester, c. 1830). (b) Wrought iron sheet fire protection to a timber beam and joist floor in the 1855 Canal Mill, Botany Bay, Chorley.

In the first cast-iron floor system, inverted T-section cast-iron beams were used to support a shallow brick arch (Figures 2.17 and 2.18). The top of the arch was levelled with cinders and cinder lime concrete; stone flags or timber floorboards were laid on top. From the outset, the main disadvantage of this type of 'fireproof' floor was the very high self-weight. The plan form of many of the early iron-framed mill buildings was very similar to that of their timber floored predecessors, except that across a width of 28 ft, two intermediate cast-iron columns might now be required to support the heavy floors. Until the 1820s, designers were reluctant to use the rather crude inverted T-section cast-iron beams for spans beyond 14 ft. A typical cotton spinning mill from this early period of iron framing might be three bays wide, with internal dimensions of 42 ft by 110 ft and of six storeys or more in height.

A second disadvantage of the arch floor system is that the gable walls must withstand the weight of the end arch thrusts. This problem could be solved to a certain extent by replacing the last arch with a series of transverse arches carried on secondary beams between the primary beam and the end wall (Figure 2.19). This system of carrying arches on secondary beams was further developed in the mid-nineteenth century, the reduced floor weight making greater spans possible.

Surviving examples of the second cast-iron floor system are rare. Lighter in weight than the brick arch system, a grid of inter-locking primary, secondary (and sometimes tertiary) T-section cast-iron beams support stone slabs on their top flanges

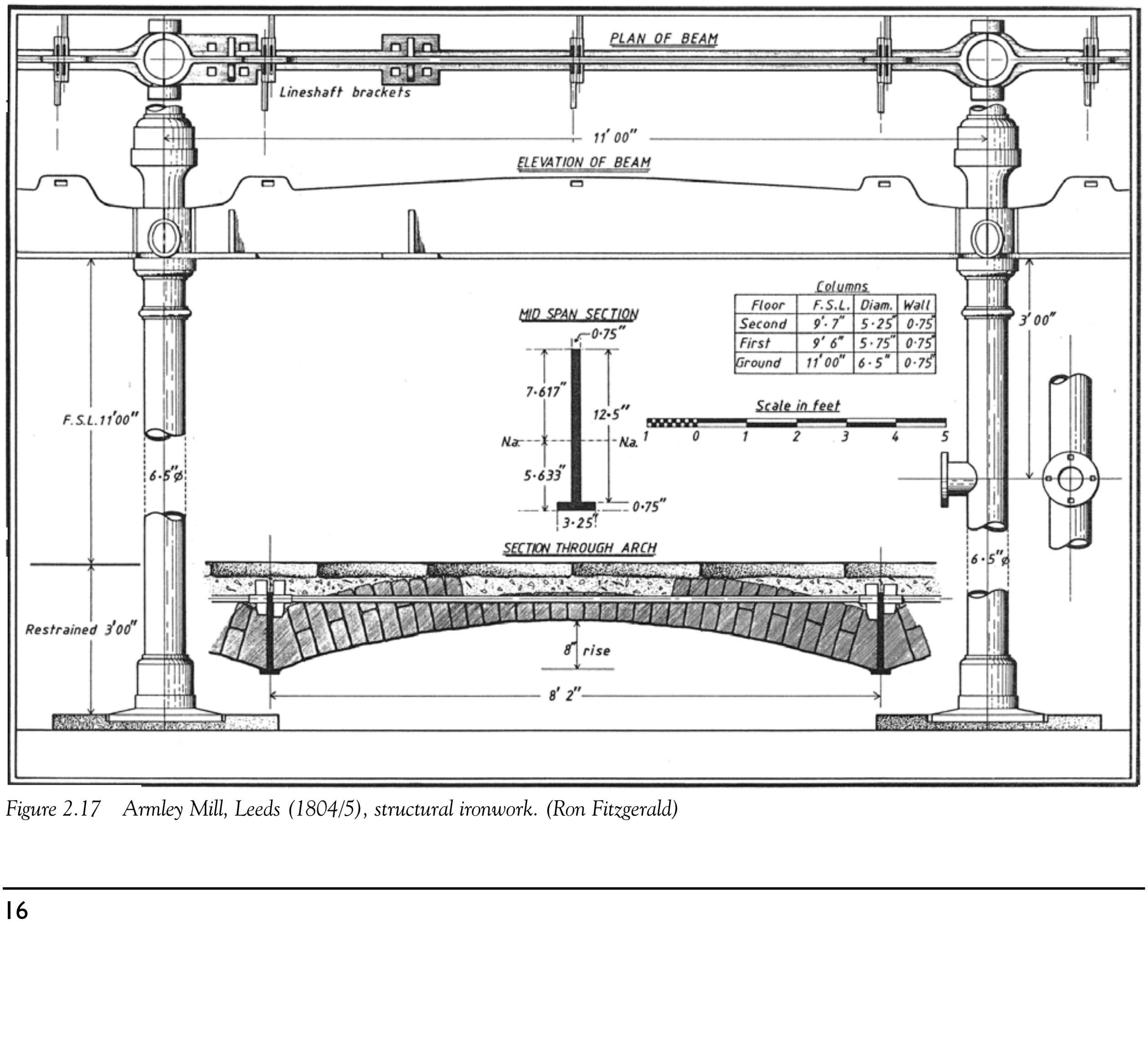


Figure 2.17 Armley Mill, Leeds (1804/5), structural ironwork. (Ron Fitzgerald)

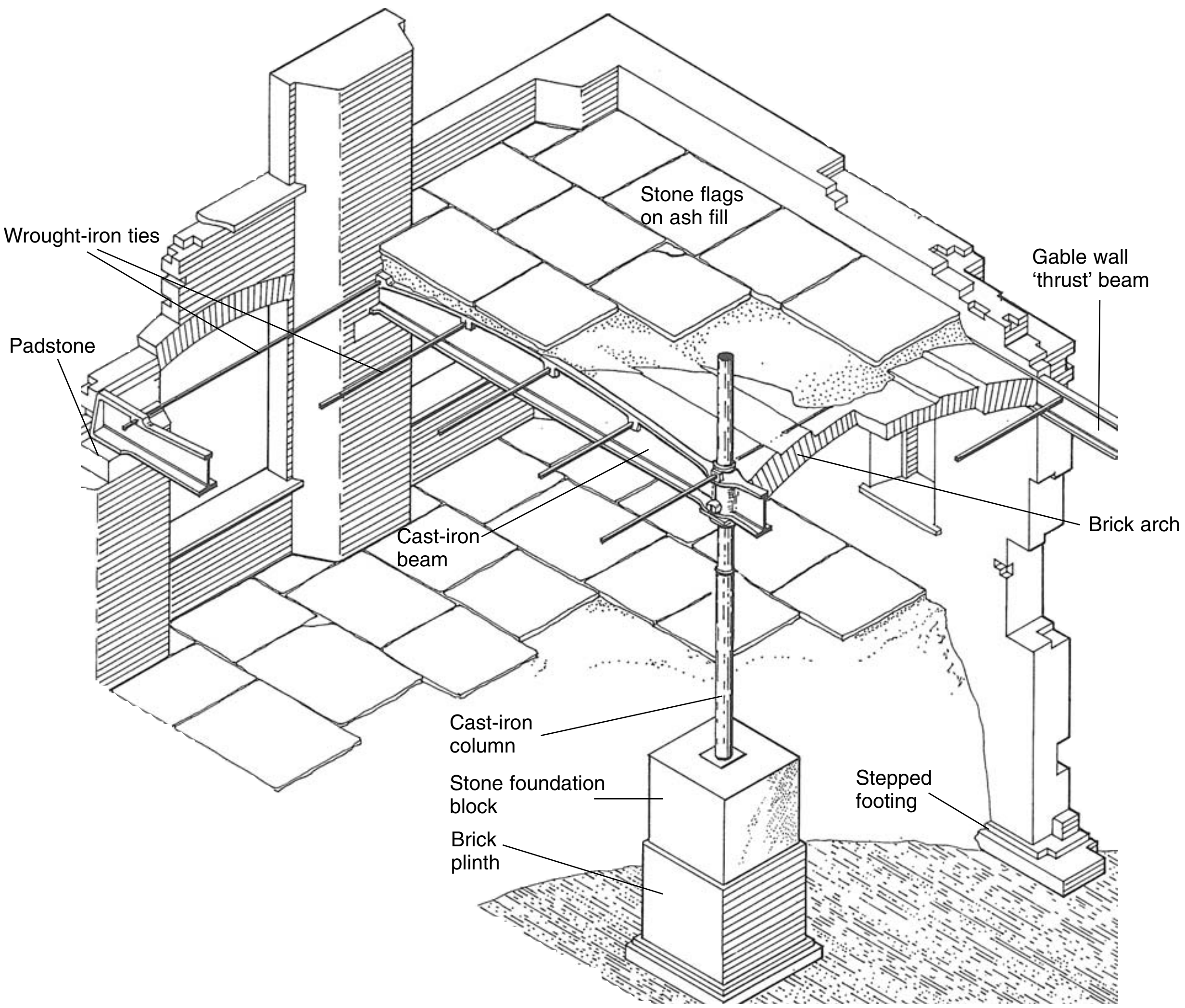


Figure 2.18 Havelock Cotton Mill, Manchester (1845), iron framing.

(Figures 2.20 and 2.21). The system was vulnerable to fire because the whole of the ironwork in the grid was exposed. It was therefore only used in small sections of buildings, such as over boiler rooms.

In both cast-iron floor systems the increased bending moment at centre span was recognized by designing the beams with an increased web depth at centre span. This gave the inverted beams a 'hog back' and the upright beams a 'fish belly'. The early T-section beams gave way to I-sections during the 1820s. Experiments by William Fairbairn and then Eaton Hodgkinson in Manchester established 'the best form of beam', with larger bottom flanges the norm in mills from the 1830s onwards (Figure 2.22). Both top and bottom flanges were often narrower towards the beam ends and wider at mid-span. The ideal theoretical proportion of the cross-sectional area of the lower to upper flange was determined by Hodgkinson as six to one but, in practice, for beams in buildings, a lower ratio was often adopted. From the 1830s Fairbairn pioneered the use of the improved form of beam for spans of up to 25 ft, enabling a width of 50 ft to be interrupted by only a single line of columns.

Figure 2.19 Manningham Mills, Bradford (1871). End bay showing change of direction to cast-iron beam and jack arch floor. Note the good quality of the through stones, lintels, cills and padstone into backing brickwork. (Stuart Millns)

Figure 2.20 Beehive Mill, Manchester, 1824. Stone slab floor on cast-iron framing.

Figure 2.21 Bowling Mill, Dean Clough, Halifax (c. 1840). Hog-backed cast-iron beams and fish-bellied cast-iron joists which formerly supported stone slabs over the boiler house. (Stuart Millns)

Figure 2.22 Havelock Cotton Mill iron framing (1845). As re-erected on the UMIST campus in 1997.

Floor systems after 1850

Weight problems, and the recognition that iron has its own problems in fires, caused a move away from large brick arch floors in the second half of the nineteenth century. Timber continued to be attractive. Various techniques were used to substitute smaller section timber, where possible. Thick section timber planks were replaced by thinner machined tongued and grooved boards laid over intermediate joists. In some cases, double boarding was used with close fitting boards over rougher boards beneath. Around the beginning of this century, Canadian maple was a very common floor covering. Trussing of beams using wrought iron was one way of avoiding the need for the more expensive large timber sections (Figure 2.23a, b). Another technique was to sandwich a flitch-plate, usually of wrought iron but occasionally of cast iron, between two timber sections (Figure 2.23c). Large rolled steel sections became available in the 1880s and provided a strong and often economical alternative to timber primary beams.

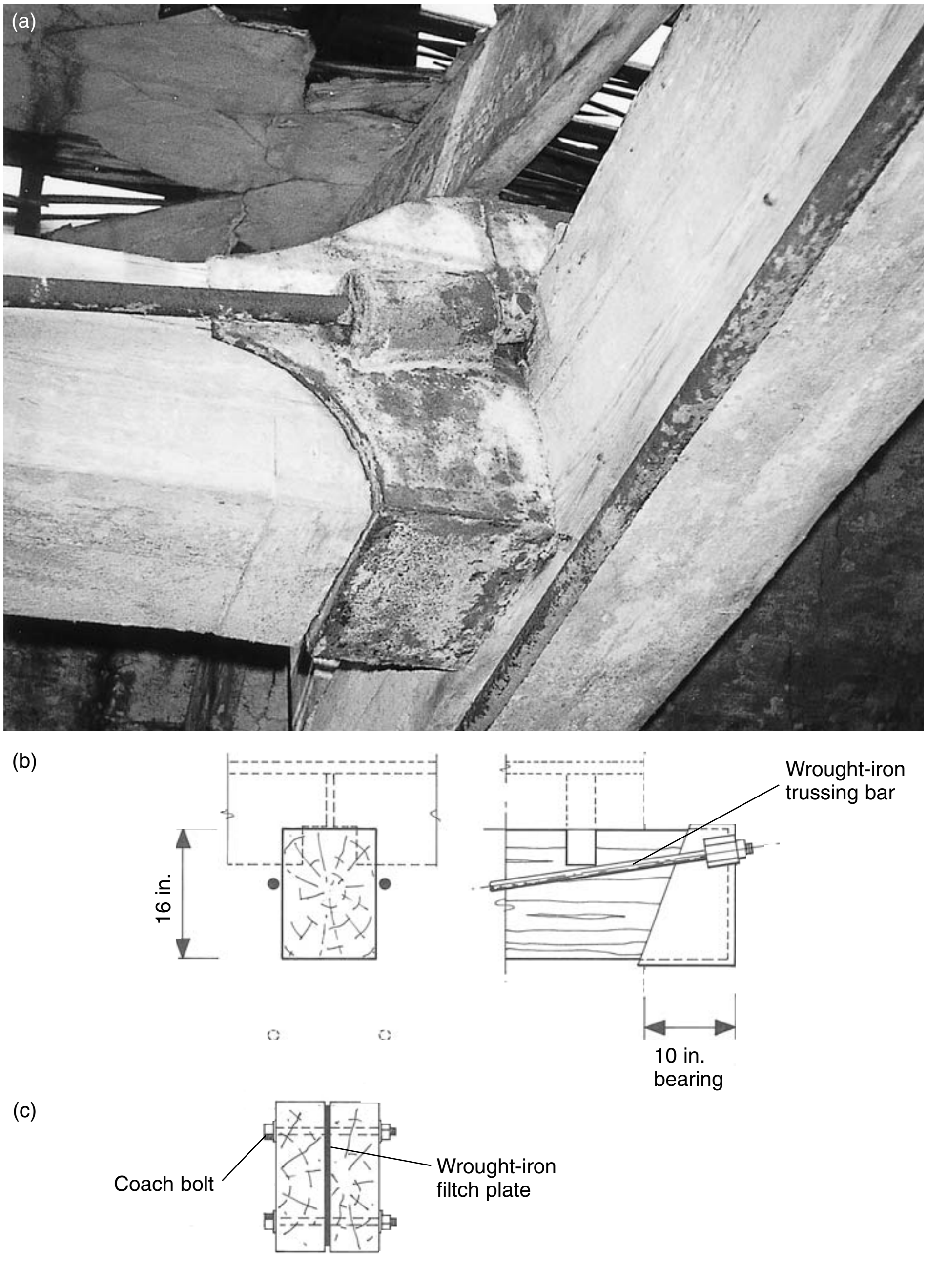


Figure 2.23 (a) Hunslet Mill, Leeds (c. 1832). Trussed beam bearing in a cast-iron 'beam hanger' on a cross beam, where a change of direction of span was required over the engine house (Stuart Millns). (b) Trussed beam. (c) Flitched beam.

The wide jack arch spanning between primary beams gave way to smaller arches spanning between secondary beams which themselves were supported by the primary beams. This system effectively reduced the thickness of the floor and hence could reduce the self-weight by as much as two-thirds. Initially the system used cast-iron primary and cast-iron secondary beams (Figure 2.24). In warehouses, fabricated wrought-iron beams replaced cast iron as supports for these 'lightweight' brick arch floors (Figure 2.25). Wrought iron and then steel provided alternative materials that avoided the risks associated with the use of a brittle material like cast iron.

In the later mills, large section rolled steel beams were used to support either brick arches or concrete and filler joist floors (Figure 2.26). Typically, wrought-iron filler joists, 4-in. deep, were closely spaced at the bottom of a concrete slab of twice that

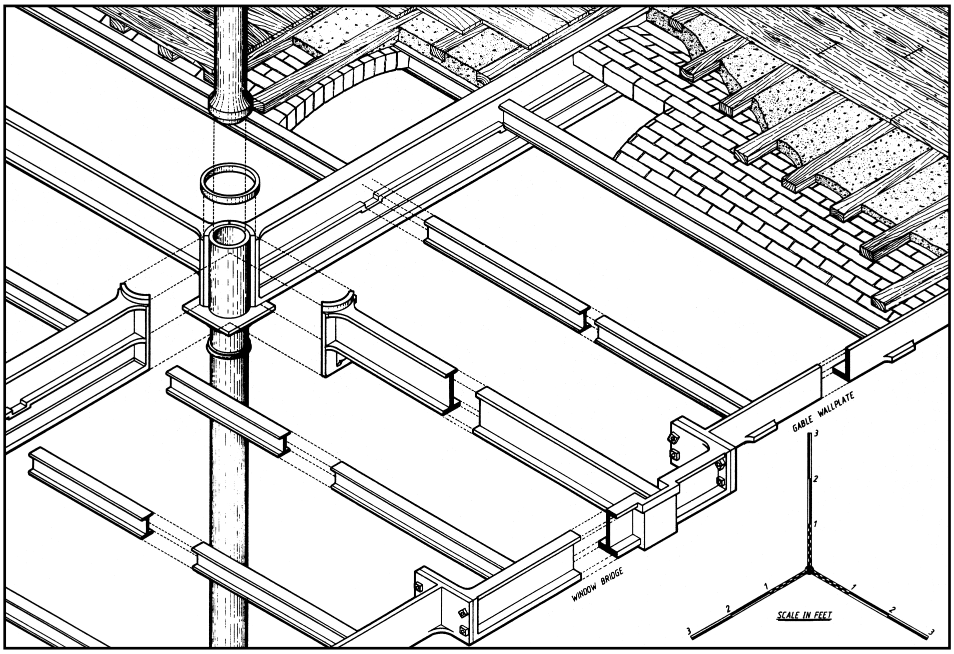


Figure 2.24 Lightweight floor system. Lee Bank Mills, Halifax (1863). (Ron Fitzgerald)

Figure 2.25 Lower Byrom Street warehouse, Museum of Science and Industry, Manchester.

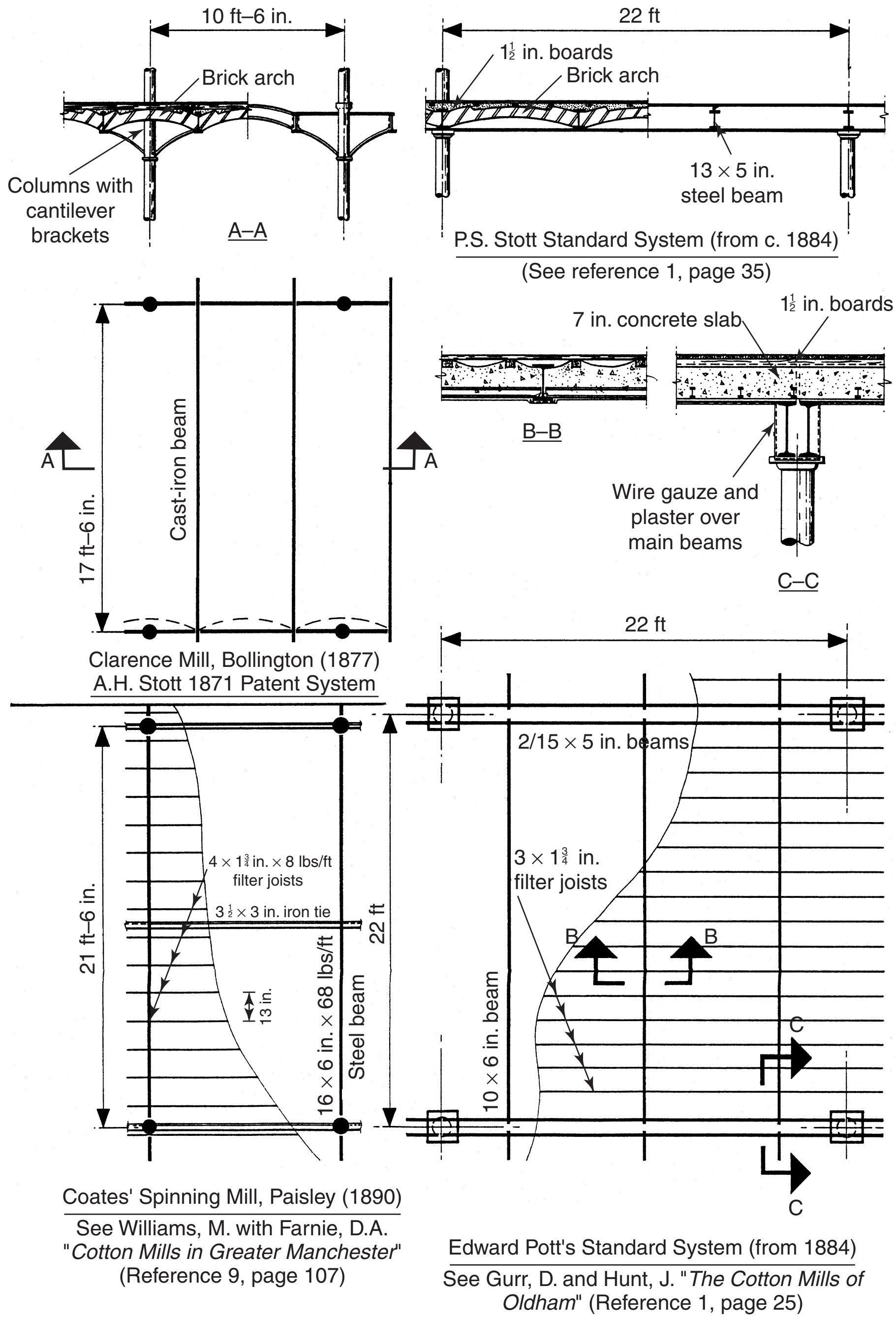


Figure 2.26 Floors and framing arrangements after 1870.

depth, in effect acting as reinforcement. The underside of the floor slab and the sides and soffits of the beams were covered with plaster for fire protection. Filler joist slabs were used over the same spans as the heavier brick arches but exerted no thrusts on the external wall piers, which as a result could be made more slender. With the introduction of large rolled steel sections in the 1880s, bay sizes of around 22-ft square became common.

From the late 1850s, well before the introduction of steel beams, there was a shift towards much wider and bigger buildings. Several mills in the cotton spinning centres of south Lancashire, dating from the 1880s, are over 100 ft wide and half as long

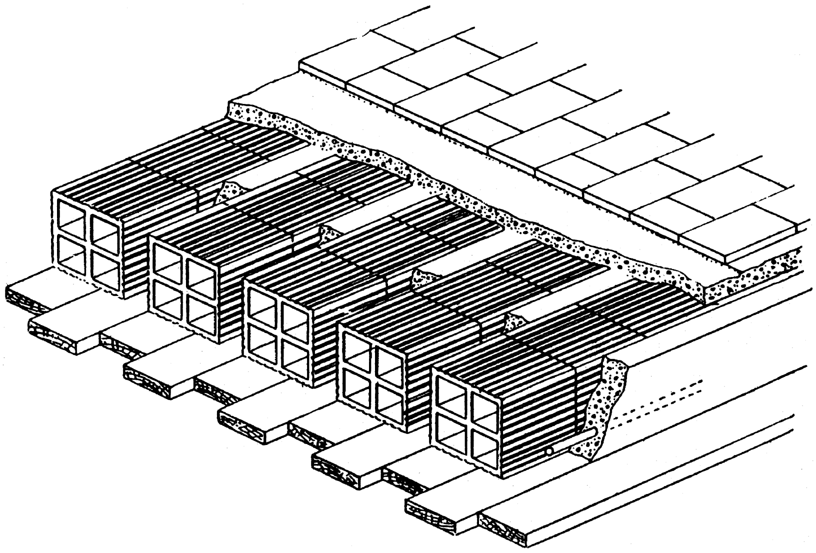


Figure 2.27 Patent fireproof floor. The 'National Floor', similar to the 'Kleine' patent floor, as found in the 1912 extension to Paragon Mill, Ancoats. (From T. Potter's Concrete, Its Uses in Building, *Batsford, London, 1908.)*

again. Taller windows and increased floor to floor heights let more light into these deeper buildings and provided space for modern services. Mill design and construction in this period was very much the province of the specialist architect and builder and, in these circumstances, innovations in other sectors of the building industry were slow to be adopted. Some designers still used the old-style brick arch and cast-iron primary beam system into the twentieth century. Heavier machinery was often located at ground floor level, where a heavy floor with cast-iron beams would be less prone to vibration than a lighter floor. Many lightweight 'fireproof' floor systems were patented around this time, often using hollow clay pots. Only a few examples have been noted in mill buildings (Figure 2.27 [27]).

Roofs and roof spaces

In the first half of the nineteenth century, mill roof spaces were used as working spaces with varying degrees of success. Traditional slate-clad pitched roofs using queen post timber trusses gave some working or storage space. Sometimes cast-iron members were substituted for some of the timber members of trusses. One solution involved the use of cast-iron knee braces to raise the ends of the principal rafters clear of the floor (Figure 2.28). In some cases, segmental cast-iron arches with their feet tied by the floor/ceiling beams, replaced the trusses entirely. This 'ironbridge-like' solution to providing useful roof space can look quite elegant (Figure 2.29). In a few later mills, laminated timber arches were used instead of iron (Figure 2.30).

Other solutions to the roof problem developed around the use of wrought iron (Figure 2.31). Often, the use of iron was not confined to the building frame and some of the earliest iron trusses support mill roofs. Well-engineered examples from early in the nineteenth century betray the influence of firms such as Boulton & Watt on the structural design of the mill buildings that were to be powered by their steam engines. High double-pitched roofs gave way to much lower multi-bay roofs with networks of wrought-iron tension rods and wrought-iron angle sections supporting ridges and valleys. These boarded and slated roofs with north facing roof lights were early space frames, requiring the minimum of intermediate support. For longer span building roofs in the second half of the nineteenth century, development was driven, for the most part, by the needs of railway stations and station-type roofs are to be found over some mills.

Figure 2.28 Timber roof with cast-iron knee braces. Garden Street Mill, Halifax (c. 1860). The brackets are fixed to the cast-iron floor beams and have a second leg within the wall. In this case there was no evidence of roof spread. (Stuart Millns)

Figure 2.29 Cast-iron arched roof to part of Beehive Mill, Manchester (1825).

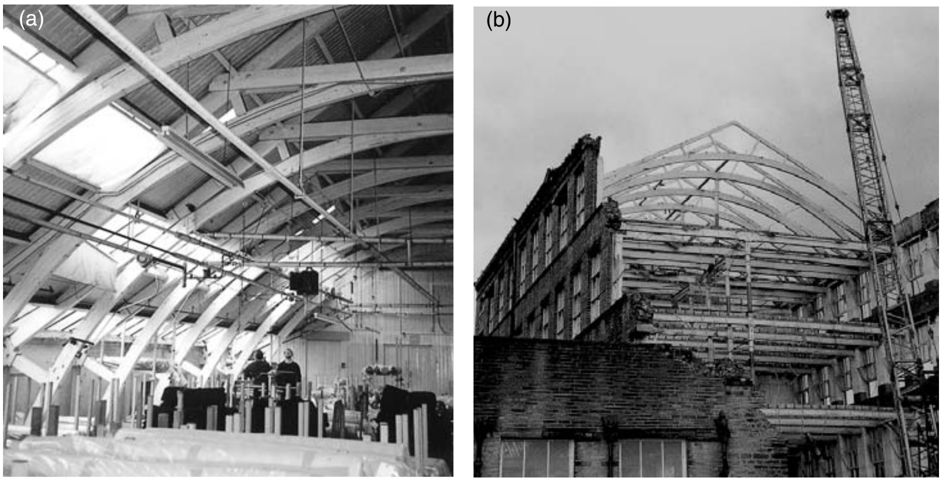


Figure 2.30 (a) Laminated timber roof arches with cast-iron wall brackets. Lumb Lane Mills, Bradford (1856). Up to six timber planks were screwed together with through-bolted truss connections (Stuart Millns). (b) Similar timber arches and floors seen during demolition of a mill. (Ron Fitzgerald)

Figure 2.31 Trussed iron roof to part of Beehive Mill, Manchester (1824).

The final solution for roofing the larger mills simply involved putting an asphalt covering over the top of a concrete floor. The resulting flat roof was occasionally used to store water or, in at least one case, to grow grass for feeding the sheep which would provide the wool to be turned into cloth by the mill below.

Repairs and alterations

Not many existing mill buildings have survived completely as originally constructed. It was common to repair and replace damaged components (Figure 2.32). Also common was alteration of buildings to upgrade their load carrying capacity. Timber beams might be upgraded by trussing (Figure 2.33a, b), through the addition of similar timbers in parallel, or through the 'bolting on' of wrought iron or steel plates or rolled sections (Figure 2.33c).

Nineteenth-century strengthening work was often linked to the need to provide greater clear floor space for new larger machines. Many of the original slender cruciform iron columns on the lower floors of early mills have been replaced by more sub-

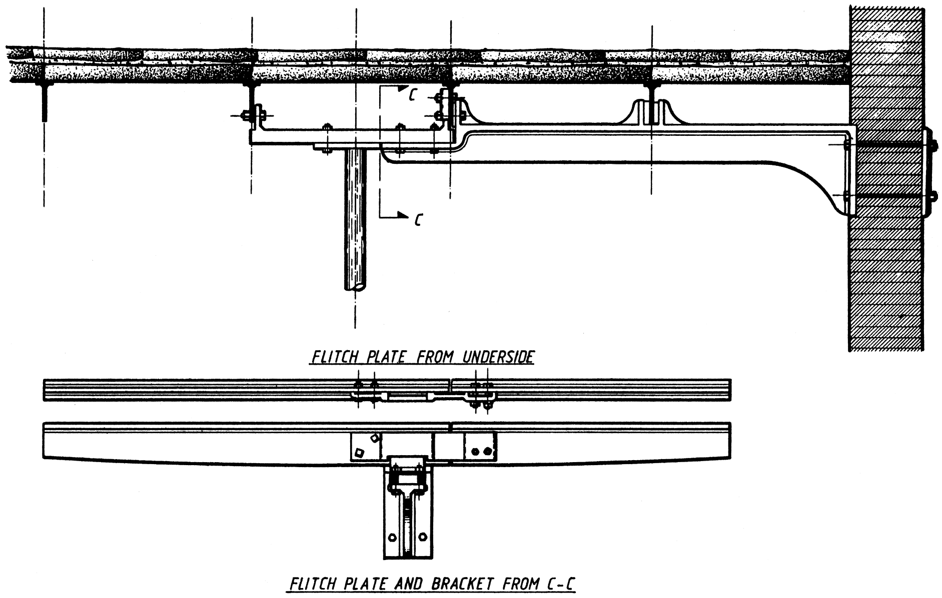


Figure 2.32 Beam repair, Water Lane Mill, Leeds (c. 1838). (Ron Fitzgerald)

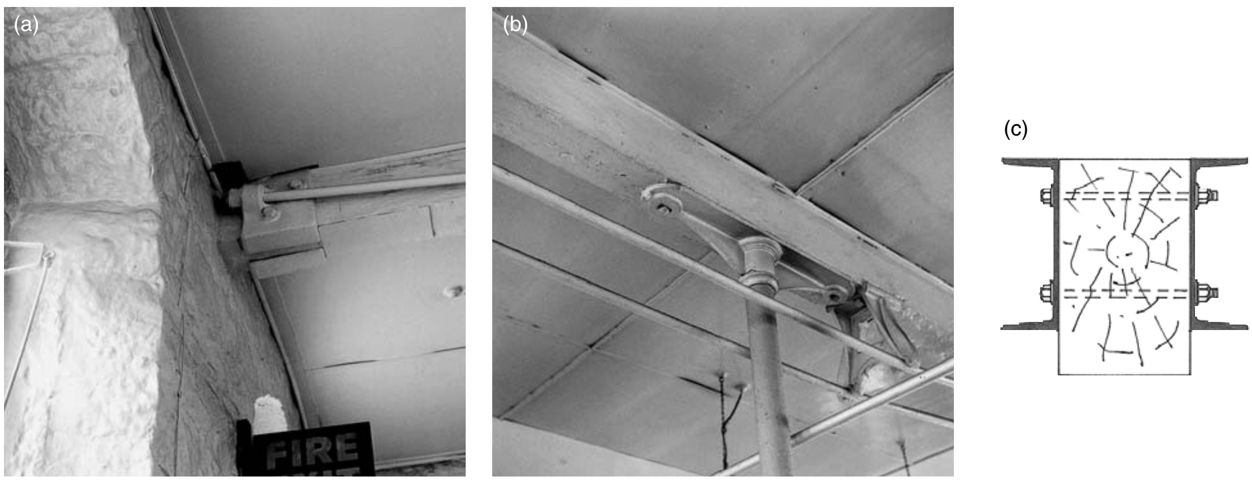


Figure 2.33 Strengthening of timber floors by (a) the addition of wrought-iron trussing to timber beams or (b) by propping with additional columns. Varley Mills, Stanningley, Yorkshire built 1803, strengthened later (Ron Fitzgerald). (c) Strengthening of a timber beam by bolting on steel channels.

Figure 2.34 Floor supports in Murray's New Mill, Manchester. Built in c. 1804, with two lines of cruciform columns to continuous timber beams, the building was modified c. 1900. The original columns were replaced with a single line of hollow circular cast-iron columns and the floors supported by channels placed either side of the original timber beams.

stantial hollow circular columns at wider spacings (Figure 2.34). Individual columns or rows of columns may have been moved or removed without much consideration of the wider structural implications (Figure 2.35–2.37 [28]). On occasion, complete floors have been taken out (Figure 2.38).

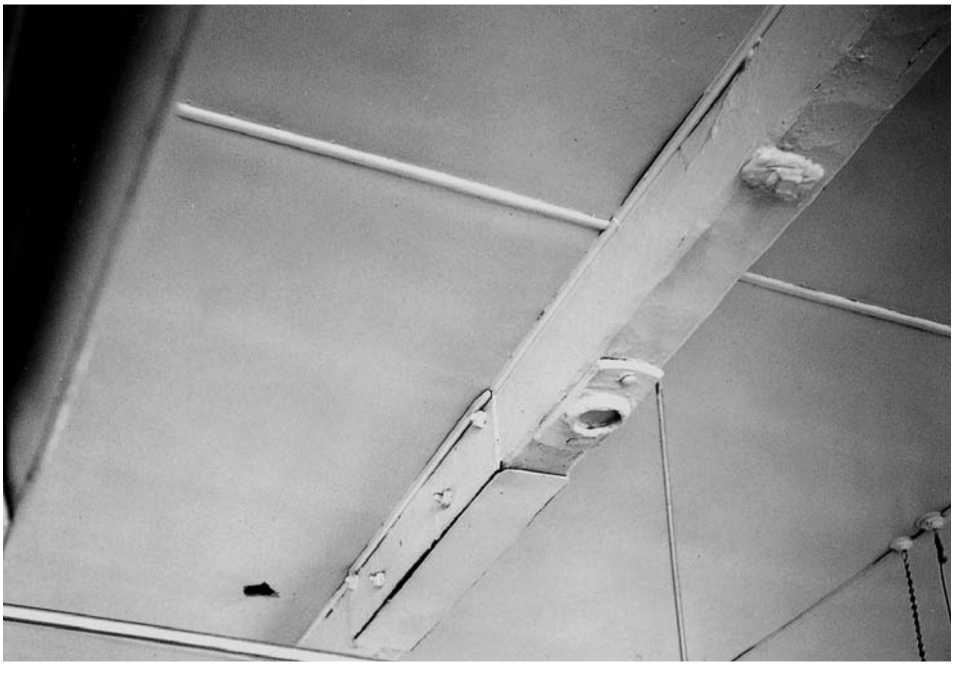


Figure 2.35 Iron storey post-to-timber beam removed and beam flitched with wrought iron plates (at scarf joint?). Varlrey Mills, Stanningley, built in 1803, altered later. (Ron Fitzgerald)

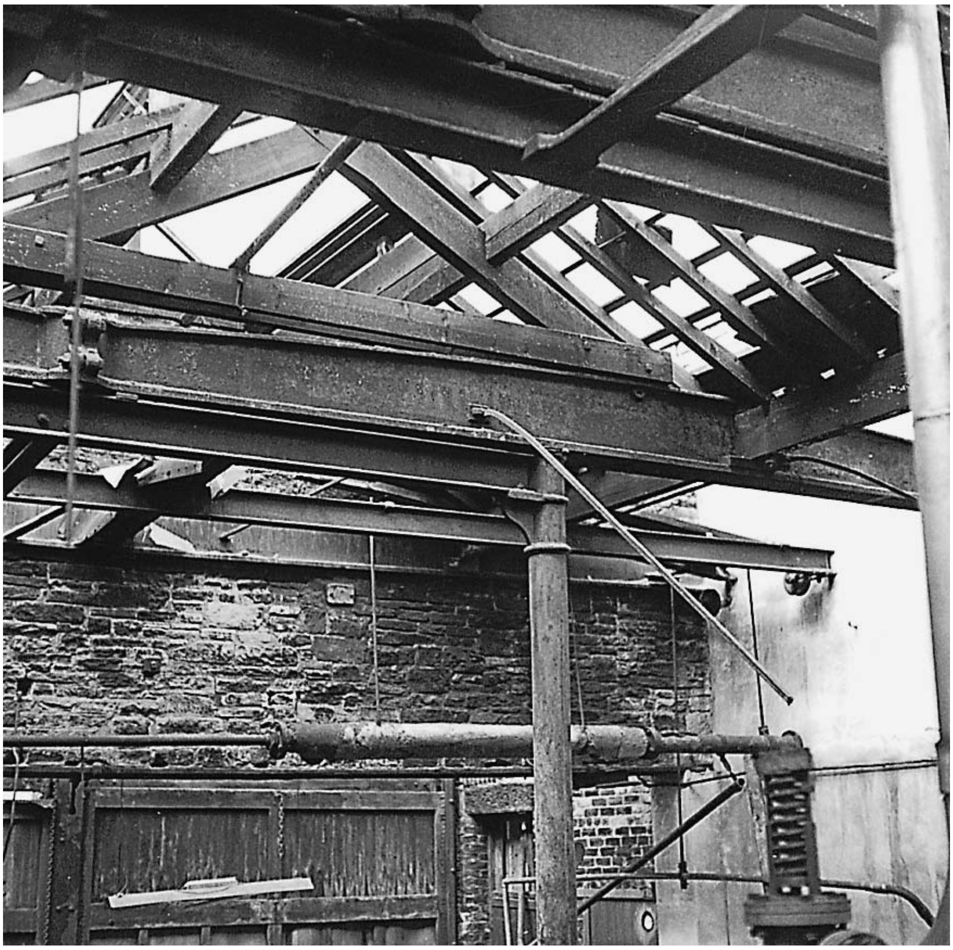


Figure 2.36 Cast-iron column moved under cast-iron beam. Boiler house, Pellon Lane Mill, Halifax (1865). (Ron Fitzgerald)

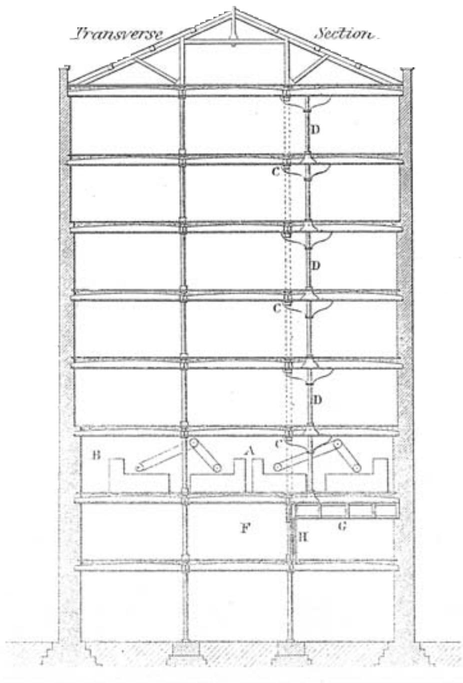


Figure 2.37 Fairbairn's alterations to a Manchester cotton mill. This figure is one of several which accompanies William Fairbairn's paper 'Description of the removing and replacing of the iron columns in a cotton mill' (PIMechE, 1866, pp. 181–5). Moving one line of columns enabled a pair of self-acting spinning mules to be accommodated down the centre of the building. The work was completed a floor at a time without stopping production at the mill, which stands in its modified form today.

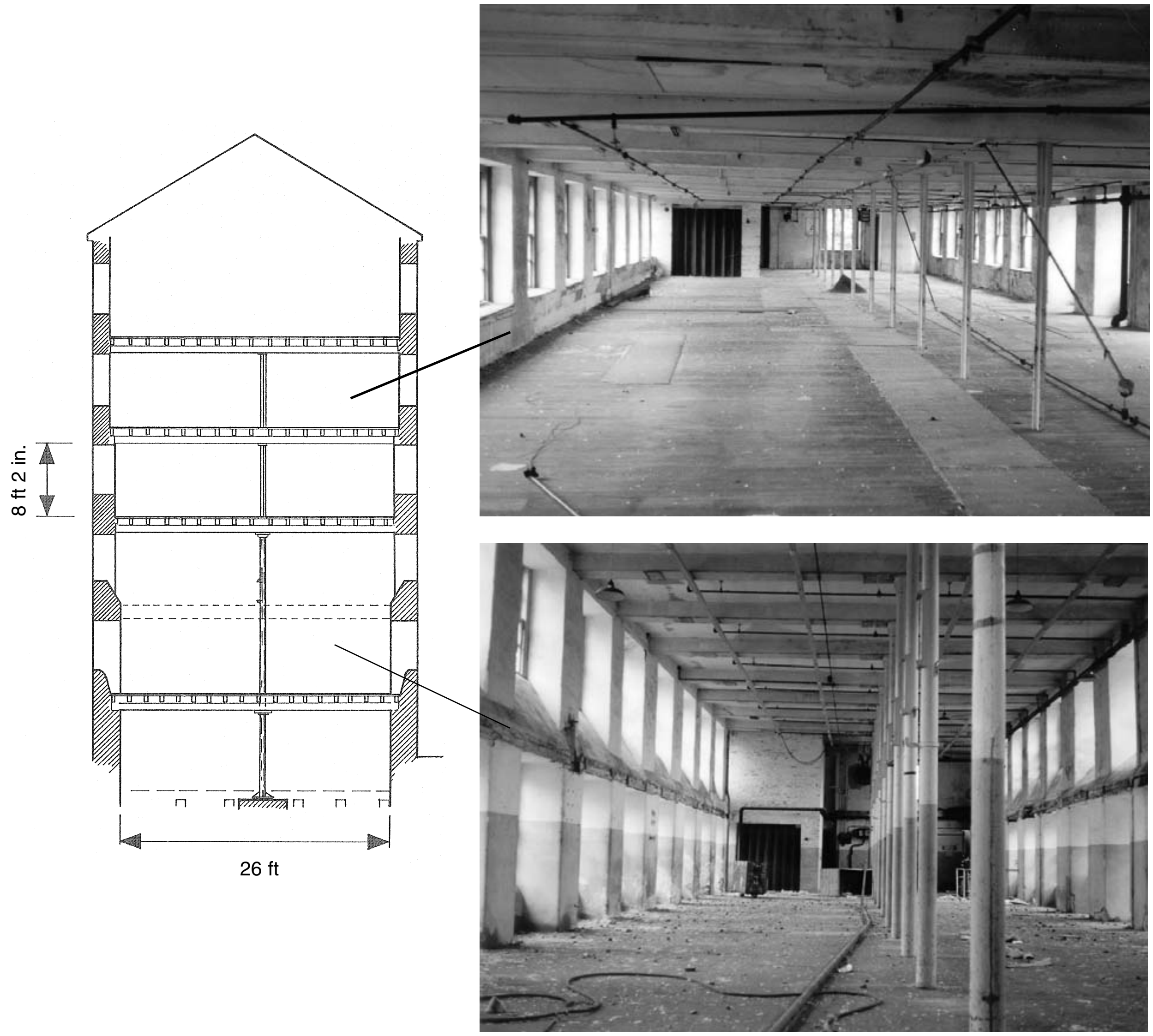


Figure 2.38 *Floor removal in Bell Mill, Stanley, near Perth. Built in c. 1787 a complete floor removed about 100 years later.*

Figure 2.39 *Murray's New Mill, Manchester (1804). Cast-iron queenposts inserted as a modification to the original all-timber truss.*

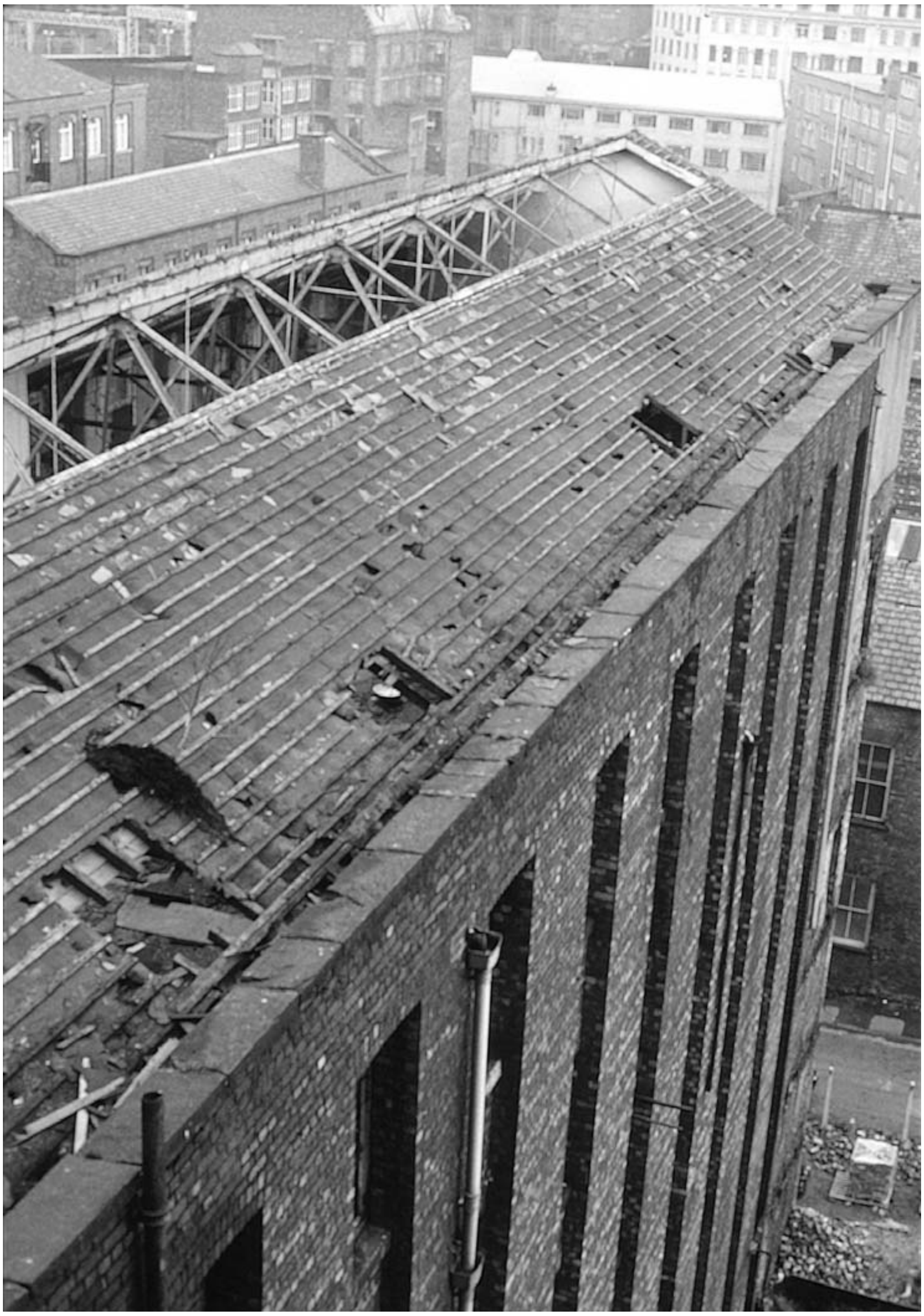


Figure 2.40 Chorlton Mills, Manchester. Built in c. 1803, the outer leaf to the mill is probably a late nineteenth-century addition.

Roofs are areas where extensive modification can be expected. Many mills have had upper storeys removed. In others, heavy single-pitched roofs have been replaced by lighter structures. In yet others, heavy timber sections were replaced by special cast- and wrought-iron sections (Figure 2.39).

Exterior structural walls are unlikely to have suffered significant alteration although some new window and door spaces will have been opened in many buildings. A Manchester mill had an additional complete outer leaf added sometime after the original construction, but that treatment was exceptional (Figure 2.40).

3. Robustness and whole structure considerations

General background to regulations and code requirements

In recent times, major building failures have been few, so the partial collapse of the Ronan Point flats in London in 1968 came as a sharp shock [29]. A gas explosion in an eighteenth floor flat blew out a pre-cast concrete external wall panel, causing part of the five floors above to fall. Unable to sustain the debris load, the eighteenth floor failed in turn and the collapse progressed downwards almost to ground level. Publicity in the press, questions in Parliament, circulars from the Ministry of Housing and local government and from the Institution of Structural Engineers preceded the introduction of the 'Fifth Amendment' to the Building Regulations. This provided the basis for the present day requirement for buildings of five or more storeys. Building Regulation A3 requires that:

> The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause

Approved Document A provides some guidance on the alternative methods that may be adopted in order to comply with Building Regulation A3. In simple terms, these are:

— the 'tying' method
— the 'bridging' method
— the 'key element' method.

More detailed guidance for the engineer concerned with a building of five storeys or more is given in the codes of practice for the main present-day structural materials that are referred to in the Approved Document.

The basic principle on which the guidance in the Approved Document is founded is that, if a structural member in a building cannot survive an accident, then its removal should not lead to a disproportionately more extensive collapse. A building structure needs to possess some redundancy to be able to survive the loss of a member, the load from which must find an alternative path to safety at foundation level. Of course, a building need only be expected to withstand accidental events of reasonable severity and this is clearly the intent of the regulation. In the case of an extreme accidental event (for example, if a large jet aircraft were to crash into a building), it may be assumed that the collapse of the whole building, or at least a large part of the building, is not an unreasonable consequence. Any interpretation of Regulation A3 that may be sought following an accidental event will therefore depend on the cause of the accident and on the likelihood of its occurrence.

In checking an existing building against the requirements of Regulation A3, it is sensible to first consider how effectively the structure is tied together. In an iron-framed textile mill, the cast-iron columns are usually discontinuous and the load-bearing masonry walls will be ineffective as vertical ties. Wrought-iron ties may or may not be present in the floors, between the beams and/or columns. In later mill buildings, more complete horizontal tying may be provided by steelwork framing to the floors.

In the absence of satisfactory tying, the 'bridging' method of assessment should be adopted. The Approved Document requires that:

> ... each support member should be considered to be notionally removed, one at a time in each storey in turn, to check that on its removal the area at risk of collapse of the structure within the storey and the immediately adjacent storeys is limited ...

The specified limits are that the structure at risk of collapse should be limited to three adjacent storeys and that the area at risk in any one storey shall be limited to the lesser of:

— 15% of the area of the storey
— 70 m^2.

Bridging over or around a lost member may be achieved by bending, arching or catenary action, two- or three-dimensionally, with loss of serviceability and large deflections acceptable. Large deflections are, of course, a requirement for catenary action, which is often utilized to good effect in the design of modern relatively ductile structures of steelwork or reinforced concrete. However, excessive deflections in frames of more brittle cast iron might be expected to break beam end connections before a catenary profile is achieved.

If the assessment shows that the notional removal of a supporting member will lead to a collapse beyond the permitted limits, then that member must be designed as a 'key member' to sustain accidental loading of 34 kN/m^2 in accordance with BS 6399: Part 1.

Strength, stability and robustness

If a multi-storey textile mill were to be built today, a civil or structural engineer would play a key role in designing for resistance to both normal and accidental loadings. The building would need to be robust enough to be able to withstand undefined accidental events of reasonable magnitude. Therefore, owners and users of textile mills should expect significant engineering input to be required when the suitability of a large old building of this type is to be evaluated for a particular use.

Although the floors of a textile mill and their supports will often be strong enough for normal imposed loadings, or can be made so unobtrusively, a rehabilitation may bring new risks, such as gas explosions or vehicle impact. Neither Regulation A3 nor Approved Document A define an accident but guidance in the latter aims to ensure that buildings of five storeys or more can be provided economically with sufficient inherent robustness to enable them to withstand a reasonable level of abuse. If a user or a potential user of an existing textile mill should specifically require that the building be assessed for a particular risk (for example, an accidental gas explosion) specialist advice would need to be sought.

It is worth attempting to clarify the terms ‘stability’ and ‘robustness’. It might be said that a building which is stable but not robust is perfectly satisfactory in normal circumstances but that a robust building has something more—the ability to withstand the unexpected. If a building has given good service and been well maintained over a hundred years or more, this is, in many ways, proof of strength and stability. However, very few buildings ever suffer an accident that represents a severe robustness test, so age gives very limited assurance of adequacy.

An iron-framed textile mill is likely to have been exposed to severe wind loading conditions during its long life, so it is worth considering how wind loads are carried. In simple terms, lateral wind loads are probably resisted to a lesser extent by frame action and to a greater extent by the plate or diaphragm action of floors and roof between gable or intermediate shear walls or cores. For the strictly temporary case, during demolition, much more may be expected of the wall piers, iron beams and columns, working together as a frame. Avoidance or reduction of hazards during demolition or during the course of structural alterations is a requirement of the Construction (Design and Management) Regulations 1994 [30]. If, during the life of a building, its structure has been modified, or if modifications are proposed, verification of stability under wind loading may be required.

Beam ends in iron-framed textile mill buildings were commonly set well into the external wall piers so as to provide greater lateral stability. Recent site tests in three different buildings have demonstrated that such beam ends may be considered ‘partially fixed’ (Figure 3.1) [31]. For beam strength calculations, a simple support assumed at the inner face of the wall pier is a little bit conservative.

At low loads, cast-iron beams and the supporting arch brickwork have been found to act together as continuous beams over the supporting columns. At higher loads, with the brickwork cracked, this continuity is lost. The beam to column connections usually have a very limited moment capacity. In fact, the form of many connections

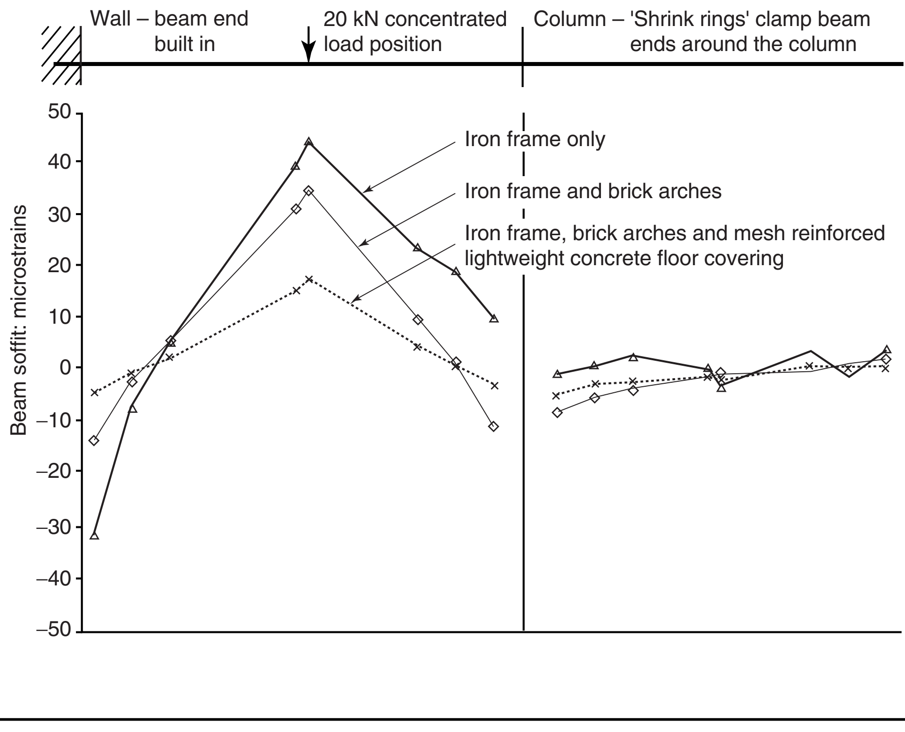


Figure 3.1 Beam concentrated load tests.

suggests some awareness on the part of the designer of the intolerance of a cast-iron frame to differential settlement. Simple support of a beam at a column should be assumed unless the beam/column connection detail can be proved to be adequate for some other end condition. Clear evidence of real beam end conditions has been seen in many buildings (Figure 3.2).

After the Ronan Point collapse, the robustness of load-bearing masonry buildings came under close scrutiny. Full scale gas explosion tests were carried out in real buildings and detailed guidance on robustness followed [32, 33]. Much of this guidance is useful for the engineer appraising an iron-framed textile mill, but these buildings have some unique features that require special attention. For example, what happens if a floor arch fails and under what circumstances is this likely to happen? By what means may the structure accommodate the removal of a cast-iron beam or column?

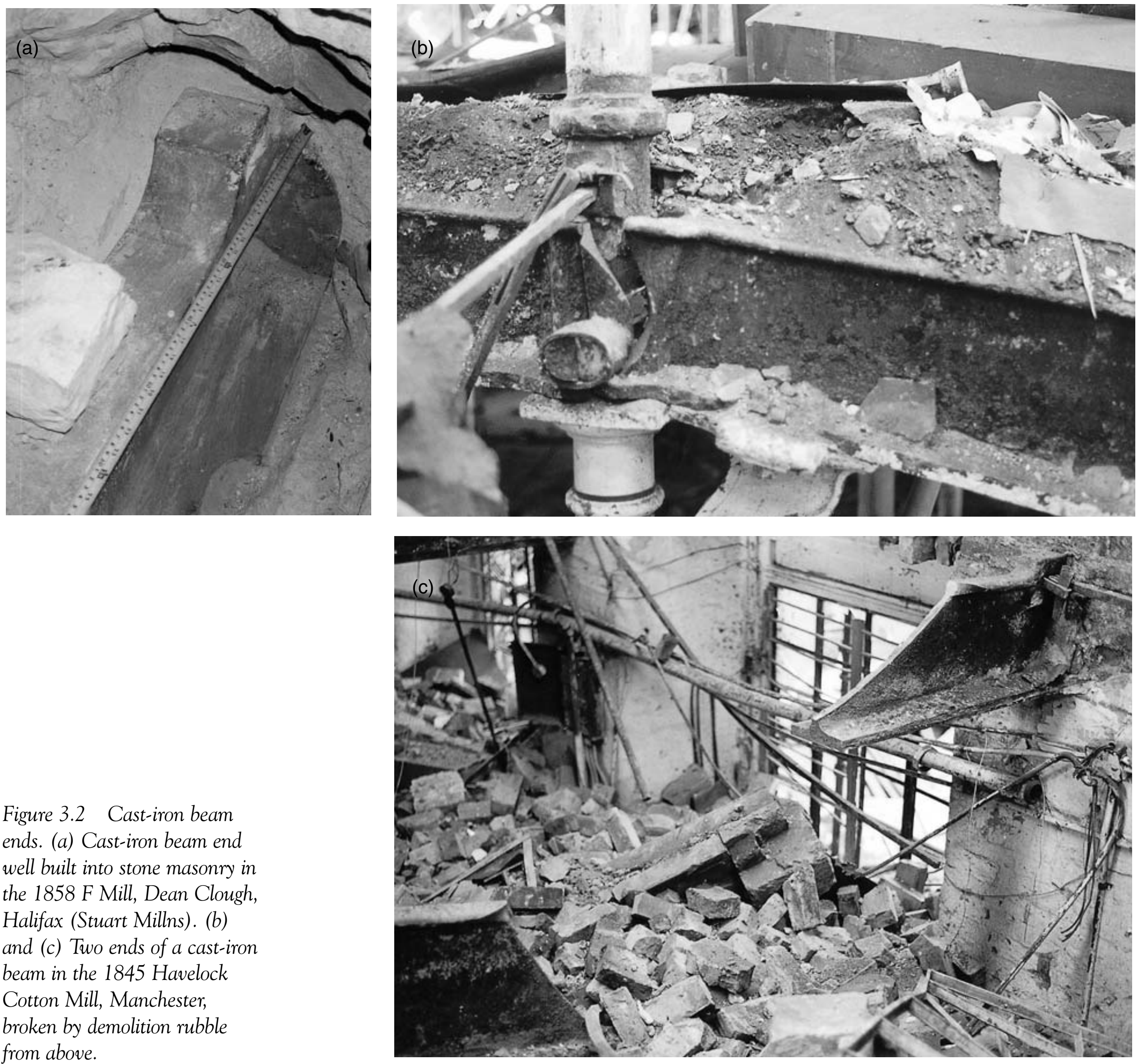


Figure 3.2 Cast-iron beam ends. (a) Cast-iron beam end well built into stone masonry in the 1858 F Mill, Dean Clough, Halifax (Stuart Millns). (b) and (c) Two ends of a cast-iron beam in the 1845 Havelock Cotton Mill, Manchester, broken by demolition rubble from above.

If a building is not sufficiently robust, how can it best be made so? Some recent site tests have provided partial answers to some of these questions [34]. To help provide a clearer appreciation of these buildings, the results of these tests are considered here alongside some nineteenth- and twentieth-century building failures. The collapse of Radcliffe's cotton mill in Oldham in late October 1844 was as dramatic as the collapse at Ronan Point [35]. The fireproof five-storey mill was three bays wide and built on an earlier structure (Figure 3.3).

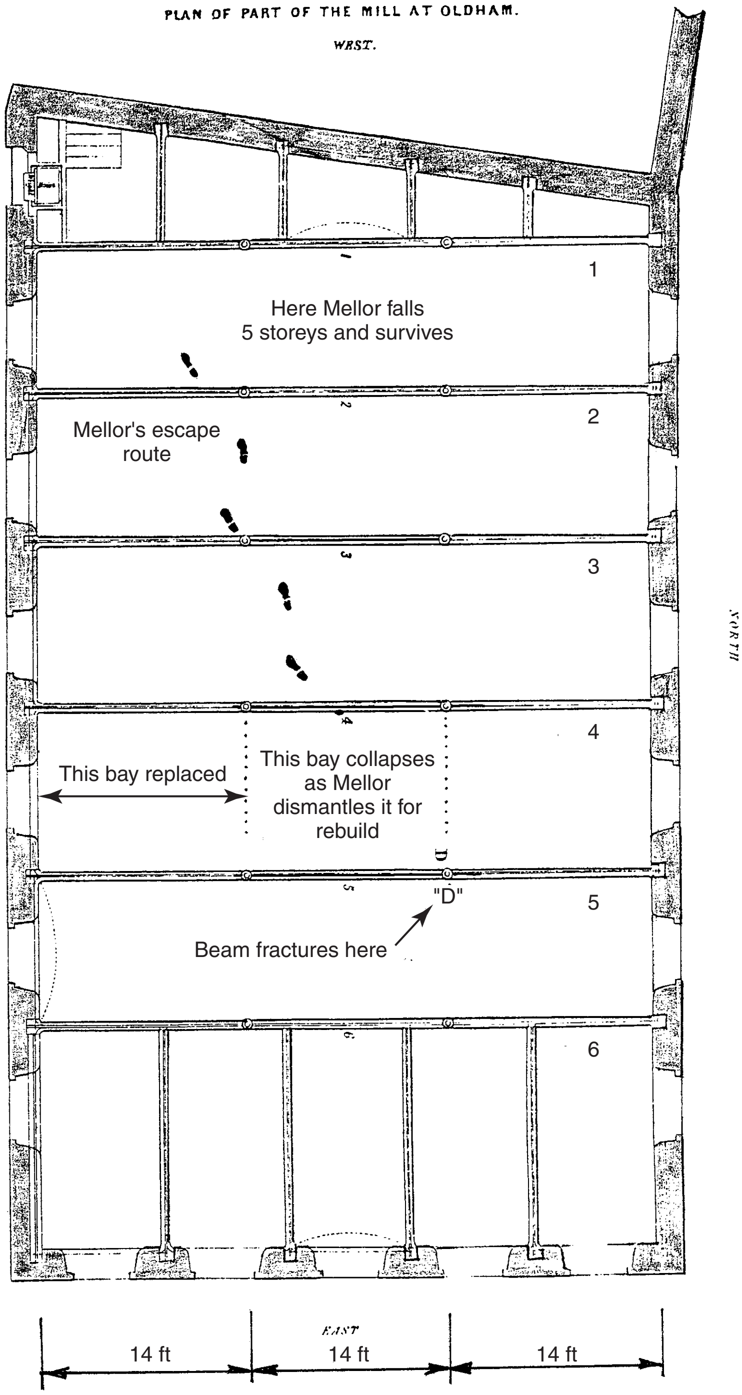


Figure 3.3 *Radcliffe's Cotton Mill, Oldham (based on a figure in 'Report on the fall of the cotton mill at Oldham and part of the prison at Northleach', HMSO, 1845).*

The top floor was almost complete when sagging of the arch between beams four and five was noticed. It was decided to replace the entire arch, one 14-ft bay at a time. An outer bay had been replaced and temporary timber struts fixed between the beams to the central bay to take the thrust from the neighbouring arches. Wrought-iron ties were in place between the beams, but too high to be of great use in tying the arches. What happened next is described in the Report of the Royal Commission of Inquiry. Bricklayer Thomas Mellor was standing on the arch centring and,

> taking out part of an arch, he saw the beam No. 5 break at the point marked D. He instantly fled over the other arches towards the old part of the mill, the arches falling in succession as he passed over them, until he reached that between the lines of beams Nos. 1 and 2, when that giving way beneath him he fell with it, and became entangled in the ruin.

Asked how he was saved, Mellor replied 'My body was first caught under the roof, I being on the upper storey, but my legs were clear, and were seen, and I was got out in about 10 minutes'. Others were not as lucky as this witness to a classic 'progressive collapse', as the Report states:

> By all accounts the crash was almost instantaneous, the whole building apparently giving way at the same time, and the ruin was almost complete. Twenty unfortunate persons perished in it, and many others were more or less injured.

Local engineers William Fairbairn and David Bellhouse investigated the collapse and found that some of the cast-iron beams were both undersized and badly made; it was their failure that led to such a complete collapse. These experts were at pains to point out that this building had been particularly badly designed and constructed, the owners taking advice from neither architect nor engineer.

A modern illustration of the knock-on effect of an arch failure has been provided by an engineer who inspected a derelict fireproof building (with leaky roof) in the Scottish lowlands. A top floor arch collapsed because water had leached mortar from the brickwork joints. The thrust from the neighbouring arch then pushed both the supporting cast-iron beam and column over, causing the latter to break at its support in the floor below (Figure 3.4, [36]). Fortunately, the broken column section then seems to have 'jumped', coming to rest almost upright on the floor. The displaced beam remained unbroken, further collapse being prevented by the wrought-iron ties, this time well placed lower in the arch section. However, the ties became effective only after significant movement at their end connections had allowed the formation of a hinge in the arch barrel. This case also indicates the importance of the floor arches in providing lateral restraint to the columns if ties between the columns are either ineffective or absent altogether.

The Oldham mill report contains accounts of other failures too, the first of these again showing problems with arches. During the building of a fireproof prison building at Northleach in Gloucestershire, six arches in lime mortar sagged, pushed out the walls, then collapsed. Here the blame was placed on a failure to protect the arches against heavy rain leading to degradation of the lime mortar.

In 1824, a spectacular example of the consequences of the failure of a single cast-iron beam to the top floor of a building was seen. A quarter of a two bay wide and six storey high fireproof mill in Oldfield Road, Salford collapsed, killing 17 people. The proceedings of the inquest were reported in the *Manchester Guardian*, which gave a good description of the building and an excellent analysis of the probable causes of failure.

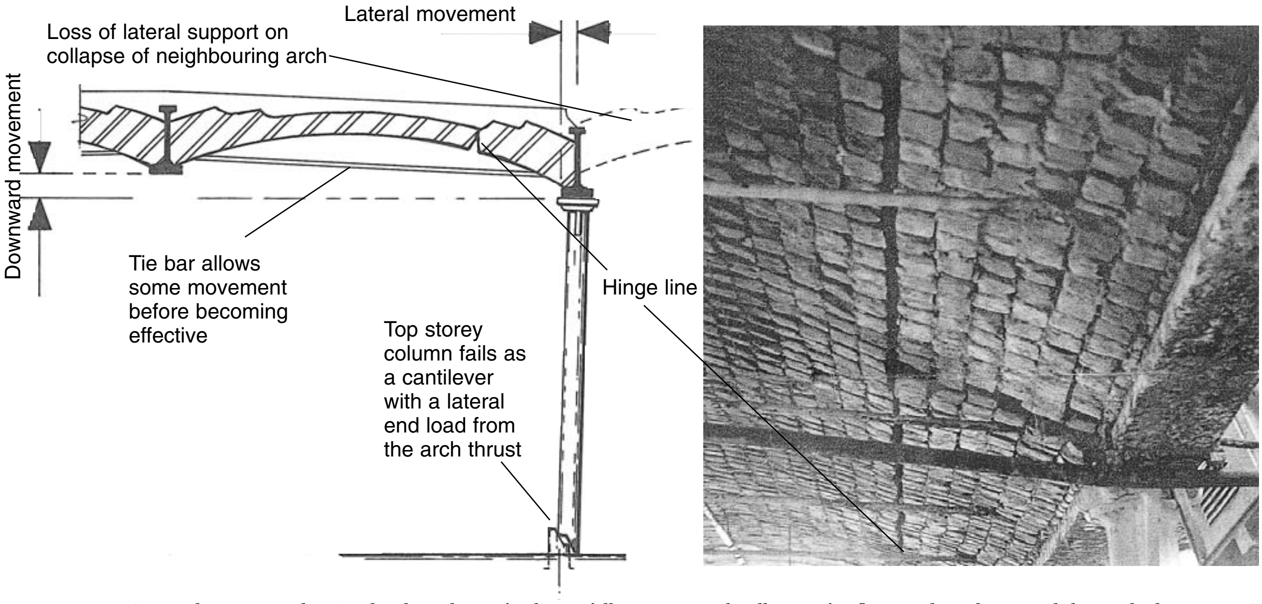


Figure 3.4 Gourock Rope Works, Scotland. Failure of column following partial collapse of a floor arch. (Photograph by Malcolm Watters, courtesy of Historic Scotland.)

As at Oldham, the evidence of survivors pinpointed accurately the place at which the progressive collapse started.

Top floor arches one to three above the boiler house were of greater span than the other arches (Figure 3.5). It seems that there was no increase in the size of the 20-ft span beams to allow for the greater weight, which was further increased by solid partitions labelled *a* and *b* on the plan, and by four carding machines over arch three. The beam at *c* broke suddenly and, when located in the debris, was found to contain a flaw which 'pervaded full one third the substance of the beam'. This beam pulled down with it the neighbouring beam to which it was tied and three arch spans also fell as a result. The falling debris broke successive levels of floor arches, ending up at ground floor level, but the external walls and the iron framing remained largely intact.

The *Manchester Guardian* dismissed the speculation of a rival paper 'that the ends of the beams were not put far enough into the walls', although the building '... of unusual height and width, was unusually slight in its construction'. Of the top floor, the *Guardian* stated:

> the beams had broken near their middle, whilst the ends were fixed in the walls; for they had evidently forced up the bricks above them with great violence.

The building was only a little over a year old and the top floor had become occupied by a tenant only a short time before the collapse.

Even though it seems likely that this was a weak building with some undersized beams, the collapse illustrates vividly how the safety of a large building was dependent on the strength of a single element. It is interesting that the builders were Bellhouse & Son, as David Bellhouse was to give expert evidence to the Oldham Mill Inquiry 20 years later.

A test in Havelock cotton mill in Manchester in 1995 involved propping a beam either side of mid-span, sawing through the beam between the props, then slowly

This beam fractured first

The beam to arches 2 and 1 is tied to the first beam, breaking as a result of being pulled down and sideways.

A progressive collapse of the numbered arches (to 14) follows.

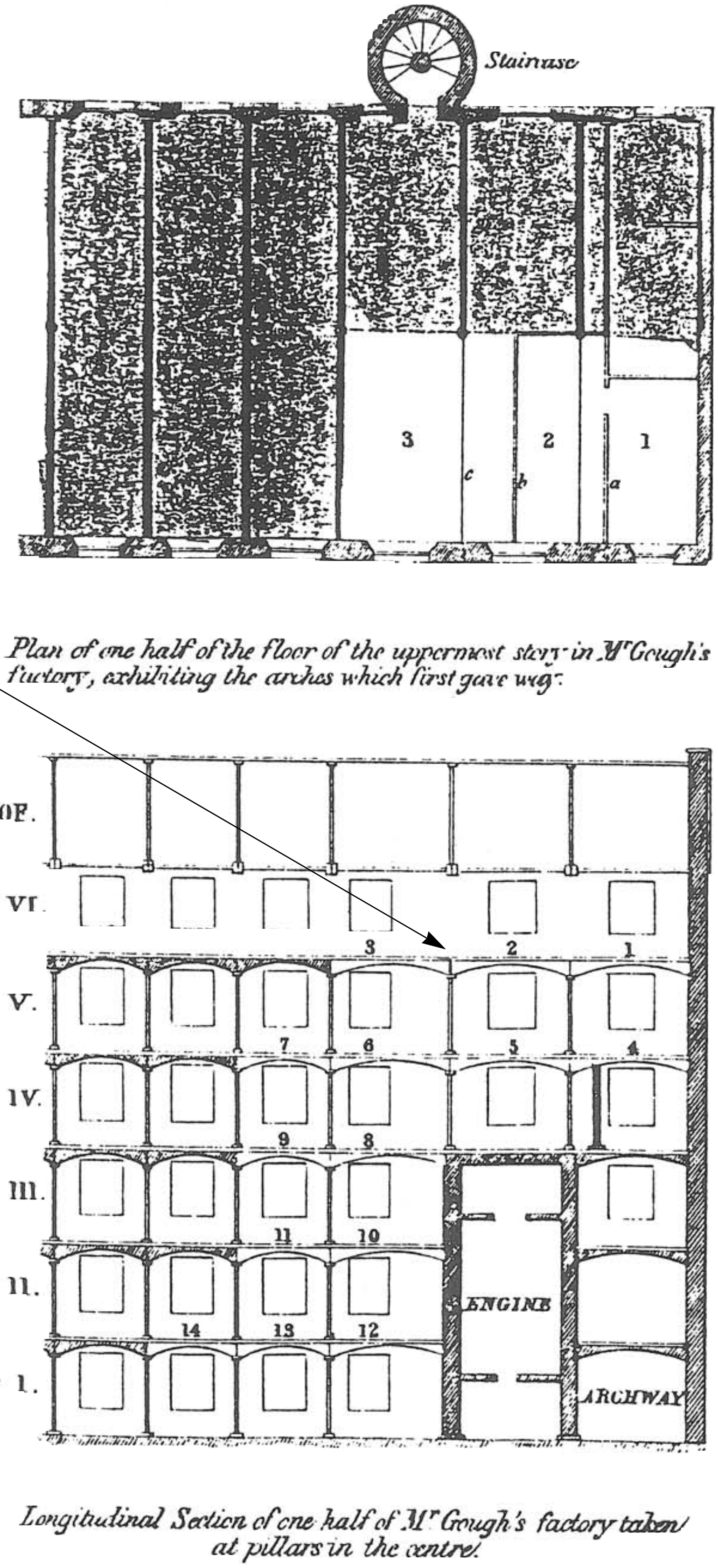


Figure 3.5 Gough's Mill, Salford (based on a figure in 'Report on the fall of the cotton mill at Oldham and part of the prison at Northleach', HMSO,1845).

lowering the props (Figure 3.6). Under these controlled conditions the props were completely removed, to leave the separated beam halves supported by a column at one end and along their lengths by the intact arch masonry. That most iron-framed mill buildings can safely withstand the 'not so careful' removal of a beam or an arch has been proven countless times on demolition sites (Figure 3.7). Many buildings that still stand and show little sign of distress have had arches removed with no special precautions being taken to prop the neighbouring spans in either the temporary or the permanent condition. Although such a cavalier approach is not recommended, it is worth trying to understand how floors modified in this manner can apparently continue to perform well (Figure 3.8).

Beams taken from Havelock cotton mill in Manchester after demolition were tested in the laboratory in bending between simple roller supports. Their fractured halves jumped two feet apart on failure. The response of a building to the sudden failure of a beam by brittle fracture may be different from the response to a slow, careful beam removal or to the failure of a ductile beam of wrought iron, steel or reinforced concrete. However, although the guidance given in Approved Document A does not require the manner of element removal to be taken into account, the tying force required to hold together the fractured halves of a cast-iron beam is modest.

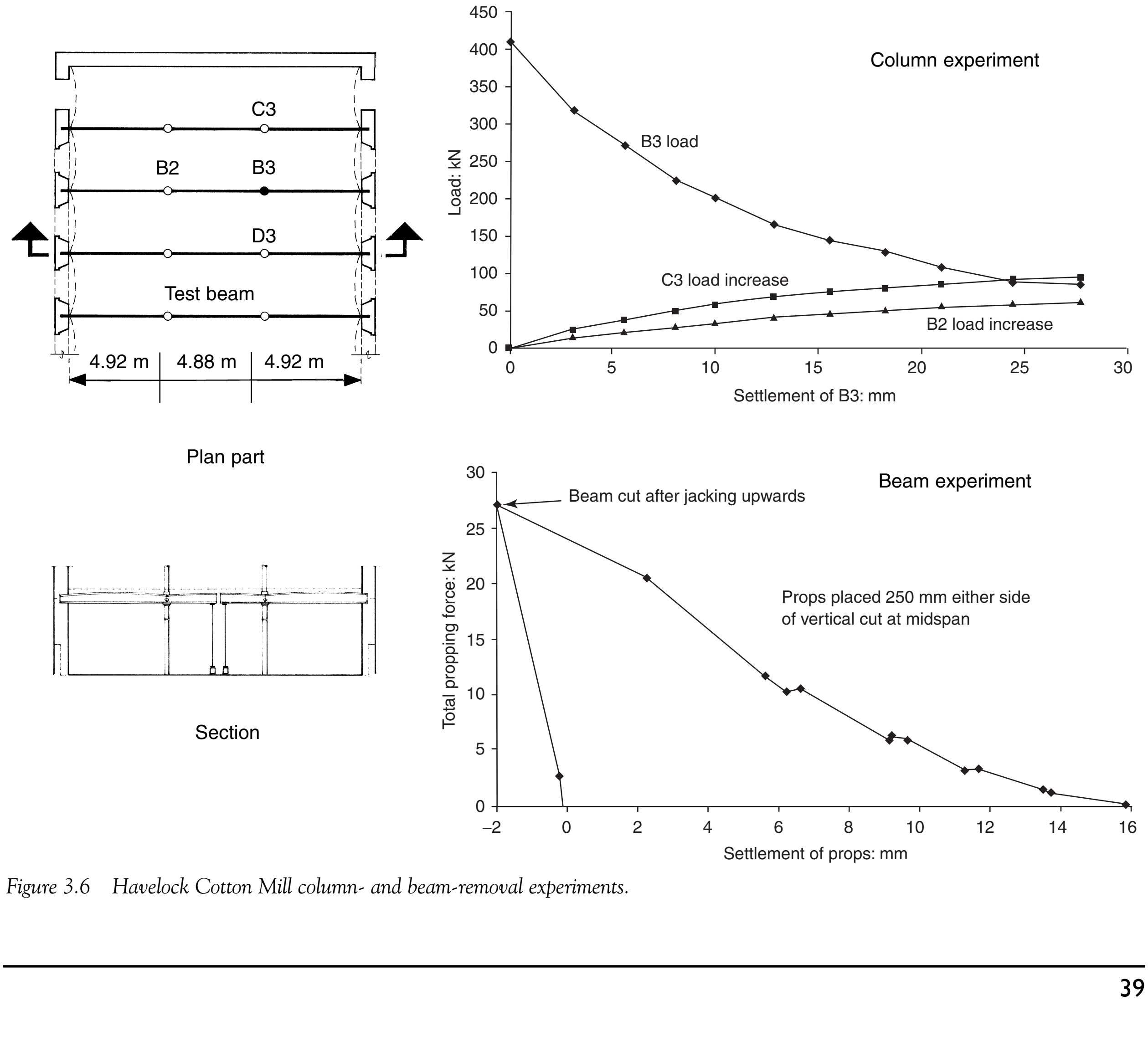


Figure 3.6 Havelock Cotton Mill column- and beam-removal experiments.

Figure 3.7 *An iron-framed building during demolition in 1970. Fairbairn's block at Bean Ing Mill, Leeds (1823–4). (John Goodchild, reproduced with permission)*

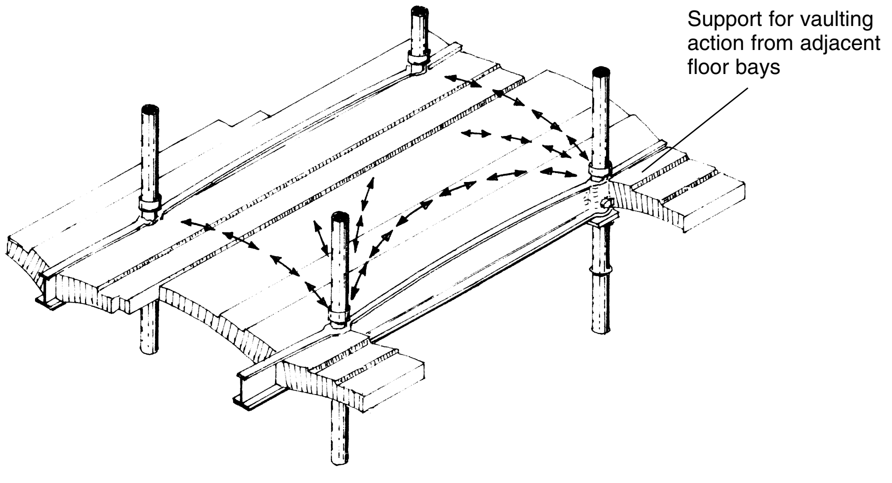


Figure 3.8 *Floor arch vaulting action.*

Some lessons may also be learnt from building failures in the United States. In 1860, 88 people were killed and 116 seriously injured when Pemberton Mill in Lawrence, Massachusetts collapsed suddenly. The inquest verdict stated that the cause was, in part, the 'outrageously defective' cast-iron columns. A historian of technology has reported that the hollow circular columns were seriously undersized and that some also had eccentric cores, giving a wall thickness of three-quarters of an inch on one side and a quarter of an inch on the other [37]. Another American author gives details of a New York building that collapsed in 1904, killing 25 men, cast-iron columns again taking the blame [38]. The Darlington Apartments were to have been 13 storeys high, but collapsed when construction of the unbraced frame reached the eleventh storey. Here, brackets from cast-iron columns supported steel beams at considerable eccentricity, producing relatively large column moments. The weakness, in this case, was a structural concept that left a frame unbraced during construction, rather than in columns which, when braced via the floors to the external masonry walls, would have been unlikely to fail.

The sudden failure of a cast-iron column might be expected to be more serious than that of a beam, so the effect of slowly removing a basement level column was tested in Havelock cotton mill in 1995 (Figure 3.6). First, the beams supported at each floor level by the test column were propped down to basement level. Next, a section of column was removed and replaced with a load cell and a hydraulic jack. With the propping to the beams removed, the ram of the jack was lowered slowly and the effect on the structure monitored. About three-quarters of the load in the column was shed with a downward displacement of less than 30 mm. Most of the load was shed transversely via the arches rather than longitudinally within the frame of which the column was a part.

From the load/displacement curve it is clear that complete redistribution of the column load was not quite achieved before the end of the test. Had it been considered safe to continue the test, another load shedding mechanism reliant on catenary action might have become apparent at a much greater displacement. The test was stopped before this stage could be reached because it was feared that a beam end might fracture, with unpredictable consequences. Structural modifications to the building in accordance with the principles described in the next section would have enabled removal of the column without risk of disproportionate collapse.

Improving robustness

Most floor arches would have little resistance to an upward accidental loading of 34 kN/m^2 to BS 6399: Part 1, but would, if strengthened to resist such a load, transfer its effects to the supporting cast-iron beams. These would then need to be strengthened to cater for upward bending, in line with the 'key member' method of assessment referred to in Approved Document A. Strengthening of arches has been achieved by the provision of a structural screed of lightweight concrete in place of stone flags and ash fill. This increases the effective depth of the floor within which arching or vaulting can take place. Reinforcement in the screed may be detailed to provide an effective tying action in two directions.

The guidance in Approved Document A focuses on the provision of robustness to counter an undefined accidental event. To safeguard a building against a specific risk such as a gas explosion, it might be advisable to leave some areas of arch unstrengthened to permit venting of an explosion. Greater blast pressures will be developed in a uniformly strong structure with no venting. The large window openings found in textile mills would, of course, also act as vents for a gas explosion.

In using the 'bridging' method, it is necessary to consider carefully the means by which a removed element, such as a basement level column, is to be bridged (Figure 3.9). If the bridging is by catenary action, displacement will be large and a flexible structure is essential. Modern reinforced concrete and steel-framed buildings with relatively shallow and/or ductile floor members are well tied together and have the necessary flexibility for 'catenary bridging'. On the other hand, with the more 'brittle' but deeper and stiffer floor construction of nineteenth-century mill buildings, bridging by arching is far more sensible. The choice of form of bridging will depend to some extent on whether the structure on either side of the bridging elements is better able to resist compression than tension. Arch floors are, of course, compression structures.

In certain situations columns and piers may be at risk from vehicle impact, or perhaps vandalism. If the column is checked as a 'key member', the 34 kN/m^2 accidental loading of BS 6399: Part 1 should be applied to the face area of the column, to which area should be added the area of any surviving attached cladding or partitioning.

Although arching or three-dimensional vaulting action may be used for bridging purposes, horizontal tying to the floor is still needed parallel and perpendicular to the frames for a number of reasons. First, the external walls need to be restrained

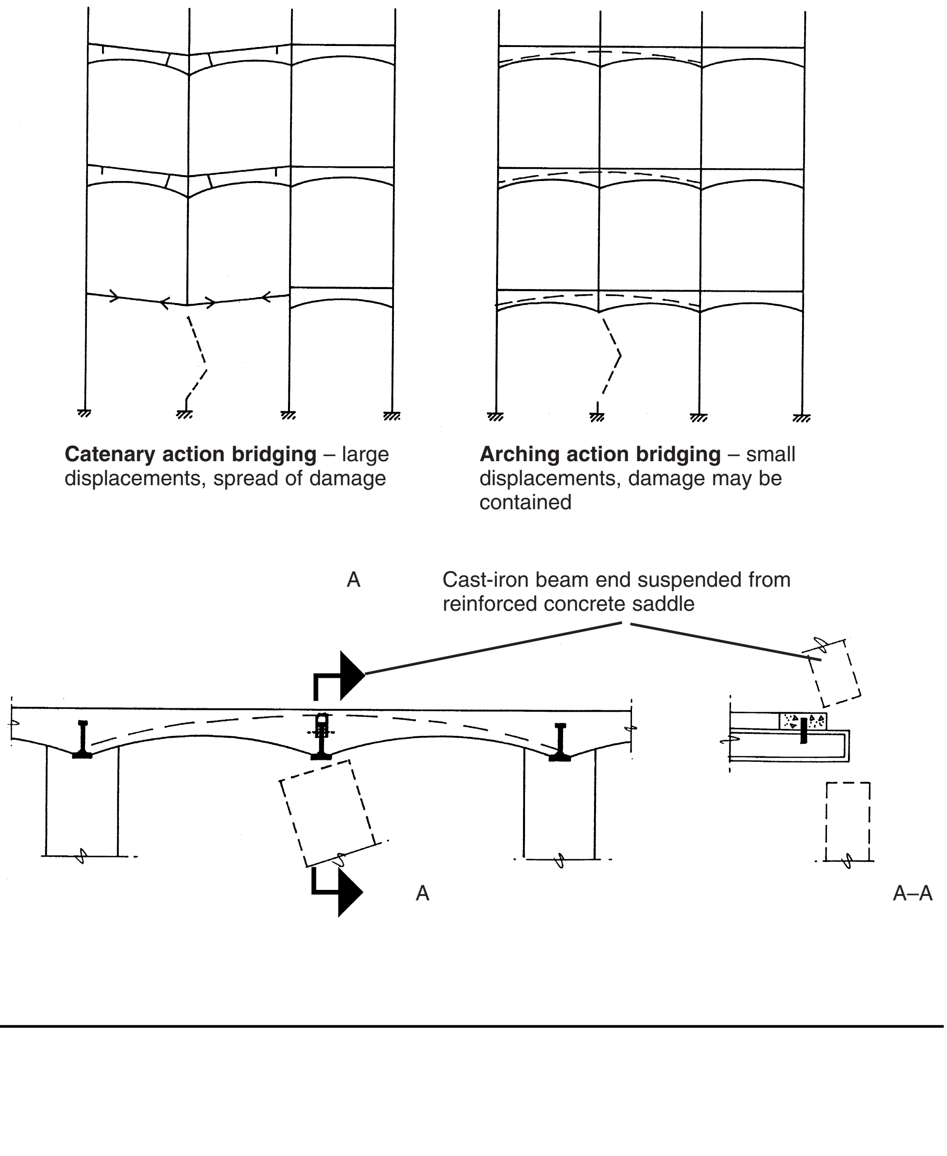


Figure 3.9 Bridging mechanisms after loss of vertical support.

laterally. Second, if a cast iron beam breaks, tying to hold together the parts of the beam and to contain the effects of a sudden brittle fracture is desirable. Finally, even ties high in the floor section may be effective in supporting the arches that neighbour a removed arch bay, though with a much reduced load carrying capacity.

The resistance of modern masonry construction to accidental loading (represented by a load of 34 kN/m^2) depends on reinforced concrete floor slabs sandwiched between the inner leaves of the external walls to each storey. The walls arch vertically between the slabs under lateral loading and the slabs tie the walls back into the building. The resistance of the walls to lateral load depends to some extent on the inner leaf vertical pre-compression due to loads from above, commonly calculated on the basis of dead plus one-third of imposed live load.

In the side walls of an iron-framed textile mill, the cast-iron beam ends act as springings to the piers which arch vertically under the lateral accidental loading. The beam ends are also the points of anchorage from which the piers are tied back into the building floors. Should the beams break in their span under upward or downward accidental loading, provision must be made for continuity of the tying action (though, in some cases, a double storey height pier may be able to bridge vertically across a removed tie). Beam to column connections in iron-framed buildings are quite weak and a continuous reinforced concrete tie across this joint will usually be required.

The gable walls of typical mill buildings are not usually tied via beam ends in the same way as the side wall piers. However, continuous thrust beams will be found, built into many gables to support the end arch span. Even where ties to the thrust beam are high in the section, these can be effective both in preventing spread of the end arch span and in restraining the gable wall. Internal masonry buttressing to the gables can be provided if necessary, well tied to the building floors. Where a reinforced concrete floor screed is to be laid over brick arches, reinforcing bars resin-anchored into the external walls, prior to concreting, may be used to provide additional restraint.

In common with many other structures, the iron-framed textile mill is at its most vulnerable, in terms of stability, when incomplete. Both the details of any structural alterations and the methods by which they will be undertaken should be approved by an engineer, taking full account of the whole structure considerations already outlined.

4. Structural inspection and element appraisal

General

Details of several recent publications that deal with the survey, inspection and appraisal of existing structures are given in the references at the end of this design and practice guide. This guide is narrower in scope than these more general guides and concentrates on the special characteristics of a particular type of building. Although it is vital to consider the structure as a whole, for convenience this chapter deals with each of the main elements of an iron-framed textile mill in turn.

The lower floors of a mill awaiting refurbishment may be occupied but the upper floors may be empty and in a dilapidated and, perhaps, potentially dangerous state. Lack of maintenance can quickly lead to leaking roofs and defective drainage, the serious effects and implications of which may perhaps not be apparent to tenants occupying the lower floors. If survey and inspection work is carried out by a person working alone, he or she should 'report in' at agreed intervals. It is a good idea to walk around the outside of a building to get an idea of its general layout and boundaries before entering.

Water is probably the biggest cause of structural problems. Lime mortar in jack arch brickwork may be leached out as water percolates down through a building, perhaps weakening a floor to near the point of collapse. A solid floor may, therefore, not be quite as safe as it appears with timber ends to floors, ceilings and roofs rotten and iron beam ends perhaps severely corroded. It is, therefore, important to view construction from below before stepping onto it from above.

Other hazards in semi-derelict mills include insecure loading bay doors, holes in floors and unsound trap doors, lift shafts with no lifts, timber stairs between floors with missing or rotten treads, weakly attached external fire escapes, defective services, inadequate lighting, dirt and dust, and the general debris of previous occupants. On the other hand, a well-maintained building may have none of these faults and it should not be supposed that textile mills are more prone to defects than other old buildings.

Walls

'Fireproof' textile mills were more expensive than the timber floored alternative and their walls are often very well built of solid brick or stone bedded in lime mortar. The masonry is usually bonded across the wall or pier thickness with a core of built masonry rather than of loose rubble. Although the compressive strength of vertical supports should be checked under proposed floor imposed loads, the stresses in fairly massive masonry wall piers are generally low. Cast-iron beams usually have wide bearings extending deep into the wall piers and rest on padstones, although confirmation of beam bearing details will be needed as part of a robustness check.

As well as a visual inspection, verticality checks are made easily by plumb line or theodolite. Wall thickness usually reduces at the inner face from the lower to the upper storeys. Taking levels or measuring the slopes of floors and cills is useful in evaluating distortions due to possible foundation movement. Significant movement may have occurred with little cracking of the masonry. Damage due to foundation settlement may only be apparent where there is an obvious structural discontinuity, such as a series of loading bay doors over the height of the building (Figure 4.1).

Some lime mortars set over a long period. This means that the explanation for irregular bed joints in a heavy wall could be the squeezing of lime mortar during construction rather than any problem at foundation level. Bed joints were made as thin as masonry units, sometimes of irregular size and shape, allowed. Lime mortar masonry is better able to accommodate movement than either brick or stone masonry in stronger cement mortars. Re-pointing or raking out and filling of weathered joints should therefore be done using materials compatible with those already present.

Where bulging or separation of masonry leaves has occurred, opening up works may be needed to determine the quality and bonding of the wall or pier construction (Figure 4.2). In stone masonry, it may well be possible to identify a pattern of through-stones which tie together the inner and outer leaves for each bay. Good practice in West Yorkshire, for example, was to build through-stones into solid walls at staggered centres of 3 ft horizontally and 3 ft vertically. Full width stone lintels and cills are common too. In the event of inadequate connection between inner and outer leaves, the basic options are to partially rebuild or to drill through and fix suitable ties (Figure 4.3).

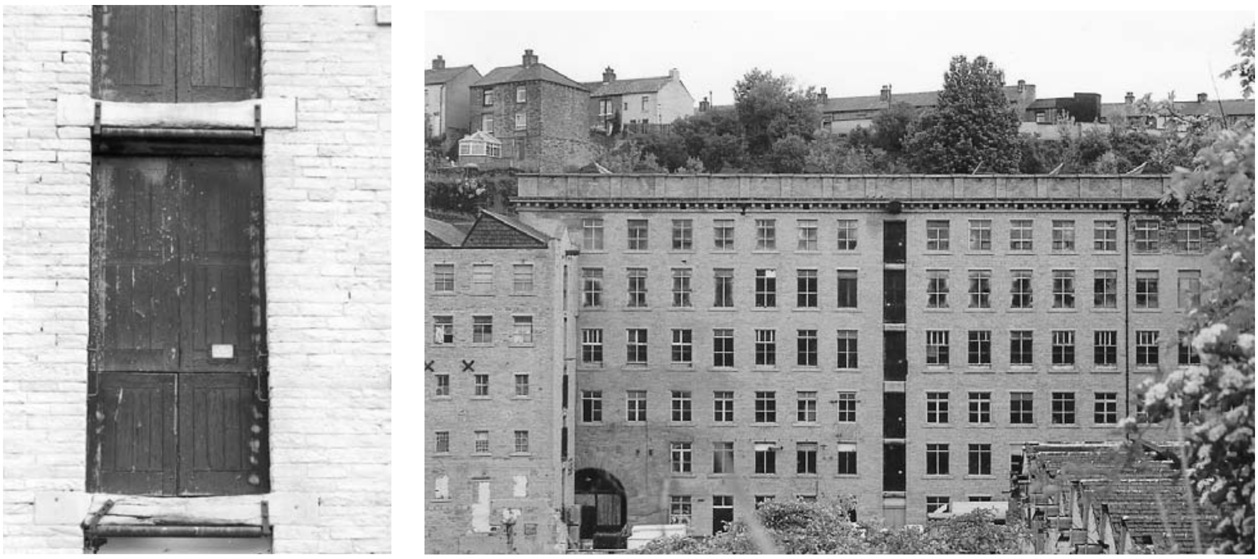


Figure 4.1 F Mill, Dean Clough, Halifax, 1858. Differential settlement to the front elevation is apparent in the form of cracks to the stone lintels across the full height loading bay doors. Note also the damage caused by the corrosion and expansion of iron fixings. (Stuart Millns)

Figure 4.2 Poorly carried out structural alterations to a wall. Bricking up of openings prevented early collapse of this 1830s mill with cast-iron columns and timber floors. Complete rebuilding of the wall was required. (Stuart Millns)

Figure 4.3 Separation of inner leaf brickwork from outer leaf stonework, due to the combined effects of low srength brickwork, poor bonding and settlement. (Stuart Millns)

Foundations

Wall foundations are usually stepped brick or stone footings and column foundations are stone block on a stepped brick or stone plinth. It was accepted that buildings of very considerable weight would settle during and after construction in most ground conditions. Therefore, signs of foundation movement do not necessarily indicate ongoing structural problems. It is often assumed that the movement ceased long ago and this is frequently the correct diagnosis. Structural monitoring involving accurate measurements over a period of about a year may be needed if there is any doubt.

A change in floor imposed loads may have little effect on ground-bearing pressures Other influences are more likely to disturb the equilibrium of the foundations, shallow footings being the most susceptible to disturbance. Water may again have serious effects, either directly due to defective drainage or indirectly through the action of tree roots.

Cast-iron beams

From beneath a jack arch floor, only the edges and soffits of the beam bottom flanges can be seen. These may be 1–1.5 in. thick and from 3–12 in. wide, with distinct square corners perhaps blurred by several coats of paint. A bottom flange of variable width (wider at mid-span) and wraparound beam to column head details will establish the material as cast iron. Rivet heads, on the other hand, are a strong indication of wrought iron.

The cast iron of the nineteenth century is flake graphite or grey cast iron. It is relatively weak in tension, though still very strong in compression, with little ductility compared to wrought iron or steel. If visual inspection is insufficient to confirm that a section is cast, a piece of 0.5 in. diameter taken from a lightly stressed area with a non-percussive rotary core drill can be polished, etched and examined under a metallurgical microscope. Brinell hardness values for the same polished surface give an indication of the grade of the iron in terms of modern material standards.

Most nineteenth-century castings were moulded in 'green sand', a blend of clayey sands able to bind together and retain the shape of a pattern under the heat and pressure of molten iron. The pattern would be of wood, made in demountable sections and with tapered flanges, to enable it to be withdrawn from the sand without damaging the mould. The moulds were finished off with a thin layer of fine 'facing sand' and then a coat of combustible 'blacking' which burned off and acted as a barrier to prevent excessive fusion of the iron and sand. I-section beams were moulded with their webs horizontal and T-section beams usually with their stems vertical. Large sections would be bedded into the foundry floor, the top part being formed using a removable moulding box in perhaps more than one section.

The raw materials for structural castings comprised a blend of pig irons with perhaps a little scrap iron and wrought iron. These would usually be remelted at the foundry in a cupola or small furnace from which casting ladles would be charged. The molten iron would be poured into a reservoir to maintain a head during casting. The mould would be fed continuously under gravity from the reservoir, via a number of channels, with a large casting of several tons being completed in only a few minutes. Vertical vents or risers were provided to allow mould gases to escape and, with the mould deliberately filled to overflowing, to trap any slag or mould debris that had risen to the surface of the casting. Quality control usually involved the casting of small test bars from the same cupola and testing them in bending.

Ventilation of gases generated during casting relied to a large extent on the porosity of the mould. The foundry floor sand, for example, would be compacted in layers over a layer of granular material. This would then be 'vented' to the layer of granular material using a thin metal spike or 'vent wire' pushed through at around six inch centres. Inadequate venting, perhaps due to excessive compaction of the moulding sand, would result in surface blowholes, particularly to the top of the section as cast. These are very common but rarely structurally significant, revealing something about the method of manufacture and the mould material but not very much about the properties of the iron.

More serious is the far less common contamination of a casting with large detached pieces of mould or the presence of a cold joint due to an interruption in casting (Figure 4.4). These gross internal defects are rarely apparent from a visual inspection and are difficult to detect by other means, making the use of a large factor of safety necessary. However, the safety factor does not remove the need for inspection of surfaces for more serious flaws such as fractures, an operation best carried out after removal of paint and with good lighting. In the unlikely event that a gross casting flaw is found, or if a beam has suffered severe mechanical damage, a visually intrusive repair may be unavoidable. An iron frame is intolerant of differential settlement, which in extreme cases may cause beam ends to fracture over or near to a support (Figure 4.5).

Significant loss of section due to corrosion of cast-iron beam ends may exceptionally occur where ends are built into external solid masonry walls. Good embedment of beam ends into supporting wall piers is very important for the stability and robustness of the building. At locations where beam end details are checked, exploratory holes formed in the masonry should be solidly built up after the investigation.

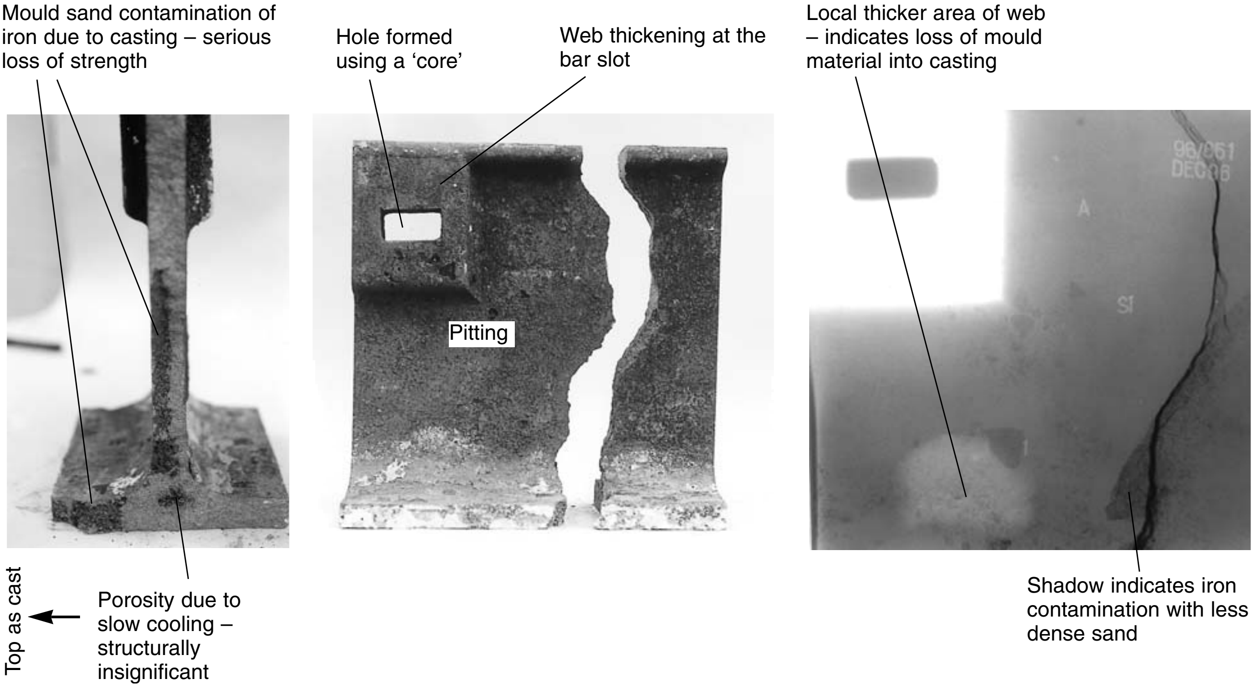
Cast-iron beams should be assessed using elastic analysis. However, the permissible tensile bending stresses quoted by different authorities vary widely. A summary of the results of recent tests on several batches of nominally matched cast-iron beam specimens are presented here for comparison purposes (Figure 4.6).

The strength values show wide scatter and there is a relatively poor correlation between minimum 'modulus of rupture' and tensile strength. Beam strength assessment based on material sampling and testing is therefore not likely to give higher permissible bending tensile stresses than those given in BD 21/97 [24]. Before either proof-load testing of floors or the taking of materials samples for laboratory testing is undertaken, very careful thought should be given to the potential usefulness of the results. Continuous monitoring of strain gauges or displacement gauges to the bottom flange of cast-iron beam is required during a test. It must always be possible to stop a test quickly, if necessary, before a predetermined safe strain level is exceeded. Contingency arrangements must be in place to allow for the possibility of a beam failure.

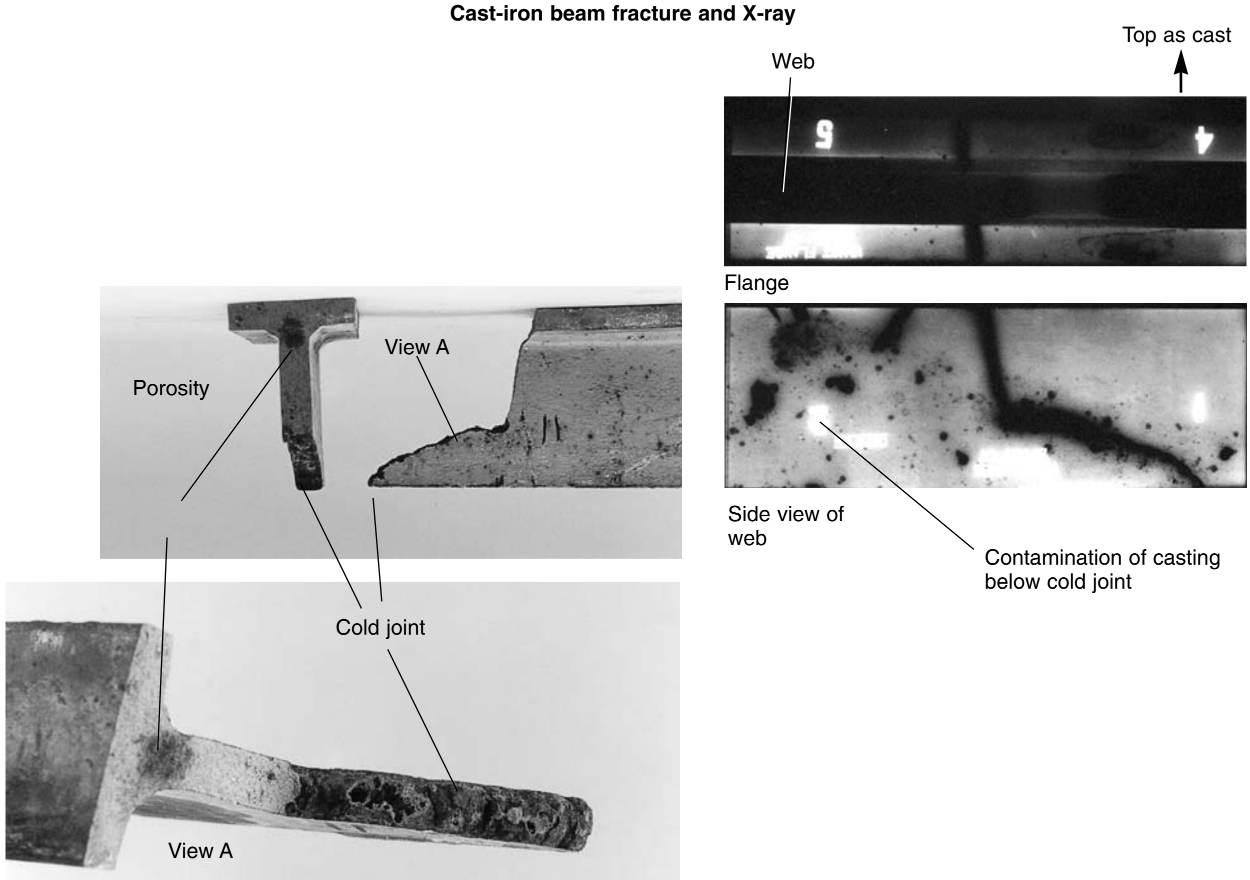
Cast-iron columns

The cast-iron hollow circular section is the most common form of mill column, with a diameter of 6–12 in. and with a wall thickness of 0.5–1.5 in. Bolting faces for drive shaft brackets are often present just below the beam seating plate. A rough vertical line on opposite faces of a column indicates that the section was cast horizontally in a two-part mould.

The moulding and casting of columns was very similar to beams. Most circular columns with a diameter of up to 12 in. were cast horizontally, with the hollow centre formed using a core comprising a bar wrapped with rope and covered with mould-



Cast-iron beam fracture and X-ray



Cast-iron joist fracture and X-ray

Figure 4.4 *Defects in cast-iron members.*

Figure 4.5   Inverted T-section cast-iron beams. As exposed in the 1797 Marshall, Benyon & Bage Flax Mill in Shrewsbury. Some beams are cracked where continuous over an intermediate column and a splice repair has been made using bolted wrought-iron straps. (Stuart Millns)

T-section cast iron joists from Covent Garden warehouse

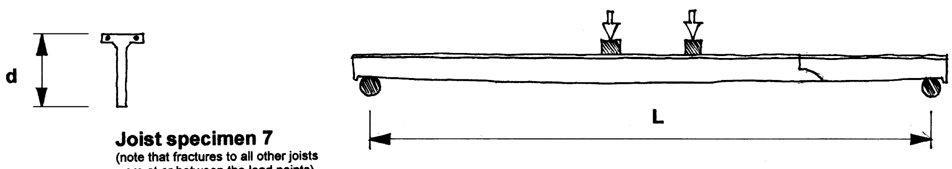
nominally matched specimens (series 1)	test span L (m)	depth at fracture d (m)	section modulus Z_t (mm^3)	'modulus of rupture' f_{bt} (N/mm^2)	tensile strength f_t (N/mm^2)
1	3.49	0.177	1.77E+05	269	
2	3.52	0.178	1.79E+05	258	
3	3.54	0.181	1.90E+05	254	
4	3.65	0.182	2.04E+05	243	
5	3.63	0.181	1.89E+05	237	
6	3.52	0.176	1.78E+05	234	
7A	2.73	0.176	1.76E+05	227	
8	3.54	0.179	1.83E+05	194	
9	3.55	0.178	1.76E+05	194	
10	3.55	0.179	1.77E+05	177	
11	3.66	0.177	1.69E+05	157	
12	3.46	0.178	1.83E+05	136	205
7	3.54	0.164	1.51E+05	66	
averages (excluding specimen 7)			***1.82E+05***	***215***	

The longer broken part of specimen 7 was re-tested as *7A* over a shorter span. The tensile strength value is an average of two tests on 12mm diameter bars cut from the joist flange.

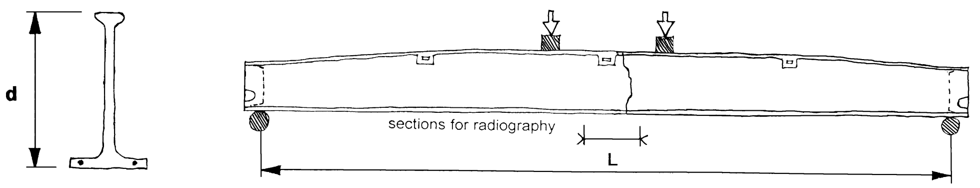
I-section cast iron beams from Havelock Cotton Mill

nominally matched specimens (series 2)	test span L (m)	depth at fracture d (m)	section modulus Z_t (mm^3)	'modulus of rupture' f_{bt} (N/mm^2)	tensile strength f_t (N/mm^2)
13	4.66	0.422	2.52E+06	186	
14	4.66	0.419	2.41E+06	173	
15	4.66	0.420	2.47E+06	153	
16	4.00	0.418	2.36E+06	153	186
17	3.99	0.417	2.40E+06	142	175
18	4.66	0.420	2.41E+06	141	
19	4.05	0.424	2.60E+06	131	202
20	4.01	0.422	2.37E+06	130	177
21	4.00	0.420	2.43E+06	130	
22	4.66	0.422	2.45E+06	128	
23	4.03	0.417	2.42E+06	127	158
24	4.66	0.420	2.28E+06	122	183
25	4.66	0.420	2.43E+06	105	198
averages (excl. specimen 25)			***2.43E+06***	***143***	***180***

The tensile strength values are averages of two tests on 16mm diameter bars cut from a piece of the bottom flange of each beam close to the position of the beam fracture.

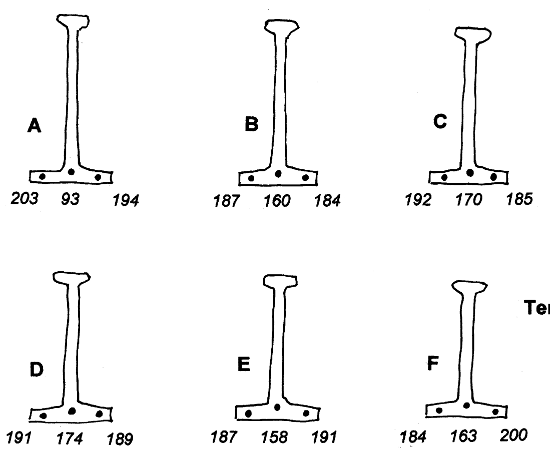


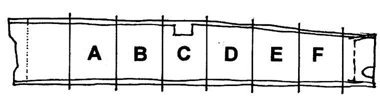
Joist specimen 7
(note that fractures to all other joists were at or between the load points)



Beam specimen 25

Cast iron bending specimen sections and test arrangements





Tensile test results (in N/mm^2, from 16mm dia. bars) from one half of beam specimen 25

Figure 4.6 Cast iron beam and joist test results.

ing sand. The bar might be hollow for mould ventilation purposes but, in any case, the core as a whole would be buoyant in the denser molten iron. A core would certainly be well fixed at its ends but would tend to bow upwards between intermediate restraints placed too far apart in the mould. This results in a thinner section to the upper part of the column wall as cast, where holes due to entrapped gas will also tend to be found (Figure 4.7).

Occasionally, a column will distort during casting and will be found bowed or out of plumb when inspected in the building. Capacity must be assessed taking such defects into account. Columns may have suffered impact damage from fork-lift trucks or warehouse trolleys and should be inspected as carefully as beams for cracks. Splicing, banding, crack stitching or concrete encasement may be necessary. Smaller section columns usually support the upper floors. Sometimes the outside diameter of the columns is constant over the height of the building but the column wall thickness reduces with height.

Cast iron has very good corrosion resistance, but where columns serve as rainwater downpipes or even as steam pipes, internal corrosion may be severe. Connections and attachments to a column that are associated with these uses may be apparent on inspection. Investigation should concentrate initially on the worst cases. Corrosion may increase the 'effective' wall thickness, although the thickness of sound metal is reduced. It may therefore be better to take a small core rather than measure the thickness via a drilled hole. Column bases on damp foundations are another potential problem area.

No significant testing of real cast-iron columns, complete with casting defects and with a variety of end conditions, has ever been carried out. The Gordon–Rankine formula provides the basis for the most enduring of the capacity checking methods. It is a semi-empirical fit to a very extensive series of tests by Eaton Hodgkinson in the 1830s on relatively small column and strut sections. Straight line formulae have been popular in the United States, used with large safety factors after the Darlington Apartment collapse, and based on a small number of rather inconclusive tests carried out for the New York Building Department. A wide range of answers may be obtained from the different capacity checking methods available (Figure 4.8, [39]). It is

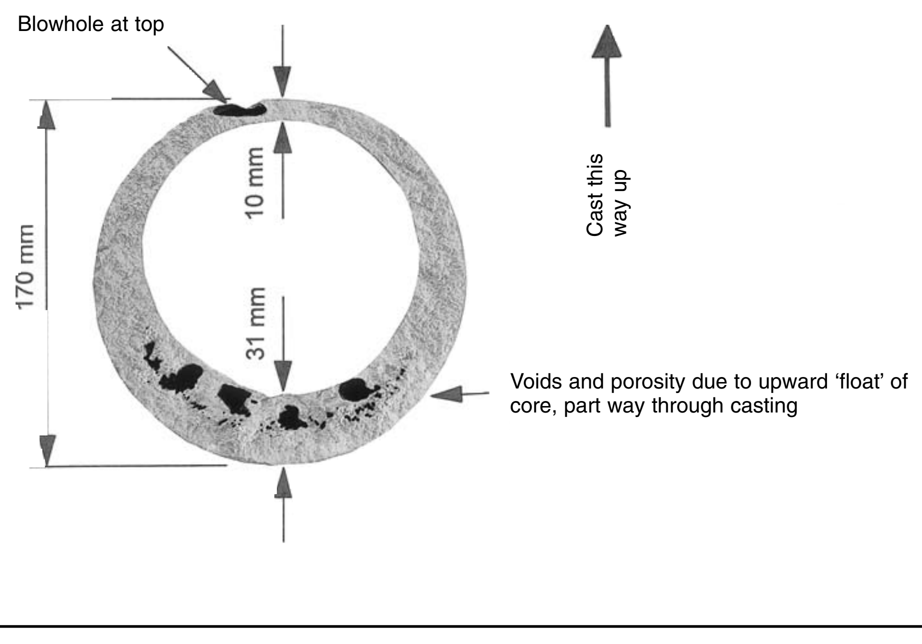


Figure 4.7 Cast-iron column fracture.

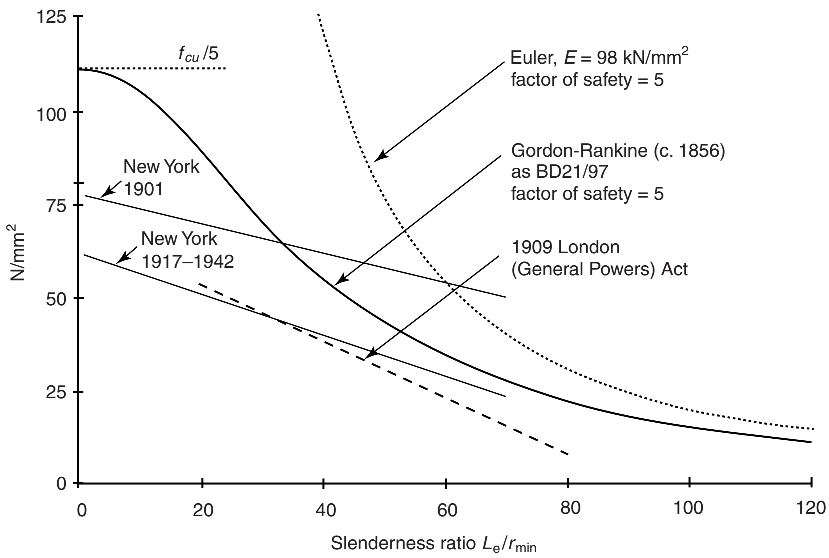


Figure 4.8 Cast-iron column design capacity curves.

recommended that cast-iron columns in mills should be assessed using the Gordon–Rankine formula following the approach of BD 21/97 [24].

The Gordon–Rankine formula is for columns with axial loads, though a large factor of safety may be said to cater for 'small eccentricities'. Where eccentricities are significant or where column moments are taken from a frame analysis, the use of a modified version of the formula is required. The combined effects of bending and compression also need to be considered.

Arches and ties

The safety of undisturbed jack arch floors should rarely be a cause for concern, as the example calculation in the next section shows. In textile mills, however, with a span of 9 or 10 ft and span-to-rise ratio of around 10, the shallow arch exerts a considerable thrust and is sensitive to disturbance. The arch soffit profile is invariably a circular arc and the brickwork, though thick at the springing, usually reduces to the thickness of a brick on its edge at the crown (Figure 4.9).

Even where the brickwork may be of rather poor quality, the use of header bricks produced a bonded construction and, over the flattest and thinnest central part of the arch, slate was often used to pack the joints. This packing would minimize the settlement of the arch on removal of centring that might otherwise be a problem due to the squeezing of soft lime mortar. With packed joints, the long-term arch strength depends on the arch geometry rather than the material strength. Sometimes the original surface of the floor will have been replaced or added to, increasing the dead weight. This will be more significant for the beams than for the arches.

Sometimes settlement or flattening of an arch may be apparent, with or without cracking perpendicular to the arch span. This type of defect may date from the time of construction or may be due to overloading, deterioration of mortar joints, or lateral movement of the arch supports. Occasional examples of bad construction practice will be found, as in all types and ages of building. There is some anecdotal evidence that one 'dodge' was to lay the bricks dry over the temporary arch formers and brush sloppy lime mortar into the open joints with a broom.

Floor intact

No ties below arch soffit

Shrink rings to beam–column connections within arch brickwork

Flags and ash fill removed

Ties between columns visible

Joint between arch sections, tie bar within

Detail at arch crown

Figure 4.9 Havelock Cotton Mill, Manchester (1845), floor details.

Soffit cracks parallel to the arch span may coincide with embedded tie bar positions and diagonal cracking may indicate where heavy concentrated loads have been supported or where openings have previously been knocked through the floor. Quite large openings may have been formed for services but plugged later. The plugs are a potential source of weakness and may well be concealed by soffit paint and dirt. Local rebuilding or stitching across cracks may be needed and a lightweight reinforced concrete topping can be used as bridging across local weak spots.

In a multi-bay floor, each arch span is supported laterally by the arch spans on either side. A cast-iron 'thrust beam' may be built into the inner face of the thick gable end wall to provide a springing for the end arch bay. Alternatively, the problem of thrust on the gable end wall may be reduced where the arches to the end bay have been turned through 90° and supported on cross beams.

Where an arch has failed or has been removed as part of previous alterations, attention should be paid during an inspection to the effect that this may have had on adjacent spans (cracked, sagging?), supporting beams, columns (lateral deflection?) and walls (cracked, bulging?). If it appears that removal of part of a floor has had no effect, it may be that the remaining floor is working harder by vaulting around the opening. Thought should be given to the effect that floor openings have on floor diaphragm action.

Wrought-iron ties between the beams and columns are common and were probably provided to hold the iron frame together during erection. The ties may be positioned too high in the floor to be very effective as ties to the jack arches, sometimes being completely concealed within the arch brickwork. However, they may serve to provide restraint to the narrow compression flange of the beam (in early mills the beams may be inverted Ts without a top flange). The corrosion resistance of wrought iron is better than mild steel but much worse than cast iron, so tie bars embedded in damp masonry are particularly vulnerable. To some extent the ties would obstruct the bricklayers work and the arches were sometimes built as short bays with incomplete bonding across joints containing the ties.

Filler joist floors

Filler joist construction became the most common form of fireproof floor towards the end of the nineteenth century (Figure 4.10). Most common in textile mills are wrought-iron filler joists of 3- or 4-in deep and spaced at around 16 in in a slab of about twice their depth. Corrosion within damp masonry walls is a common problem and local opening up to expose joist ends in vulnerable positions is recommended. Rust staining at soffit level may sometimes be seen. The concrete encasing the joists is usually weak, with aggregate of broken brick, cinders, etc. In some cases, the wheels of fork-lift trucks or warehouse trolleys have been reported to punch through between joists.

The filler joists are usually placed in the bottom of the slab and are at least partially effective as reinforcement in the composite section. Tests carried out at the National Physical Laboratory in the early 1920s indicated that, under certain circumstances, the safe working load for a composite floor of this type might be twice that obtained by considering the strength of the joists alone [40]. The strength increases with the concrete cover over the joist. Guidance on the assessment of filler joist floors is contained in the obsolete permissible stress codes for structural steelwork. Instrumented proof-testing has been used with success to confirm the adequacy of a floor of this type for a specified imposed load where it was not possible to do so by calculation [39]. As for arches and ties, other examples of bad floor construction practice will be found (Figures 4.11 and 4.12)

Figure 4.10 A 4 m brick wall built directly off a filler joist floor in the c. 1908 Rugby Mill, Chadderton, Oldham. The continuous wall arches between the primary steel beams. (Stuart Millns)

Figure 4.11 A structural alteration to a mid-nineteenth century warehouse and workshop showing gross lack of fixity. Note the absence of any bolts or coach screws and the very poor packing between the column end plates and the beam ends. (Stuart Millns)

Fire resistance of structural iron-framing members

For many years there has been concern about the possibility of fracture due to thermal shock as cold water from a fireman's hose hits one side of a hot cast-iron column. Tests in Germany in the nineteenth century showed that there was apparently no problem but a few columns failed very severe fire tests carried out for the Greater London Council in the 1970s. In several actual building fires, unprotected cast-iron columns have survived intense heat, though becoming slightly bent under the combined effects of heat and load [41, 42].

Encasement can provide protection against accidental damage as well as fire protection, but it may be visually unacceptable. Alternatively, intumescent paint can protect beams and columns in an inconspicuous way. In fireproof floors, the beams are already partially encased in masonry, as may be the wrought-iron tie bars between the beams. The strength of iron reduces at high temperatures but theoretical justification of unprotected beam flanges has been achieved in one mill refurbishment project by allowing for heat absorption by the floor arches [43].

Figure 4.12 Replacement floor details. An arch floor removed and cast iron beam cut away at Manningham Mills, Bradford, and replaced with a reinforced concrete slab. Note the unsymmetrical cross-section of the beam and the means of cutting by a series of drilled holes. (Stuart Millns)

Roofs

Mill roofs will not be considered in detail here as they have few features that are not found in the roofs of other old buildings. Roof spread is quite common, particularly where a knee braced roof has been provided to maximize the useable space. Where a roof is recovered or the rafters underdrawn and insulation provided, either an increase or decrease in dead weight may need to be allowed for. Timber roof member ends bearing in damp masonry or in the vicinity of defective gutters are particularly prone to wet rot (Figure 4.13).

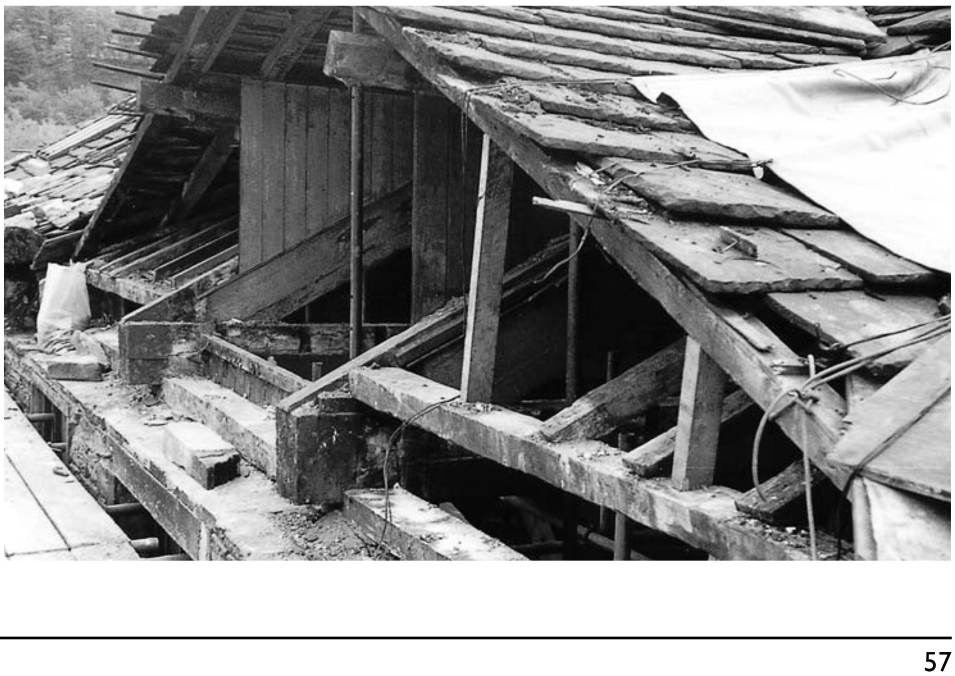


Figure 4.13 Remedial work to timber roof in progress. The roof to Greenup's Worsted Mill, c1791, Sowerby Bridge Mills. Lack of maintenance to gutters and eaves masonry often leads to rapid deterioration of mortar, requiring rebuilding of masonry, replacement of timber wall plates and remedial work to timber truss bearings. (Stuart Millns)

5. Example: Havelock Cotton Mill, Manchester

The building chosen for the example is a 150-year-old iron-framed building of five storeys with brickwork external walls and brick arch floors. The isometric view shows a range of modification works that may be needed to ensure adequate strength and robustness (Figure 5.1). The arrangement of the structure is also shown in plan and section (Figure 5.2).

Outline calculations are accompanied by a commentary. The calculations show element capacity checks for normal and accidental loadings. They are not exhaustive and do not include connection checks, for example. If a weakness is identified the choice of solution depends very much on the circumstances of the particular case, so detailed solutions for strengthening works are not presented here.

Index to calculations

	Calculation
Figure 5.3	Floor arch loads
Figure 5.4	Floor arch capacity check
Figure 5.5	Floor beam capacity check
Figure 5.6	Column capacity check
Figures 5.7 and 5.8	Disproportionate collapse checks

Floor imposed loads

For this example, the strength of the floors and their supports are checked assuming that the building is to be refurbished to provide office accommodation. An imposed load of 4 kN/m^2 allows for general office loadings at 2.5 kN/m^2 and a reasonably generous allowance of 1.5 kN/m^2 is made for lightweight partitions, services, etc. Textile mills are large structures and it is common for a single converted building to cater for several of the occupancy classes listed in BS 6399: Part 1 [21].

Floor dead loads

The floor dead load calculation is made for the original construction. Flagstones, ash fill and brickwork are allowed for in the average weight of 18.5 kN/m^3. Ideally, any alterations to the floor should not involve a significant change in the dead load. Increased foundation pressures might lead to further settlement, to which, as site tests have demonstrated graphically, this type of structure is particularly sensitive. On the other hand, a reduction in floor weight could affect the stability of the masonry piers and the overall robustness of the building.

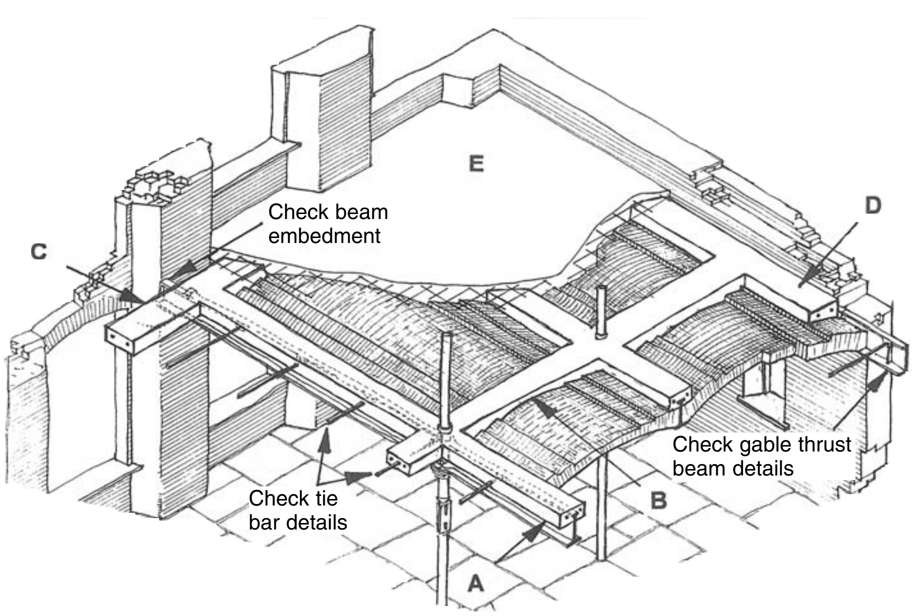


Figure 5.1 Options for increasing strength and robustness. Possible modifications: (A) transverse tie/flange for composite iron/reinforced concrete beam; (B) longitudinal internal tie/column bridging saddle; (C) peripheral longitudinal tie/pier bridging saddle (from which a beam end may be supported); (D) peripheral transverse tie, parallel to the gable wall (to which gable ties may be attached); (E) continuous mesh-reinforced lightweight concrete screed (may be cast integrally with framing elements A–D).

Notes

1. *In cast-iron framed buildings of five storeys and over, modification A will often be needed. The need for other modification will depend on the details of the particular building structure and any specific risks associated with the intended building use.*
2. *Limited horizontal framing can be introduced below the original floor level if retention of the original finishes (e.g. stone flags or timber) is required, with minimum disturbance.*

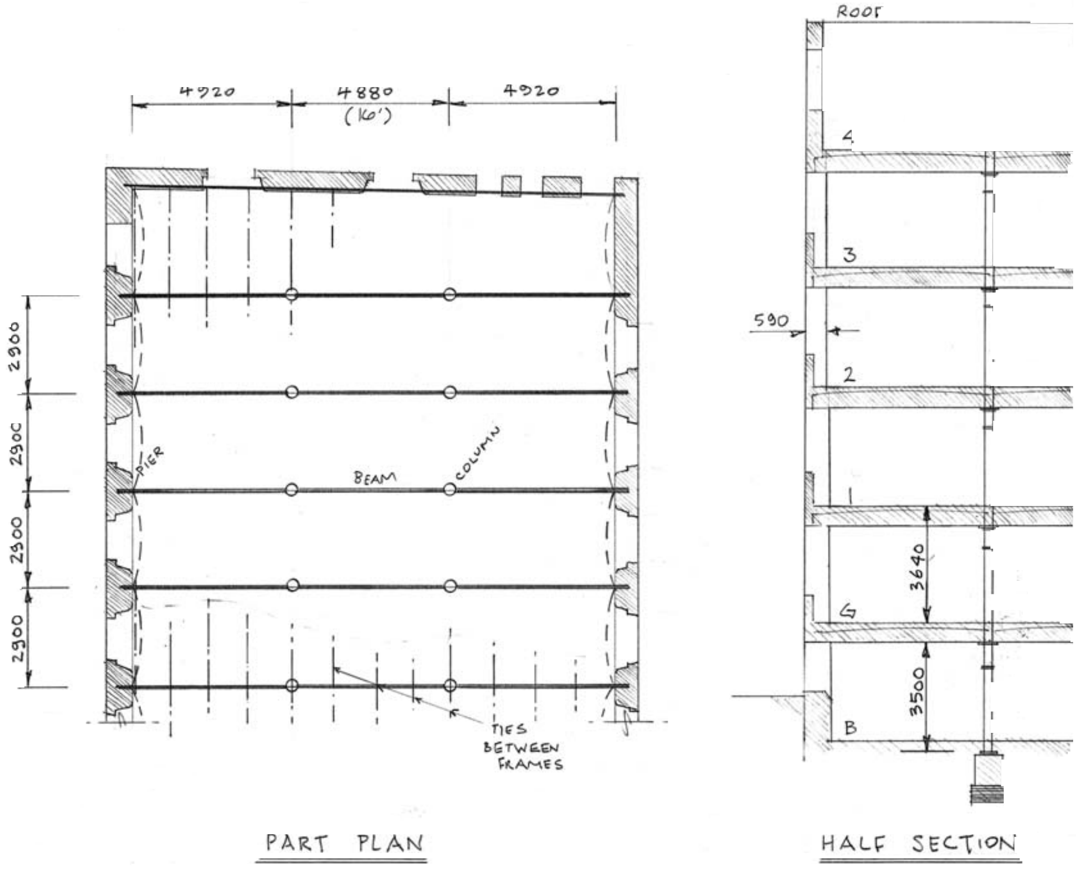


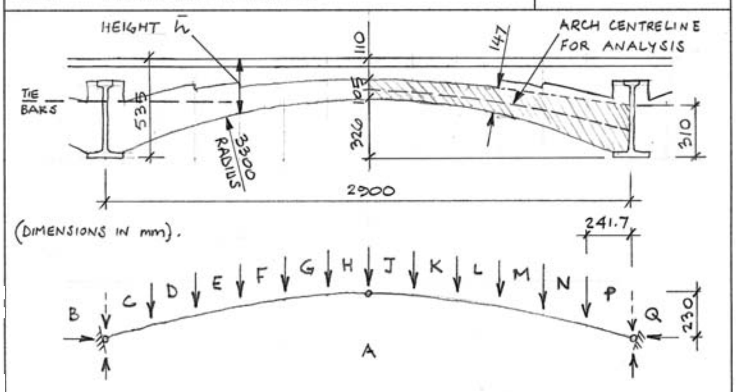
Figure 5.2 General stuctural arrangement to accompany worked example.

Floor arch analysis

From the floor section (Figure 5.3), it is clear that the arch is thinnest at the crown, increasing in thickness towards the springings. The tie bars are too high in the section to be effective, so adjacent arches therefore depend on each other for lateral support. For the calculations, a metre width of the 2.9 m span arch is divided equally into 12 pieces and the dead and imposed loads determined for each. The tabulated values include both the minimum load (0.9 G_k) and the ultimate load (1.4 G_k + 1.6 Q_k), as required for capacity checks to the limit state code of practice for the structural use of unreinforced masonry, BS 5628: Part 1 [22].

For this example, a simple elastic analysis and the conservative assumption of a three-pinned arch are considered adequate. A symmetrical load case and an unsymmetrical load case for a single arch span are considered, with balancing loadings assumed on adjacent spans. For high imposed loads, due to warehouse-type storage for example, pattern loading over several spans must be considered. First, the arch is checked with

Example calculation	Made by TS
Title FLOOR ARCH LOADS	



Element loads, kN per metre width of arch:

Dead G_K = 18.5 kN/m³ × 2.9/12 × $\bar{h}$ = 4.47 $\bar{h}$ kN

Imposed Q_K = 4 kN/m² × 2.9/12 = 0.97 kN.

Ref.	$\bar{h}$ (m)	G_K (kN)	n	W_{min} 0.9G_K (kN)	n × W_{min} (kN)	W_u 1.4G_K+1.6Q_K (kN)	n × W_u (kN)
BC, PQ	0.5	2.24	2 (× ½)	2.02	2.02	4.69	4.69
CD, NP	0.44	1.97	2	1.77	3.55	4.31	8.62
DE, MN	0.36	1.61	2	1.45	2.90	3.81	7.62
EF, LM	0.30	1.35	2	1.22	2.43	3.44	6.88
FG, KL	0.25	1.12	2	1.01	2.02	3.12	6.24
GH, JK	0.22	0.99	2	0.89	1.78	2.94	5.88
HJ	0.21	0.94	1	0.85	0.85	2.87	2.87
					15.6		42.8

Total unfactored loads for 2900 span

Dead 15.6/0.9 = 17.3 kN

Imposed 4 kN/m² × 2.9 = 11.6 kN

Figure 5.3 Floor arch loads calculations.

maximum load on the full span and, second, with maximum load on one-half of the span and minimum load on the other half (Figure 5.4).

For each load case, the vertical and horizontal components of the support reactions are determined by considering equilibrium in the usual manner. The diagrams are labelled using Bow's notation, with each force identified by the two letters that label the spaces to either side of the force. The left-hand reaction in case 2, for example, is force AC (H_{AC} = 43.9 kN; V_{AC} = 16.4 kN).

A force diagram is then drawn. Only the force diagram for the unsymmetrical load case is shown here, drawn with an exaggerated vertical scale. Each sloping line from the force diagram gives the slope of a corresponding piece of the thrust line, which is drawn piece by piece around the arch rib. In practice, these graphical calculations are best performed by drawing to a much larger scale at a drawing board. Spreadsheets offer a convenient computer-based alternative.

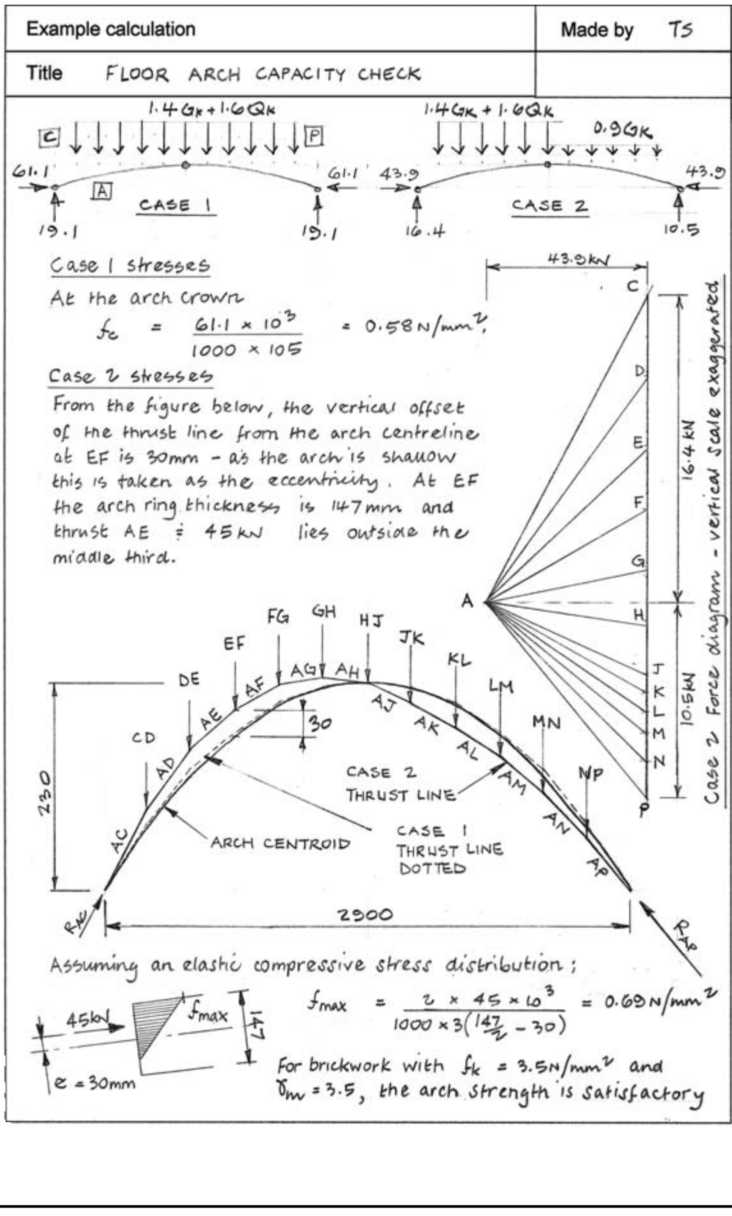


Figure 5.4 Floor arch capacity check calculations.

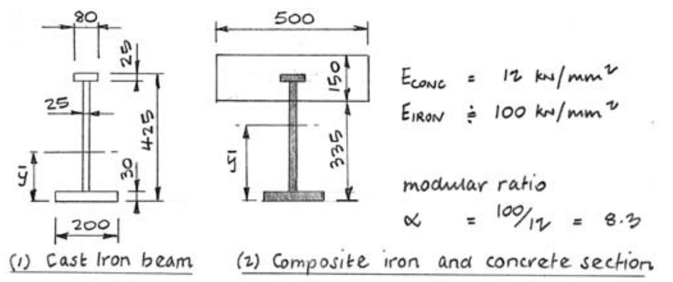
The thrust is highest for the symmetrical load case, but the eccentricity of the thrust line is small. The lower thrust obtained for the unsymmetrical load case acts at greater eccentricity and this is, in fact, the critical case, assuming that the brickwork has no tensile strength. Despite the flatness of the arch it is clear that the modest compressive stresses can be accommodated by relatively weak brickwork (10 N/mm^2 bricks in mortar designation (iv), for example).

This elastic analysis gives a 'lower bound' for the arch strength as clearly the arch is not a three-pinned arch. If an estimate of ultimate strength is required, plastic analysis may be used (with plastic hinges rather than frictionless pins). Theory and practical examples are given in Heyman's *The Masonry Arch* [44].

Section and material properties

The section properties for elastic analysis of the cast-iron beam shown in Figure 5.5 are calculated from flange tip thicknesses and an estimated web thickness measured after partially exposing one beam on site. Section property calculations are also shown

Example calculation	Made by TS
Title FLOOR BEAM CAPACITY CHECK	



$E_{CONC} = 12$ kN/mm^2

$E_{IRON} \doteq 100$ kN/mm^2

modular ratio

$\alpha = 100/12 = 8.3$

For the cast iron beam (1)

$A = 17250$ mm^2 $\bar{y} = 168$ mm, $I = 387 \times 10^6$ mm^4

$Z_{t1} = I/\bar{y} = 2301 \times 10^3$ mm^3

For the composite section (2)

The concrete flange is 'transformed' to an iron section of equivalent width $500/8.3 = 60$ mm.

$$\bar{y} = \frac{(60 \times 150 \times 410) + (17250 \times 168)}{(60 \times 150) + 17250} = 251 \text{ mm.}$$

$$I = 387 \times 10^6 + 17250(251-168)^2 + \frac{60 \times 150^3}{12} + 60 \times 150(410-251)^2 = 750 \times 10^6 \text{ mm}^4$$

$Z_{t2} = 2989 \times 10^3$ mm^3 $(1.3\, Z_{t1})$

Total unfactored loads (see p51A)

Dead	arches, etc.	4.88 × 17.3 kN/m =	84.4 kN
	self wt.	4.88 × 1.2 kN/m	5.9
			90.3
Imposed		4.88 × 11.6 kN/m =	56.6

$$\text{Bending moment } M = \frac{(90.3 + 56.6) \times 4.88}{8} = (55 + 35) \text{ kNm}$$

For (1); $f_{bt(1)} = M/Z_{t1} = (24 + 15)$ N/mm^2

For (2); $f_{bt(2)} = M_D/Z_{t1} + M_I/Z_{t2} = (24 + 11)$ N/mm^2

Figure 5.5 Floor beam capacity check calculations.

for a composite cast-iron and concrete section. The low E value for the concrete used in this example allows for creep. In practice, an effective E value should be determined according to the concrete properties, age at loading and curing conditions.

At working loads, composite action between the arch brickwork and the cast-iron beam gives a section about 1.5 times stiffer than the beam alone. An elastic analysis shows that the tension flange section modulus alters little with the brickwork present, so the brickwork contributes little to the beam strength.

Beam span and support conditions

Site tests have shown the cast-iron beam to wall pier connection to be 'partially fixed', so, in this case, the centre span between columns is the critical one. Although here the beam is assumed to be simply supported, for the composite section, the continuity provided by the reinforced concrete top flange could be taken into account if necessary. It should be noted that, without propping during construction, the composite section supports the imposed load only, not the dead load.

Permissible stresses

The beam capacity check is based upon elastic analysis under unfactored working loads. For the cast-iron beam, the combination of calculated dead and imposed load stresses falls only just outside the permissible stress envelope given in the standard for the assessment of highway bridges and structures, BD 21/97 [24]. The composite beam is satisfactory. Compare the stresses calculated here with the range of values of modulus of rupture obtained from the dozen beams taken from the building during demolition (Figure 4.6).

Column load

The calculated column dead load is greater than the column load measured, with a good degree of accuracy, during the site tests. Material weights may vary and a small amount of differential settlement, perhaps during construction, could alter the column load significantly.

Column capacity

The ultimate strength P_u of an axially loaded column may be estimated using the empirical Gordon–Rankine formula (Figure 5.6):

$$P_u = \frac{c_1 A}{1 + c_2 (l_e / r)^2} \times 10^{-3} \text{ kN}$$

For cast iron, the constants are: c_1 = 555 N/mm^2 (uniaxial crushing strength), c_2 = 1/1600; A is the cross-sectional area in mm^2; l_e is the effective length: L for pinned ends, $0.71L$ for one end pinned, one end fixed, $0.5L$ for fixed ends, $2L$ for a cantilever (one end fixed, one end free); r is the radius of gyration.

Note that the version of this formula in BD 21/97 incorporates a factor of safety of five for 'cast-iron struts which are adequately braced'.

Example calculation	Made by TS
Title COLUMN CAPACITY CHECK	

Basement level column loads

Beam dead load reactions $5 \times 90.3 = 452$ kN

Column self-weight 18×1 kN/m $= 18$

Beam imposed load reactions $60\% \times 5 \times 56.6 = 170$

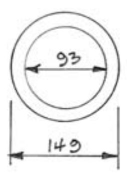
640

Column section

$$A = \frac{\pi(149^2 - 93^2)}{4} = 10\,644 \text{ mm}^2$$

$$I = \frac{\pi(149^4 - 93^4)}{64} = 20.5 \times 10^6 \text{ mm}^4$$

$$r = \sqrt{I/A} = 44 \text{ mm}$$



Column capacity

$$P_u = \frac{(10\,644 \times 555) \times 10^{-3}}{1 + \frac{1}{1600}\left(\frac{0.71 \times 3500}{44}\right)^2} = 1973 \text{ kN}$$

Factor of safety $= 1973/640 = 3.1$.

Figure 5.6 Column capacity check calculation.

Column end conditions and factor of safety

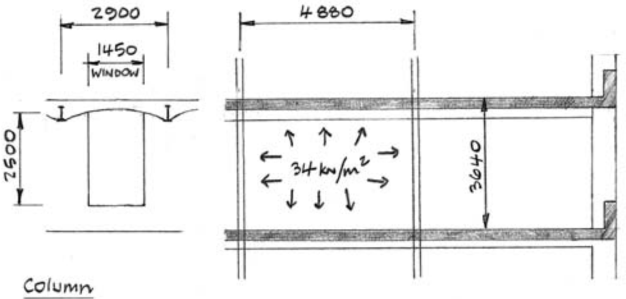
The symmetrical wraparound beam to column connections in this structure will not produce large eccentricities of load on the column. In these conditions, the safe load may be obtained by dividing the ultimate load from the Gordon–Rankine formula by a suitably high factor of safety which contains an allowance for slight eccentricity.

The columns here are more slender than in most buildings of this type and strengthening of at least the basement level columns might be recommended.

Disproportionate collapse regulations

As the building has five or more storeys and a change of use is proposed, the structure must be checked for compliance with Building Regulation A3 [16], relating to resistance to disproportionate collapse (Figure 5.7). A check is made here of the capacity of structural members to withstand a lateral accidental loading of 34 kN/m^2 (the IStructE guide '*Appraisal of Existing Structures*' suggests 17 kN/m^2 where there is no possibility of a piped gas supply being provided to the building).

Example calculation	Made by TS
Title DISPROPORTIONATE COLLAPSE CHECKS	



Column

Transverse bending under lateral accidental loading:

$$M = \frac{(0.15 \times 34\,kN/m^2) \times 3.64^2}{8} = 8.4\ kNm$$

$$f_m = \frac{8.4 \times 10^6 \times (0.5 \times 143)}{20.5 \times 10^6} = 31\ N/mm^2$$

Floor beam

Total force on the centre span beam (up or down)

$$W = 34\,kN/m^2 \times (4.88 \times 2.9) = 481\ kN$$

$$M_{UP} = (481 - 90) \times 4.88/8 = 239\ kNm$$

$$M_{DN} = (481 + 90) \times 4.88/8 = 348\ kNm$$

For the cast-iron beam

$$f_{bt(UP)} = \frac{239 \times 10^6 \times (425 - 168)}{387 \times 10^6} = 159\ N/mm^2$$

$$f_{bt\ (DN)} = \frac{348 \times 10^6}{2301 \times 10^3} = 151\ N/mm^2$$

(I and Z values for the non-composite section from page 53A).

Figure 5.7 Resistance to disproportionate collapse; column and floor beam calculations.

Columns

Many buildings of this type may have been erected a storey at a time, the external walls and the internal iron framing progressing in parallel. The cast-iron columns are usually discontinuous over the height of the building, with a simple spigot and socket connection between storey-height column sections. Here the bending strength of the thin column is adequate. However, if the upward force due to the lateral accidental loading on the floor immediately above is greater than the total column load, the column base will be lifted out of its socket. This is a problem in the upper storeys only.

Arches and beams

The floor arches would withstand the downward accidental loading, transferring the full force to the cast-iron beams. Should a beam break, the arches might stand up by an arching/vaulting action, as demonstrated in the site tests, or fail, as in the 1824 mill collapse described in Chapter 3 (Figure 3.5).

Upward pressure would blow out the crown of the arches and, perhaps, the beams would also fail in upward bending. Complete removal of an arch would deprive neighbouring intact arch bays of lateral support. A beam unable to balance an arch thrust would fail in lateral bending unless the arches were able to vault three-dimensionally between columns. With tie bars high in the section, as here, provision of a continuous reinforced structural screed is one solution.

Wall piers

The wall piers arch vertically between the built-in beam ends (Figure 5.8). The lateral failure load q per metre width of pier is largely independent of the properties of the masonry and is given by:

$$q = \frac{8nt}{h^2(1.05)} \text{ kN/m (from BS 5628 : Part 1)}$$

Example calculation	Made by TS
Title DISPROPORTIONATE COLLAPSE CHECKS	

Wall pier

Area subject to lateral accidental loading (see p55A)

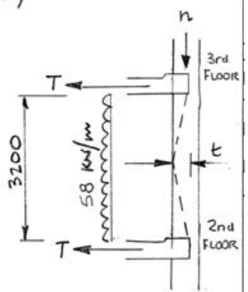
A ≑ (2.9 × 3.2) − (1.45 × 2.5) = 5.7 m²
window

Average lateral force per metre height of 3.2m high pier

w = 5.7 × 34 kN/m² / 3.2 = 61 kN/m

Tie force T = 0.5 × 3.2 × 61 kN/m
= 98 kN.

Beam embedment t = 350 mm



Second storey pier precompression

Roof truss reaction (7.5 × 2.9) × 0.5 kN/m² = 11 kN
Beam dead load reactions (p53A) 2 × (0.5 × 90) = 90
Pier self. weight ≑ (1.1 × 0.59) × 7.2 × 18 kN/m³ = 84
Spandrel panels ≑ (1.8 × 0.23) × 4 × 18 kN/m³ = 30
215
Beam imposed load reaction 2 × (0.5 × 57) = 57

n = (0.9 × 215) + (0.35 × 57) = 213 kN

q_{lat} = 8 × 0.35 × 213 × 1.1 / 3.2² × 1.05 = 61 kN/m (just adequate)

Second storey piers ok.
but third storey piers have insufficient resistance to lateral accidental loading, so provide bridging within the fourth floor.

Figure 5.8 Resistance to disproportionate collapse; wall pier calculations.

where *n* is the precompression ($0.9\,G_K + 0.35\,Q_K$); *t* is taken, in this case, as the depth of beam embedment into the wall; *h* is the clear height between the beams.

The piers are checked for a lateral accidental loading of 34 kN/m^2. The precompression in upper storey piers may be relatively low and provision may need to be made, as in this case, for an edge 'strip' to bridge over a lost support. Typical details are shown in the isometric view at the beginning of this section. More heavily loaded lower storey piers will be satisfactory.

Gable walls

In this building, continuous cast-iron 'thrust beams' provide bearings for the end arch bays and are built into the gable walls at each floor level. The gables are as thick as the piers and, although buttressed by return walls at their ends, they are 'vented' by fewer windows. Assessment should be as for the piers, as a laterally loaded panel with pre-compression. If additional ties provided to the gable extend through the full thickness of the wall, then that thickness may be used in the above formula for the lateral failure load.

References and further reading

1. Gurr D. and Hunt J. (1989). *The Cotton Mills of Oldham*, 2nd edn. Oldham Leisure Services.
2. English Heritage (1995). *Manchester Mills: Understanding Listing*, English Heritage leaflet, May 1995.
3. Binney M., Machin F. and Powell K. (1990). *Bright Future: The Re-use of Industrial Buildings.* SAVE Britain's Heritage, April, 1990.
4. Derry T.K. and Williams T.I. (1960). *A Short History of Technology*. Oxford University Press, Oxford. (Chapters on textiles, the steam engine, building construction after 1750, and coal, iron and steel.)
5. Cossons N. (1975). *The BP Book of Industrial Archaeology*, David & Charles, Newton Abbot. (Chapters on textiles, the steam engine, building construction after 1750, and coal, iron and steel.)
6. Stratton M. and Trinder B. (1997). *Industrial England*. Batsford/English Heritage. (Deals with the architecture and technology of factories and warehouses; well illustrated.)
7. Various authors (1988 and 1993). Textile mills. *Industrial Archaeology Review*, Vol. 10, No. 2, Spring 1988 (six articles) and Vol. 16, No. 1, Autumn 1993 (seven articles).
8. Giles C. and Goodall I. (1992). *Yorkshire Textile Mills, 1770–1930*. HMSO, London, 1992.
9. Williams M. and Farnie D.A. (1992). *Cotton Mills in Greater Manchester*. Carnegie Publishing, Preston, 1992.
10. Calladine A. and Fricker J. (1993). East Cheshire textile mills. *RCHME*, March, 1993.
11. Aspin C. (1982). *The Woollen Industry*, Shire Album 81, first publ. 1982; Aspin C. (1981) *The Cotton Industry*, Shire Album 63, first publ. 1981; Bush S. (1987) *The Silk Industry*, Shire Album 194, first publ. 1987; Shire Publications, Buckinghamshire.
12. Powell K. (*c.* 1981). *Pennine Mill Trail*. SAVE Britain's Heritage.
13. Hay G.D. (1986). *Monuments of Industry: an Illustrated Historical Record*. RCAHMS (for Scotland).
14. Hills R.L. (1970). *Power in the Industrial Revolution*. Manchester University Press, Manchester.
15. Gale W.K.V. (1981). *Ironworking*, Shire Album 64, first publ. 1981 and Gale W.K.V. (1967). *The British Iron and Steel Industry*, David & Charles, Newton Abbot.
16. Department of the Environment (1991). *The Building Regulations, Part A, Schedule 1* and *Approved Document A: Structure*. HMSO, London.

17. The Institution of Structural Engineers (1991). *Guide to Surveys and Inspections of Buildings and Similar Structures*. Institution of Structural Engineers, London.
18. The Institution of Structural Engineers (1996). *Appraisal of Existing Structures*, 2nd edn. Institution of Structural Engineers, London, 1996.
19. CIRIA. Structural renovation of traditional buildings. CIRIA Report 111. CIRIA, 1986.
20. Building Research Establishment (BRE). Digests include: 366 *Structural Appraisal of Existing Buildings for Change of Use*; 318, 348, 381, 383, 411 *Site Investigation for Low-rise Buildings*; 359 *Repairing Brick and Block Masonry*; 329 *Installing Wall Ties in Existing Construction*; 402 *Static Load Testing*; and also BRE Information Paper 2/95 *Guidance for Engineers Conducting Static Load Tests in Buildings*.
21. British Standards Institution (1996). BS 6399: Part 1: 1996 *Loadings for Buildings: Code of Practice for Dead and Imposed Load* (also refer to *Office Floor Loading in Historic Buildings,* English Heritage, 1994).
22. British Standards Institution (1992). BS 5628: Part 1: 1992 (+AMD 7745, July 1993) *Code of Practice for Use of Masonry: Structural Use of Unreinforced Masonry.*
23. Bates W. (1984). *Historical Structural Steelwork Handbook*, BCSA 11/84. The British Constructional Steelwork Association.
24. *The Assessment of Highway Bridges and Structures*, BD 21/97. The Department of Environment, Transport and the Regions, 1997.
25. Bussell M. (1997). *Appraisal of Existing Iron and Steel Structures*, SCI 138. The Steel Construction Institute.
26. Rees A. (1820). *The Cyclopaedia: or Universal Dictionary of Arts, Science & Literature*, published in parts from 1802 to 1820 (45 volumes completed August 1820).
27. Potter T. (1908). *Concrete: Its Uses in Building From Foundations to Finish*, 3rd edn. Batsford, London
28. Fairbairn W. (1866). Description of the removing and replacing of the iron columns in a cotton mill. *Proc IMechE*, pp. 181–185.
29. Ministry of Housing & Local Government. Report of the Inquiry into the Collapse of Flats at Ronan Point, Canning Town, London. HMSO, London, 1968.
30. *The Construction (Design and Management) Regulations 1994* and *Approved Code of Practice*. HMSO, London.
31. Swailes T. (1995). An experimental investigation of wall–floor–frame interaction in nineteenth century textile mills. *Proceedings of the 4th International Masonry Conference*, Vol. 2. British Masonry Society, pp. 370–374.
32. N.F. Astbury *et al.* (1970). *Gas Explosions in Load-bearing Brick Structures*. British Ceramic Research Association, Special Publication 68.
33. Morton J. (1985). *Accidental Damage, Robustness and Stability*, Brick Development Association (based on BDA seminars by Korff J.O.A., Sutherland R. J. M and Morton J.).
34. Swailes T. (1997). On site tests to investigate the robustness of a nineteenth century cotton mill. In *Structural Assessment: The Role of Large and Full-scale Testing*, Virdi K. *et al.* (eds). E & F.N. Spon, London, pp. 517–524.
35. *Report on the Fall of the Cotton Mill at Oldham and Part of the Prison at Northleach*. HMSO, London, 1845.
36. Private communication to the first author, 1997.
37. Wermiel S. (1995). Rethinking cast-iron columns. Building *Renovation (USA)*, Winter, pp. 37–40.
38. Friedman, D. (1995) *Historical Building Construction: Design, Materials and Technology*. W.W. Norton, New York.

39. Swailes T. (1997). The assessment of nineteenth century fireproof floors for twentieth century loadings. *Proceedings of the 7th International Conference on Structural Faults and Repair*, Edinburgh, July.
40. Etchells E.F. (ed.) (1927). *Modern Steelwork: a Review of Current Practice in the Application of Structural Steelwork to Buildings and Bridges*. Nash & Alexander, London (pp. 44–55 on 'joist concrete floors', by 'W.B.S.').
41. Barnfield J.R. and Porter A.M. (1984). Historic buildings and fire: fire performance of cast-iron structural elements. *The Structural Engineer*, Vol. 62A, No. 12, December, pp. 373–380.
42. Porter A. *et al.* (1998). The behaviour of structural cast iron in fire. *English Heritage Research Transactions*, Vol. 1: *Metals*. April, pp. 11–20.
43. Bussell M. N. and Robinson M. J. (1998). Investigation, appraisal and re-use of a cast-iron structural frame. *The Structural Engineer*, Vol. 76, No. 3, February, pp. 37–42.
44. Heyman J. (1981). *The Masonry Arch*. Ellis Horwood.