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Design for Manufacture Guidelines

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Design For Manufacture Guidelines

CIMsteel

Foreword

This document titled "Design For Manufacture Guidelines" was developed in the Design and Standards phase of the Eureka CIMsteel project and is the result of a collaborative effort of fabricators, designers and research bodies. Principal authors of the document were :

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As the title implies, the content of this document is aimed at bringing a degree of understanding of the manufacturing implications to the early design phases of a project, the main target audience of these papers being the Consulting Engineer and engineering students. However it is hoped that the contained information may also be of use to Quantity Surveyors, Architects, Estimators and Fabricators. The general contents of the document are intended to be both informative and of practical application. Where possible the document has attempted to address cost information covering both material and labour costs. This data can obviously be criticised from many directions but it was felt necessary to express information in quantitative as well as qualitative terms. The cost information presented should always be viewed as relative rather than absolute information, i.e. it should allow comparative judgements to be made but should not be seen as an absolute cost estimate as the true market cost will depend on many factors which cannot possibly be accounted for in a presentation of this nature.

Underlying the whole document is the belief that the greatest improvements to increase productivity and reduce costs will be generated by the provision of clear and appropriately detailed information, particularly at time of tender. The better the information at time of tender, the lower the risk provisions allowed by the fabricator and the lower the likelihood of subsequent claims for additional work or programme variations. Recognising the time and fee pressures which exist on the Professional Teams, this document attempts to present information in ways which can be applied either in terms of general principles or more direct applications, using information in simple look-up tables.

Acknowledgements.

An earlier draft of the document was reviewed at a one-day workshop attended by a wider section of the structural steelwork sector to broaden the nature of the contents and ensure a reasonable consensus to the messages conveyed. The authors would like to acknowledge the following people who attended the workshop or gave subsequent comments on the document :

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1. INTRODUCTION AND KEY POINTS

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INTRODUCTION

This document produced under the Eureka CIMsteel initiative is entitled 'Design for Manufacture Guidelines'. The document was written by a collaborative group which included the SCI, Fabricators and Consulting Engineers. The general aim of the document is to raise the awareness of the Designer/Specifier of the effects that basic design decisions can have on the overall cost of the fabricated steel frame and provide the provide the Designer with assistance in developing his tender scheme.

The scope of the document is as follows:

- description and capabilities of modern fabrication techniques
- raw material costs and section selection considerations
- quick reference connection design capacity tables and examples of their use
- proposed fabrication classification and relative cost tables for quantitative comparisons, with illustrative examples
- recommendations on bolting and welding
- corrosion protection techniques and typical systems with relative costs
- special considerations for lattice girder detailing
- simplified transportation constraints.

The document is intended to be both educational and functional. Thus it is intended for use by Engineering students and practising Engineers but also has relevance to Quantity Surveyors, Architects, Estimators and Fabricators.

The question may well be asked, '***If the fabricator sees that commercial advantages can be gained through rationalisation and simplification of the steel frame, why does he not take full advantage of these effects himself ?***'. The answer is that in a design and build situation he may well do so, but in the more common fabricate and construct contract the critical decisions on the basic form of the steel frame have all been taken long before the fabricator is involved and contract programme constraints usually preclude the possibility of introducing such changes. Furthermore, the fabricator only deals with a portion of the integrated building and is in no position to consider the implications of design changes on other associated trades, services, finishes etc.. Hence the originating designer is the most appropriate party to consider such influences and the earlier in the design process these principles are introduced the better the final product for all parties.

It is intended that this document will be supplemented by further work in the next phase of the CIMsteel project to include information on "Design For Construction", to extend such information into the requirements of ease of erection etc.

Discuss me ...

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What are the manufacturing implications that a designer should reasonably have considered during the preparation of tender and final designs ? Why should any such points be considered at all ?

The answers to the first question are the subject of this document.

The answer to the second question is that the more the designer understands about the construction implications of his design, the more cost effective the design is likely to be; both for the ultimate client and for all parties in the construction process, including the originating designer. The cost effectiveness will come both from direct savings and indirect costs saved through reduction of dispute over information and associated potential programme over-runs. These benefits will only be realised if the designer imparts his knowledge and intentions through complete and appropriate definition of the design requirements both at tender and final design stages.

KEY POINTS

The following Key Points have been collated to form a brief summary, the numbers and headings relate to the sections of this document where greater detail can be found.

2. FABRICATION PROCESSES

- Modern Computer Numerically Controlled (CNC) fabrication equipment is more effective with:
    - i) Single end cuts, arranged square to the member length
    - ii) One hole diameter on any one piece, avoids drill bit changes
    - iii) Alignment of holes on an axis square to the member length, holes in webs and flanges aligned not staggered to reduce piece moves between drill times
    - iv) Web holes having adequate side clearance to the flanges.
- To allow efficient production of fittings :
    - i) Rationalise on the range of fittings sizes - use a limited range of flats and angles
    - ii) Allow punching and cropping wherever possible.
- If possible select connections which avoid mixing welding and drilling in any one piece. This avoids double handling of the member during fabrication.

3. MATERIALS GRADE AND SECTION SELECTION

- The designer should rationalise the range of sections and grades he uses in any one structure. This will lead to benefits in purchasing and handling during all fabrication, transportation and erection phases of manufacture.

- Make maximum use of Design Grade 50 material for main sections. This is typically 8% more expensive but up to 30% stronger than Design Grade 43 steel. The exception is where deflection governs section selection.

  Guidance has been provided for the cost-efficiency of UB and UC sections when strength considerations govern .

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- The specification of a small quantities of Design Grade 50 or other 'special' grade material should be avoided, particularly if the proposed material has poorer welding qualities.

- Choice of fittings material grade should be left with the fabricator wherever possible.

- Structural hollow sections are approximately 60-80% more expensive than equivalent weight open sections and have additional problems associated with the connection requirements. Limitations on mill lengths should also be remembered.

4. CONNECTION DESIGN CONSIDERATIONS and
5. FABRICATION CLASSIFICATION AND COSTING

- Connections directly influence 40-60% of the total frame cost. They must therefore be taken into account during the frame design.

- Least weight design solutions are rarely the cheapest. Increasing member thickness to eliminate stiffening at connections will often be an economic solution.

- The cost benefits from an integrated approach to frame and connection design will only be realised if the fabricator is given a full package of information at tender stage. Connection styles and design philosophy must be clearly marked on drawings.

Tables are provided to allow the designer rapidly to assess the connection requirements to sustain applied loads, and whether the connection requires provision of local stiffening.

Worked examples are provided to illustrate the use of such tables.

To allow relative costs of various connection styles and material weight to be investigated, a fabrication classification system for beam and column members has been included. Tables of comparative fabrication costs are included, together with worked examples of their application.

6. BOLTS AND BOLTING

- Non-preloaded bolting is the preferred method for site connections.

  Preloaded (friction grip) bolts should only be used where joint slip is un-acceptable or where there is a danger of fatigue.

- The use of different grade bolts of the same diameter on any one contract should be avoided.

- Threads should be permitted in the shear plane and in bearing.

- Direct and indirect cost savings can accrue by only using a small range of "standard" bolts.

  Recommended Standards are:

  M20 grade 8.8 for shear connections
  M24 grade 8.8 for moment connections
  Mechanical properties to BS 3692, dimensions to BS 4190
  Fully threaded for shanks up to 70 mm long.

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- The use of fully threaded bolts generally means additional thread protrusion is visible; specifiers should be aware of this and state at tender stage where this is not acceptable.

- Washers are not required for strength when using non-preloaded bolts in normal clearance holes; they may still be specified to provide a degree of protection to surface finishes.

- When used with corrosion protected steelwork, bolts, nuts and washers should be supplied with a coating which does not require further protection applications.

7. WELDING AND INSPECTION

- The welding content of a fabrication has a significant influence on the total cost of fabrication.

- In designing welded connections consideration should be given to the weldability of materials, access for welding and inspection, and the effects of distortion. Access is of primary importance - good welds cannot be formed without adequate access.

- Fillet welds up to 12 mm leg length are preferred to the equivalent strength butt weld. Generally two fillet welds whose combined throat thicknesses equal the thickness of the plate to be connected are considered as equivalent in strength to a full penetration butt weld.

- Weld defect inspection and defect acceptance criteria should be defined; the use of the National Structural Steelwork Specification criteria is strongly recommended.

8. CORROSION PROTECTION

- In selecting a corrosion protection system the designer must consider the environment in which the steelwork will be placed and the design life of the corrosion protection system.

  If the environment does not require a corrosion protection system don't specify one.

- If a protection system is required, significant advantages are gained by use of a single coat protection system applied during fabrication. These should be specified where possible.

- Wherever possible avoid using 'named' product specifications; allow the fabricator to use his preferred supplier or even alternative preferred coating system of equal capability.

  Specification of surface conditions should relate to the condition immediately prior to painting, not bound by any time-limit from shot blasting operations.

  Example coating specifications for a range of environments are given, together with an indication of relative costs.

9. TRUSSES AND LATTICE GIRDERS

- Lattice girders and trusses are effective for medium to long spans where deflection is a major criterion and are able to accommodate services within their depth, but always consider the use of a plain rolled section beam first.

- Most lattice frames are joint critical. Never select a section for the chords or internals without first checking it can be effectively joined - preferably without recourse to stiffening.

- Always check the limits on transport before starting the design.

- Be aware that SHS are only available in limited standard lengths, normally from stockists. Long lengths may therefore need additional butt welding.

- For internal members try to detail single bevel end cuts; for Angles square cut ends are better to allow use of an automatic cropping process.

- In tubular construction use of RHS chords leads to simpler end preparation for internals than that required if CHS chords are used.

- Think about access provisions for welding of internals to chords.

- Access for painting is difficult for double Angle or double Channel members; use of SHS reduces paint area and provides fewer locations for corrosion traps to be formed.

10. TRANSPORTATION

- Police notification with associated programme and cost penalties will occur for road transport loads greater than 18.3 m long, or 2.9 m wide or 3.175 m high.

2. FABRICATION PROCESSES

FABRICATION PROCESSES

Introduction

An awareness of the processes and practices of a typical modern fabrication facility is necessary to facilitate the understanding of these guidelines. The descriptions are applicable to even medium/small fabricators if they have been recently equipped. The basic processes and associated equipment are described below together with an illustration of the capabilities of the equipment where appropriate.

Steel Stockyard

Most fabricators will have some form of raw steel stock holding facility, although it is now common practice to work on the "Just-in-Time" stock receipt principles as far as possible, only purchasing sufficient quantities of major sections to known contract requirements. Small stock holdings of plates, flats and angles for general fittings requirements may also be held and replenished periodically to maintain a minimum stock requirement. The stockyard is the start of the fabricator's logistical problems of piece identification and materials handling. The fewer variations of section, grade and, to a lesser extent length, the easier it is to locate each required piece and the fewer times each piece has to be re-handled. Such logistical problems occur at nearly every stage of manufacture and erection, hence standardisation and commonalty can produce great advantages.

Blast Cleaning and Pre-fabrication Primer

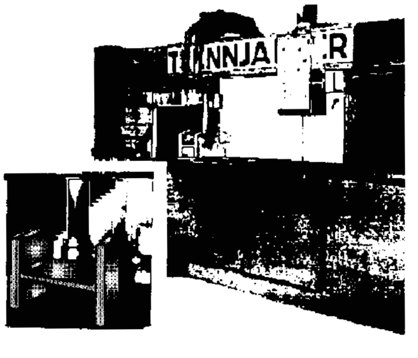
In order to remove any rust, loose mill scale etc. the sections once drawn from stock are subjected to blast cleaning. The same process occurs for plates and fittings stock. After blast cleaning, some fabricators may apply a coat of prefabrication primer to prevent rust returning during fabrication. Many fabricators will not apply such a primer as their shop conditions and throughput timing do not allow an undue amount of rusting prior to final painting and/or a post fabrication blasting will be adopted prior to final paint application. Some fabricators will perform the blast cleaning process after cutting and holing operations have been completed.

Cutting Processes

Sections, plates and fittings all require cutting to length. This can be accomplished by several techniques.

Circular cold sawing

A modern circular cold saw used in conjunction with powered longitudinal and transverse conveyor systems is probably the most popular, productive and accurate means of cutting rolled sections to length. Often used in conjunction with a length measuring device and with the ability to pivot on a vertical axis for mitre cuts, an accuracy on length



within 1 mm is normally achievable with an out of squareness of cut less than section depth /500 mm.

Band sawing.

Often used for minor rolled sections and fittings, the technique is faster but generally not as accurate as the circular saws. This is a popular method of cutting in Japan where multiple sections are cut by binding stacked sections in bundles prior to cutting.

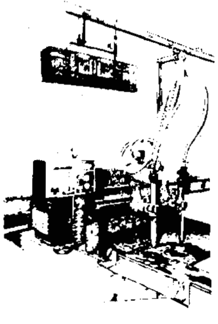
Motor-operated hacksaw.

Not as popular as either of the above but may still be in use for main units or fittings production. Not as accurate for squareness as the circular saw.

Guillotining of plates and flats.

Power operated guillotines can be used to cut plates and universal flats to length. This process can be used on all materials up to 12 mm thick, and if the machinery is capable up to 16 mm thick for sub-grade B and 20 mm thick for sub-grade C steels. For non-square cutting of greater thickness flame or plasma cutting must be used.

Flame cutting of plates and flats.



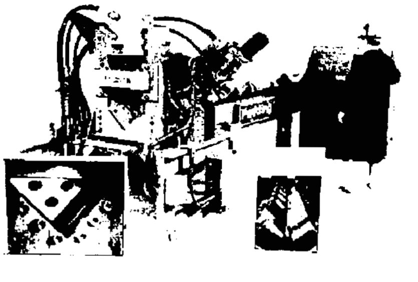
For cutting non-rectangular plates or for greater thickness of material the use of flame cutting equipment can be employed. Often such equipment is Computer Numerically Controlled (CNC), utilising software to present the proposed cutting arrangement (nesting) of several required shapes out of one larger stock plate via a computer graphic screen similar to a CAD display. The flame cutting machines often have multiple cutting heads.

Plasma cutting.

A process similar to that described above for flame cutting, but using an electric arc with compressed air creating a cutting plasma in place of the oxyacetylene flame cutting head.

Cropping of flats and angles.

For smaller angles and narrow universal flats, specialised handling and cutting machinery has been developed which will shear the piece to required length and often also punch and/or drill holes at defined pitches in each leg of an angle or face of a flat. Again CNC links allow stock lengths to be fed in, cut into variable determined lengths complete with required holing, and sorted into one of several receipt bins ready for application. Limits on thickness for



cropping and punching of holes are as for guillotine cutting, with angle legs typically limited to 150 or 200 mm maximum outstand.

Holing Processes

Drilling of Main Sections.

Whilst manual marking and drilling using radial arm drills still exists, many fabricators of even medium capacity now have CNC beam drilling lines. Typically these will be aligned with the same powered conveyor systems as the saw operations and comprise three drill heads set to drill both top and bottom flanges and beam web simultaneously, providing that the associated detailing has recognised this capability. Length measuring devices allow for position of the drill heads along the length of the member and each drill head has a choice of bit size and can traverse across the flange or web location respectively. In this manner all required positions of drill location can be defined and manipulated by the CNC programming software.

Drilling of Section Fittings.

Short sections or fittings which cannot be readily mounted on the conveyor systems used for main units, typically all sections less than 2.0m long or requirements to drill the stalks of T sections, will be marked and drilled using manual techniques.

Holing of Plates and Flats.

As noted before in "cutting" it is now common and permitted practice to punch full sized holes for untensioned bolts for connections to structures designed to BS 5950 or BS 449 or EC3, except under certain restrictions where locations of plastic moment action or yield line assumptions influence the design. Typically punching is allowed in material thickness up to the size of hole diameter being formed. Further guidance on acceptability of punching can be found in the relevant design codes. Where holing is required in materials of greater thickness, or where specifically required holes can be drilled. CNC machinery is available for both punching and drilling operations but a fabricator's preference tends toward punching where possible due to the increased speed of operation.

There are several CNC machines which combine the cutting and holing processes utilising combinations of plasma or flame cutting and punching or drilling techniques ; these combinations are usually limited to square cutting to length of defined width plate or flat combined with a holing pattern per produced fitting.

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Other Materials Preparation Processes

Coping, Flange Thinning and Stripping.

To allow beam to beam connections to maintain a constant top of steel level, and to allow access of wide flanges into narrow column webs it is often required to either notch one or both flanges from a beam or to trim the width of flanges. These processes can all be undertaken by a single or multi-head flame/plasma coping machine acting under CNC instructions. Such a Coping process is often located on the same saw and drill lines described above. Alternatively such processes will require to be manually marked and flame cut.

Vertical Surface Milling.

If the required accuracy of cold sawing of deep members cannot be ensured, or to allow for exact trimming to length and squareness of plate fabricated members, it may be required to perform milling of vertical surfaces.

Horizontal Surface Milling.

Whilst not normally required under the provisions of the National Structural Steelwork Specification, base plate to column bearing surfaces may require milling to achieve special flatness criteria, particularly for plates over 55 mm thick. Many fabricators will thus posses horizontal surface milling equipment. Certain types of milling equipment can also perform CNC controlled milling and drilling functions.

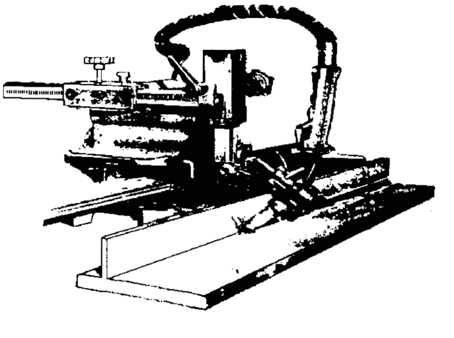
Cutting to length, intersection profile and edge preparation of Circular Hollow Sections (CHS).

Profile preparation of CHS tubular members for welded intersections is undertaken by CNC flame/plasma cutting equipment. The CHS member is rotated on its longitudinal axis, the trolley mounted cutting head is driven along the longitudinal axis and the angle of the head normal to the surface of the tube is also varied to give the weld preparation angle. By these three manipulations a full profile and prepared cut can be made. If this equipment is not available the use of manual marking using costly full-size "wrap-round" developed length templates and flame cutting and grinding would need to be employed.

Shop Assembly of Fabrications

Joining Techniques.

The two methods of joining normally employed are shop bolting and welding. Normally the preferred site joining technique is by bolting. Within the shop the controlled environment allows the use of jigs and templates to assist in positioning elements of a fabrication and a mix of methods may be employed to avoid an overload of demand on the skill requirements of the shop welders. Most fabricators will now utilise the semi-automatic MIG welding techniques with wire fed welding equipment, often boom mounted to allow ease of



access to the work place. A certain amount of manual metal arc welding may still be required to overcome particular access problems to constricted locations, but such problems should ideally be "designed out" rather than overcome. For the continuous welding requirements of plate fabrications, tractor driven submerged arc welding machines are often employed.

Mechanical Handling

Components are handled throughout the preparation, fabrication, protection, transportation and erection processes by a variety of processes i.e.. over-head cranes, jib cranes, fork-lifts, powered conveyors etc. As such, materials handling is one of the significant "hidden" costs of fabrication and an effective means of handling coupled with factory layout and production control is vital to efficient fabrication operations. Rationalisation of the selection of sections within a project should lead to a reduction in materials handling by reducing multiple handling of individual stock pieces. This will allow a degree of "batch" production even in main member preparation and hence associated fittings preparation. Commonalty of fabrication selection will also reduce handling effects during transport loading and erection processes.

Post-fabrication Painting

Many fabricators have some capability to carry out post-fabrication painting, though these facilities may be limited in their nature and capacity. There is a strong incentive to effect such protection treatment immediately after the fabrication process as this avoids multiple handling as well as a second blast treatment operation. Fabrication combined with painting facilities eliminates the requirement for a whole sequence of load building, transportation, unloading and re-loading after painting. The process of sub-contract painting has its own cost implications, but also has greater risks associated with programme delay as well as additional requirements for piece monitoring and transport load completeness checking prior to site delivery. These additional risks and associated costs should not be dismissed lightly and should have due consideration in the original selection of the corrosion protection system. Further guidance on this subject can be found in Section 8 of this document. Recent developments of shop applied intumescent paints are also now becoming more prevalent and with associated care in transport and erection handling, prove cost effective compared to site application.

3. MATERIAL GRADE AND SECTION SELECTION

MATERIAL GRADE AND SECTION SELECTION

Introduction

There are hidden costs in fabrication handling raw materials, during stockyard operations, preparation of material, fabrication, transport loading, re-handling for painting, final off-loading and handling for erection. There are other costs such as the purchasing of small quantities of diverse sections. All these costs can be reduced if the section range within a project is rationalised to obtain repetition of like pieces. It is particularly important to avoid small quantities of many different section sizes and grades.

Information has been provided to enable the designer to make an informed choice of sections and grades by taking account of purchasing, wastage and repeatability which in turn will have a beneficial effect on cost. The influence of connection design and fabrication complexity are addressed in sections 4 and 5 of this document. The designation Design Grade has been used to identify the basic steel strength designation, as presently used in BS 5950. However steel grade designations have been revised recently and suitable cross reference between product grade and current designation is given in Table 3.6 on page 3/13 of this document. Steel price information has been provided in a relative cost / m form in this section, based on British Steel list prices current in 1994. Obviously steel prices vary continuosly and also require adjustments for discounts, transport and tonnage variations, but the basic list prices have been presented as a simplified relative measure.

Purchasing considerations

Generally large quantities within a serial size are purchased from the steel mills, smaller quantities (2 to 5 tonnes and under - dependent on size) are purchased through stockholders.

The present British Steel pricing structure for UB and UC sections, is set by a rate per tonne, which varies from serial size to serial size, but not within each serial range. These are tabulated on following pages. Steel can be ordered in lengths varying by 0.1m up to 15 m and up to 26 m for certain sections at an additional cost. Typical average "wastage" of ordered lengths compared with actual lengths in the measure is around 3% comprising end cuts to ensure squareness and intermediate cuts of more than one member from a longer stock bar. Obviously this "wastage" reduces with longer member lengths and single members cut from single stock bars.

When buying steel through the mills, consideration should be given to :-

1) the mill rolling programmes which indicate when the rolling is scheduled to take place. The less common sections may be postponed if there is insufficient tonnage on order. It is not uncommon for the rolling of more popular sections to be fully subscribed, so that an order may have to wait for the next rolling if orders are not placed as soon as practicable.

2) it is not unusual to find that the sections that are required are scheduled on a rolling programme too far into the future to suit contract requirements, particularly with the present trend to short lead times between contract award and start of erection.

Both of these cases may necessitate supply from a stockholder.

Buying from stockholders :-

1) is usually more costly.

2) the purchaser pays for the length of bar in stock which increases the wastage or pays for cutting if the stockholder agrees that there is a usable balance.

Consideration of Design Grade

Design Grade 50A steel has a cost premium of £20 per tonne but has a strength advantage of around 30% over Design Grade 43A.

Sub-grades A and B are equally priced (sub-grade A is being phased out).
Typical 'extra over' rates :-

Relative to Design Grade 43A/B, are £25 per tonne for sub-grade C, £30 per tonne for sub-grade D and £55 per tonne for sub-grade DD.

Relative to Design Grade 50A/B, are £10 per tonne for sub-grade C and £55 per tonne for sub-grade D.

It should be borne in mind that small quantities of higher grades carry a price and availability penalty.

Beams

For beams Figure 3.1 shows the comparative cost per metre against Mcx for Universal Beams of Design Grades 43 & 50 and implies that an approximate saving in material cost of 9-15% (1994 prices) may be achieved through the use of Design Grade 50 material when strength governs the section selection.

Columns

Figure 3.2 illustrates the cost efficiency of Design Grade 50 material compared with Design Grade 43 for six common Universal Column sections. Savings of 15% in material costs (1994 prices) can be achieved. Naturally the efficiency of these sections depends upon their slenderness.

Hollow Sections

For pure element design and aesthetic considerations Structural Hollow Sections (SHS) are an excellent section. The Circular Hollow Section (CHS) has the best distribution of material for axial compression and torsional capacities. Similar capabilities can be claimed for the Rectangular Hollow Sections (RHS) of which Square Hollow Sections are a particular form. However the following points need also to be considered by the designer prior to his selection of these sections :

i) These sections are typically 60-80% more expensive than equivalent weight open sections, although the SHS prices include for carriage and material is all supplied at sub-grade D quality.

ii) The forming of connections to SHS normally involves welding which will tend to increase fabrication costs.

iii) The member selection (particularly wall thickness) should be sized to accept connection loads without requiring additional stiffening or saddles etc. Reference to CIDECT publications is strongly recommended as is the use of British Steel free software for joint connection capacity assessment.

iv) These sections are only available in standard mill lengths of 7.5, 10 or 12m or special lengths of up to 14 or 15m, cut to a tolerance of ± 150mm. These restrictions may lead to additional wastage and/or butt welding to obtain required member lengths. Thick walled tubes (20 to 50mm wall thickness) are produced by the seamless methods which further restrict the lengths available and are again substantially higher in price than the normal SHS sections.

Angles

Angles are readily obtainable from both the Mills and section re-rollers. They are inexpensive (around 295 to 375£/t 1994 prices) but may be difficult to obtain in Design Grade 50.

Fittings material

The selection of material grade and sub-grade of fittings should be made on the design requirements of strength, thickness and performance criteria and need not be related to the grade of the main member. Fabricators prefer to use a common grade of fittings material to simplify quality control and to avoid having to identify similar fittings which vary only in their grade.

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Table 3.1 - Section costs comparison - Universal beam range - Part 1

Based on British Steel March 1994 list price
Check with British Steel or Stockholders for latest figures

U. B. SERIAL SIZE	PLASTIC MODULUS Sxx cm^3	DESIGN GRADE 43A/B Cost / m	DESIGN GRADE 50A/B Cost / m
914x419x388	17,700	165	173
914x419x343	15,500	146	153
914x305x289	12,600	121	127
914x305x253	10,900	106	111
914x305x224	9,530	94	99
914x305x201	8,370	84	88
838x292x226	9,150	94	98
838x292x194	7,640	81	84
838x292x176	6,810	73	77
762x267x197	7,160	82	86
762x267x173	6,200	72	75
762x267x147	5,170	61	64
686x254x170	5,630	71	74
686x254x152	5,000	63	66
686x254x140	4,560	58	61
686x254x125	3,990	52	54
610x305x238	7,460	96	101
610x305x179	5,510	72	76
610x305x149	4,580	60	63
610x229x140	4,140	57	60
610x229x125	3,670	51	53
610x229x113	3,290	46	48
610x229x101	2,890	41	43
533x210x122	3,200	48	50
533x210x109	2,830	43	45
533x210x101	2,620	39	41
533x210x92	2,370	36	38
533x210x82	2,060	32	34
457x191x98	2,230	37	39
457x191x89	2,020	34	36
457x191x82	1,830	31	33
457x191x74	1,660	28	30
457x191x67	1,470	25	27
457x152x82	1,800	31	33
457x152x74	1,620	28	30
457x152x67	1,440	25	27
457x152x60	1,280	23	24
457x152x52	1,100	20	21

Table 3.1 - Section costs comparison - Universal beam range - Part 2

Based on British Steel March 1994 list price
Check with British Steel or Stockholders for latest figures

U. B. SERIAL SIZE	PLASTIC MODULUS Sxx cm³	DESIGN GRADE 43A/B Cost / m	DESIGN GRADE 50A/B Cost / m
406x178x74	1,510	28	30
406x178x67	1,350	26	27
406x178x60	1,200	23	24
406x178x54	1,050	21	22
406x140x46	890	18	19
406x140x39	718	15	16
356x171x67	1,210	26	27
356x171x57	1,010	22	23
356x171x51	895	20	21
356x171x45	773	17	18
356x127x39	655	15	16
356x127x33	539	13	13
305x165x54	843	21	22
305x165x46	721	17	18
305x165x40	626	15	16
305x127x48	706	18	19
305x127x42	612	16	17
305x127x37	540	14	15
305x102x33	481	13	13
305x102x28	408	11	11
305x102x25	336	10	10
254x146x43	567	16	17
254x146x37	484	14	14
254x146x31	394	11	12
254x102x28	354	10	11
254x102x25	307	9	10
254x102x22	260	8	8
203x133x30	313	10	10
203x133x25	259	8	9
203x102x23	232	7	8
178x102x19	171	6	6
152x89x16	124	6	6
127x76x13	85	5	5

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Table 3.2 - Section costs comparison - Universal column range

Based on British Steel March 1994 list price
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U. C. SERIAL SIZE	SECTION AREA cm^2	DESIGN GRADE 43A/B Cost / m	DESIGN GRADE 50A/B Cost / m
356x406x634	808	269	282
356x406x551	702	234	245
356x406x467	595	198	208
356x406x393	501	167	175
356x406x340	433	145	151
356x406x287	366	122	128
356x406x235	300	100	105
356x368x202	258	86	90
356x368x177	226	75	79
356x368x153	196	65	68
356x368x129	165	55	57
305x305x283	360	115	120
305x305x240	305	97	102
305x305x198	252	80	84
305x305x158	201	64	67
305x305x137	174	55	58
305x305x118	150	48	50
305x305x97	123	39	41
254x254x167	212	64	68
254x254x132	169	51	53
254x254x107	137	41	43
254x254x89	114	34	36
254x254x73	92.9	28	30
203x203x86	110	32	34
203x203x71	90.9	27	28
203x203x60	76	23	24
203x203x52	66.4	20	21
203x203x46	58.8	17	18
152x152x37	47.2	13	14
152x152x30	38.4	10	11
152x152x23	29.7	8	8

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Table 3.3 - Section costs comparison - Circular hollow sections (part range only)

SERIAL REF	MASS/m Kg/m	PLASTIC MODULUS Sxx - cm³	DESIGN GRADE 43 D Cost / m	DESIGN GRADE 50 D Cost / m
508.0x16.0	194	3,870	142.1	156.4
508.0x12.5	153	3,070	108.6	119.5
508.0x10.0	123	2,480	82.7	91
457.0x16.0	174	3,110	127.5	140.2
457.0x12.5	137	2,470	97.2	107
457.0x10.0	110	2,000	74	81.4
406.4x16.0	154	2,440	112.8	124.1
406.4x12.5	121	1,940	85.9	94.5
406.4x10.0	97.8	1,570	65.8	72.3
355.6x16.0	134	1,850	98.2	108
355.6x12.5	106	1,470	75.2	82.8
355.6x10.0	85.2	1,190	57.3	63
355.6x8.0	68.6	967	46.1	50.7
323.9x16.0	121	1,520	88.7	97.5
323.9x12.5	96	1,210	68.1	75
323.9x10.0	77.4	986	52	57.2
323.9x8.0	62.3	799	41.9	46.1
323.9x6.3	49.3	636	33.1	36.5
273.0x16.0	101	1,060	74	81.4
273.0x12.5	80.3	849	57	62.7
273.0x10.0	64.9	692	43.6	48
273.0x8.0	52.3	562	35.2	38.7
273.0x6.3	41.4	448	27.8	30.6
244.5x16.0	90.2	837	66.1	72.7
244.5x12.5	71.5	673	50.8	55.8
244.5x10.0	57.8	550	38.9	42.8
244.5x8.0	46.7	448	31.4	34.5
244.5x6.3	37	358	24.9	27.4
219.1x16.0	80.1	661	58.7	64.6
219.1x12.5	63.7	534	46.7	51.3
219.1x10.0	51.6	438	34.7	38.2
219.1x8.0	41.6	357	28	30.8
219.1x6.3	33.1	285	22.3	24.5
219.1x5.0	26.4	229	17.8	19.5
193.7x12.5	55.9	411	39.1	43
193.7x10.0	45.3	338	26.1	28.7
193.7x8.0	36.6	276	20.4	22.4
193.7x6.3	29.1	221	16.2	17.8
193.7x5.0	23.3	178	13	14.3
168.3x12.5	48	304	33.6	36.9
168.3x10.0	39	251	22.5	24.7
168.3x8.0	31.6	206	17.6	19.4
168.3x6.3	25.2	165	14	15.5
168.3x5.0	20.1	133	11.2	12.3
139.7x10.0	32	169	18.5	20.3
139.7x8.0	26	139	14.5	15.9
139.7x6.3	20.7	112	11.5	12.7
139.7x5.0	16.6	91	9.3	10.2
114.3x6.3	16.8	74	9	9.9
114.3x5.0	13.5	60	7.2	7.9
114.3x3.6	9.8	44	5.2	5.8
88.9x5.0	10.3	35	5.5	6
88.9x4.0	8.4	29	4.5	4.9
88.9x3.2	6.8	24	3.4	3.8
76.1x5.0	8.8	25	4.7	5.1
76.1x4.0	7.1	21	3.8	4.2
76.1x3.2	5.8	17	2.9	3.2

Based on British Steel January 1994 list price.

Check with British Steel or Stockholders for latest

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Table 3.4 - Section costs comparison - Rectangular hollow sections (part range)

Based on British Steel January 1994 list price
Check with British Steel or Stockholders for latest figures

SECTION REFERENCE	MASS/m Kg/m	PLASTIC MODULUS Sxx cm^3	DESIGN GRADE 43 D Cost / m	DESIGN GRADE 50 D Cost / m
200x100x12.5	53.4	417	42.76	47.03
200x100x10.0	43.6	346	25.75	28.32
200x100x8.0	35.4	286	20.21	22.23
200x100x6.3	28.3	231	16.16	17.77
200x100x5.0	22.7	186	12.96	14.25
160x80x12.5	41.6	254	24.34	26.77
160x80x10.0	34.2	213	20.01	22.01
160x80x8.0	27.9	177	15.78	17.35
160x80x6.3	22.3	144	12.61	13.87
160x80x5.0	18	117	10.18	11.2
150x100x12.5	43.6	263	25.51	28.06
150x100x10.0	35.7	220	20.88	22.97
150x100x8.0	29.1	183	16.46	18.1
150x100x6.3	23.3	148	13.18	14.49
150x100x5.0	18.7	121	10.57	11.63
120x80x10.0	27.9	134	16.32	17.95
120x80x8.0	22.9	113	13.01	14.31
120x80x6.3	18.4	92	9.96	10.96
120x80x5.0	14.8	75	8.01	8.81
120x60x8.0	20.4	95	11.04	12.15
120x60x6.3	16.4	78	8.88	9.77
120x60x5.0	13.3	64	7.2	7.92
120x60x3.6	9.7	48	5.11	5.62
100x60x8.0	17.8	71	9.64	10.6
100x60x6.3	14.4	58	7.8	8.57
100x60x5.0	11.7	48	6.33	6.97
100x60x3.6	8.6	36	4.51	4.97
100x60x3.0	7.2	31	3.62	3.98
100x50x8.0	16.6	63	8.99	9.88
100x50x6.3	13.4	53	7.25	7.98
100x50x5.0	10.9	43	5.9	6.49
100x50x4.0	8.9	36	4.66	5.12
100x50x3.2	7.2	29	3.6	3.96
100x50x3.0	6.8	28	3.38	3.72

Table 3.5 - Section costs comparison table - Square hollow sections (part range)

Based on British Steel January 1994 list price

Check with British Steel or Stockholders for latest figures

SERIAL REFERENCE	MASS/m Kg/m	PLASTIC MODULUS Sxx cm^3	DESIGN GRADE 43D Cost / m	DESIGN GRADE 50D Cost / m
150x150x12.5	53.4	348	42.8	47
150x150x10.0	43.6	290	25.7	28.3
150x150x8.0	35.4	240	20.2	22.2
150x150x6.3	28.3	194	16.2	17.7
150x150x5.0	22.7	157	13	14.3
140x140x12.5	49.5	299	39.6	43.6
140x140x10.0	40.4	250	23.6	26
140x140x8.0	32.9	207	18.6	20.5
140x140x6.3	26.3	168	14.9	16.4
140x140x5.0	21.1	136	11.9	13.1
120x120x12.5	41.6	212	24.3	26.8
120x120x10.0	34.2	178	20	22
120x120x8.0	27.9	149	15.8	17.4
120x120x6.3	22.3	121	12.6	13.9
120x120x5.0	18	98	10.2	11.2
100x100x10	27.9	119	16.3	18
100x100x8.0	22.9	100	13	14.3
100x100x6.3	18.4	82	10	11
100x100x5.0	14.8	67	8	8.8
100x100x4.0	12	55	6.1	6.7
90x90x8.0	20.4	79	11	12.1
90x90x6.3	16.4	65	8.9	9.8
90x90x5.0	13.3	54	7.2	7.9
90x90x3.6	9.7	40	5.1	5.6
80x80x8.0	17.8	61	9.6	10.6
80x80x6.3	14.4	51	7.8	8.6
80x80x5.0	11.7	42	6.3	7
80x80x3.6	8.6	31	4.5	5
80x80x3.0	7.2	27	3.6	4
70x70x8.0	15.3	45	8.3	9.1
70x70x6.3	12.5	38	6.8	7.4
70x70x5.0	10.1	31	5.5	6
70x70x3.6	7.5	24	3.9	4.3
70x70x3.0	6.3	20	3.1	3.5

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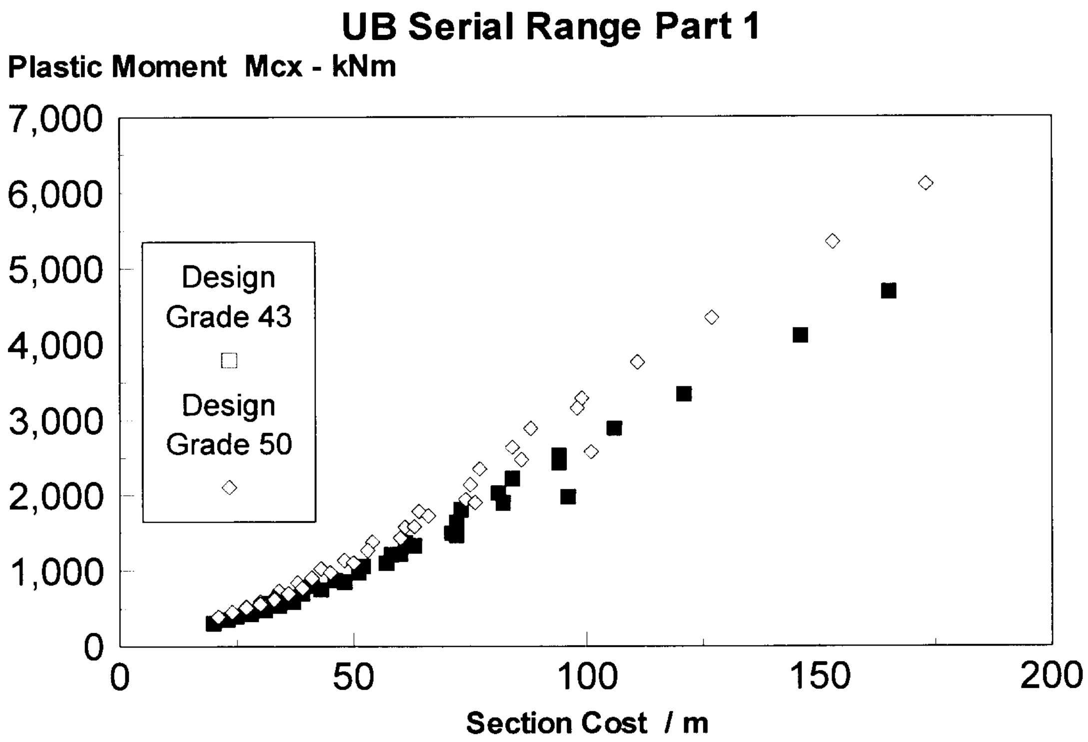

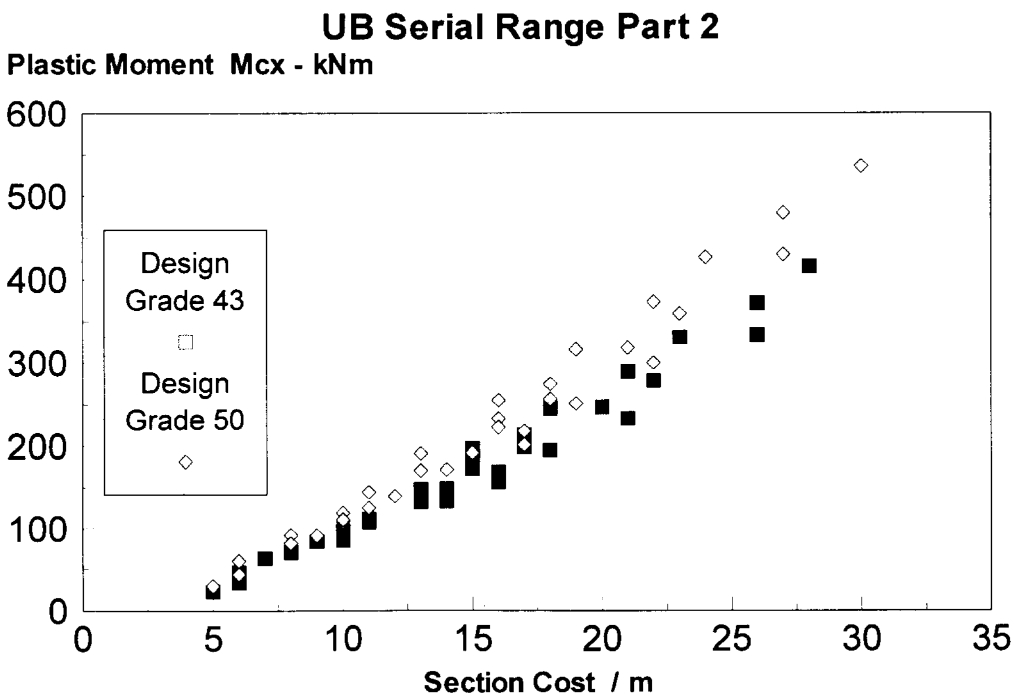

Figure 3.1 Comparison of Cost Effectiveness of Design Grade if Strength Governs - UB serial Range

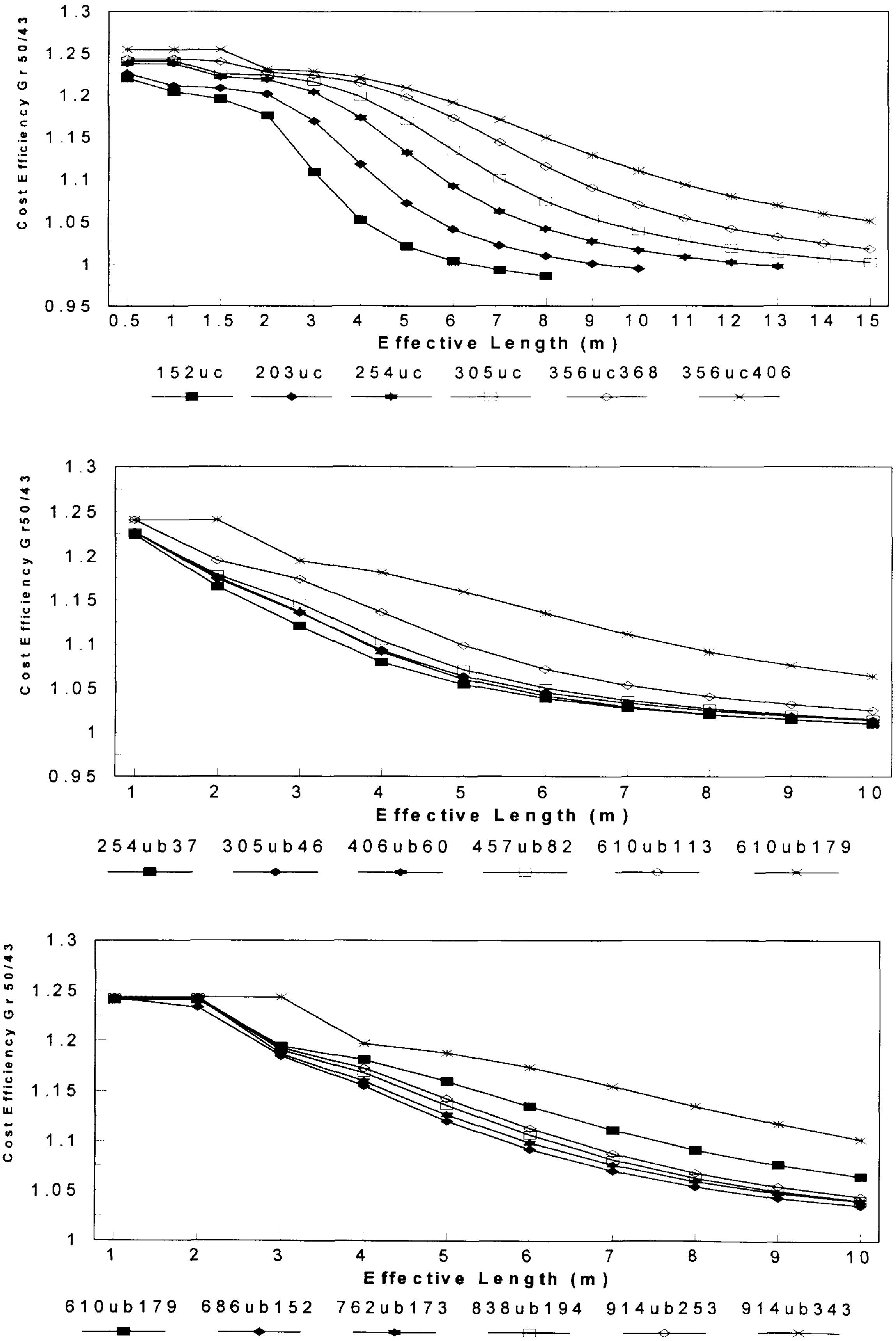

Figure 3.2 Cost efficiency of Design Gr 50 compared to Gr 43 for UB and UC Sections when maximum stress is limited by effective length.

Design For Manufacture Guidelines

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As there have been many recent revisions to the designations of steel grades and the associated product standards, this document has used the term 'Design Grade' throughout consistent with that of BS 5950. The following table addresses the cross reference of design grade to appropriate designation.

Table 3.6 Appropriate product designations corresponding to BS 5950 Design Grades

BS 5950 Design Grade	Product Form and designation							
	Sections other than hollow sections[1,2]		Plates, wide flats, strip[1,2]		Flats, round and square bars[1,2]		Hollow sections[12]	
	Old	New	Old	New	Old	New	Old	New
43A	Fe 430 A[3] or Fe 430 B	S275[3] or S275JR	Fe 430 A[3] or Fe 430 B	S275[3] or S275JR	Fe 430 A[3] or Fe 430 B	S275[3] or S275JR	[4]	[4]
43B	Fe 430 B	S275JR	Fe 430 B	S275JR	Fe 430 B	S275JR	[4]	[4]
43B(T)	Fe 430 B[5]	S275JR[5]	Fe 430 B[5]	S275JR[5]	Fe 430 B[5]	S275JR[5]	[4]	[4]
43C	Fe 430 C	S275J0	Fe 430 C	S275J0	Fe 430 C	S275J0	43C[6]	S275J0H[6]
43D	Fe 430 D	S275J2	Fe 430 D	S275J2	Fe 430 D	S275J2	43D[6]	S275J2H[6]
43DD	43DD[6]	S275J2G3	[4]	[4]	[4]		[4]	[4]
43E	[4]	[4]	[4]	[4]	43E[6]		[4]	[4]
43EE	[4]	[4]	43EE[6]	S275NL[7]	[4]		43EE[6]	S275NLH[6]
50A	Fe 510 A[3] or Fe 510 B	S355[3] or S355JR	Fe 510 A[3] or Fe 510 B	S355[3] or S355JR	Fe 510 A[3] or Fe 510 B	S355[3] or S355JR	[4]	[4]
50B	Fe 510 B	S355JR	Fe 510 B	S355JR	Fe 510 B	S355JR	[4]	[4]
50B(T)	Fe 510 B[5]	S355JR[5]	Fe 510 B[5]	S355JR[5]	Fe 510 B[5]	S355JR[5]	[4]	[4]
50C	Fe 510 C	S355J0	Fe 510 C	S355J0	Fe 510 C	S355J0	50C[6]	S355J0H[6]
50D	Fe 510 D	S355J2	Fe 510 D	S355J2	Fe 510 D	S355J2	50D[6]	S355J2H[6]
50DD	Fe 510 DD	S355J2G3	Fe 510 DD	S355K2	Fe 510 DD	S355J2G3	[4]	[4]
50E	50E[6]	S355NL[7]	[4]	[4]	50E[6]	S355NL[7]	[4]	[4]
50EE	[4]	[4]	55EE[6]	S355NL[7]	[4]	[4]	50EE[6]	S355NLH[6]
50F	[4]	[4]	55F[6]	S390J6Q[11]	[4]	[4]	[4]	[4]
55C	55C[6]	S460N[7]	55C[6]	S460N[7]	55C[6]	S460N[7]	55C[6]	S460NH[6]
55EE	[4]	[4]	55EE[6]	S460NL[7]	55EE[6]	S460NL[7]	55EE[6]	S4600NLH[6]
55F	[4]	[4]	55F[6]	S450J6Q[11]	[4]	[4]	55F[6]	[4]
WR50A	WR50A[6]	[4]	WR50A[6]	[4]	WR50A[6]	[4]	WR50A[6]	S345J0WPH[10]
WR50B	WR50B[6]	[4]	WR50B[6]	S355J0W[8,9]	WR50B[6]	[4]	WR50B[6]	S345J0WH[10]
WR50C	WR50C[6]	S355J0W[8,9]	WR50C[6]	S355J0W[8,9]	WR50C[6]	S355J0W[8,9]	WR50C[6]	S345GWH[10]

Notes :

1. Unless shown otherwise, designations in this product form are supplied to BS EN 10025
2. Products certified as complying with BS 4369:1990 having the same steel grade as the BS 5950 design grade or products certified as complying with BS EN 10025:1990 having designations to BS EN 10025:1993 are permitted alternatives.
3. Grades S275 and S355 are supplied in accordance with BS EN 10025 annex D, Non-conflicting national additions.
4. Designations in this product form are not included in BS EN 10025, BS EN 10113, BS EN 10155 or BS EN 10210 as applicable.
5. For design grades 43B(T) and 50B(T), verification of the impact quality shall be specified under option 9 of BS EN 10025 at the time of enquiry and order.
6. Designations in this product form are supplied in accordance with BS EN 10210.
7. Designations in this product form are supplied in accordance with BS EN 10113.
8. Designations in this product form are supplied in accordance with BS EN 10155.
9. Design grade has no direct equivalent, nearest equivalent is shown.
10. Designations in this product form are supplied in accordance with BS 7668.
11. Designations in this product form are supplied in accordance with BS 7613.
12. Hollow sections certified as complying with BS 2360:1990 having the same steel grade as the BS 5950 design grade are permitted alternatives.

4. CONNECTION DESIGN CONSIDERATIONS

CONNECTION DESIGN CONSIDERATIONS

Introduction

The raw material cost of a steel frame is around 35-50% of the total value of the finished product. The 50-65% of added value is dominated by labour costs which are in turn influenced by the style[1] of frame connections. Thus connections directly influence 40-60% of the ultimate value of the frame and as such are of comparable importance to basic material weight. It is therefore important that the frame designer assesses the connection viability and style at the same time that he checks the member selection against member design criteria. It may be more appropriate to choose the member on connection requirements rather than on member design requirements.

In the UK connection design and detailing has traditionally been the domain of the Fabricator and has only recently been the subject of efforts to develop standardised design approaches. This section contains basic connection capacity tables which may be applied at section selection stage to give an indication of likely fabrication complications. These design aids will not give a detailed connection design but should suffice to indicate if fabrication complications are likely to arise. The designer can then consider whether a revised section should be substituted to reduce the probable fabrication content. Whatever the final decision, the expected connections should be indicated on the design drawings. This will preclude future misconceptions on the Fabricators behalf, reduce tender prices through reduction in allowance for unknown risks and reduce possibilities of future claims and delays for unexpected complications during fabrication.

The assessment of which connection style is more cost effective, including materials and fabrication content, is the subject of Section 5 of this document. This section deals solely with design aids which assist the determination of the connection capacity and hence likely fabrication content. Whilst presented here in tabular form for manual assessment, such connection checks can be built into analysis and design checking software. Aspects of these checks are to be incorporated in software being developed under the CIMsteel project. Worked examples illustrating the use of these connection capacity tables can be found at the end of this section. These examples are continued into comparative frame costing in the next section of the Guidelines.

For further information on detailed connection design the SCI publications Design of Simple Connections Volumes 1 and 2 and Design of Moment Connections (soon to be published), all to BS 5950 requirements, are strongly recommended.

1. The term connection "style" is intended to relate to the fabrication content of the connection rather than its structural form. Refer to section 5 of this document to see the fabrication categories of "Low, Medium and High"; the connection styles Medium and High relating to the ends of beam members both have moment carrying capabilities, but they have significantly different fabrication content and hence cost.

Simple connections

Few problems should occur with simple shear connections with member end reactions up to 50% of shear capacity. For higher end reactions the use of double angle web cleats or full depth flexible end plates can extend connection capacities to 70-100% of member shear capacity. For guidance, the capacities in Table 4.1 are provided based on Design Grade 43 member capacity.

Moment connections

Moment connections using full depth or extended endplates with M20 or M24 grade 8.8 bolts, form moderately more complex beams compared with those with simple endplates. Attention should be given to column design when considering moment connections to avoid stiffening where possible.

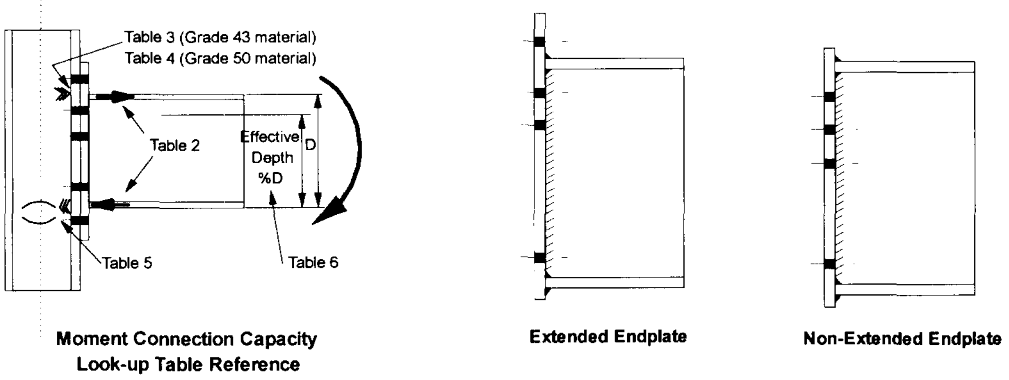

Figure 4.1 - Moment Connection Capacity Table Reference Diagram

Table 4.2 estimates maximum moment capacity for such endplate connections, expressed as a percentage of the relative UB section capacity Mcx. This table is based on the strength of the beam and the capacity of the bolts. Greater connection capacity can be achieved using haunched connections but with a significant increase in complexity. Short haunches may also be required to reduce bolt tension forces to avoid column stiffening in the tension zone. Compression zone stiffening may also be avoided by haunching the beam; this may be preferable where stiffeners would interfere with beams framing into the column web. Similarly beam haunches may be employed to avoid shear stiffening of column webs.

Tables 4.3 & 4.4 estimate the capacities of unstiffened UC flanges and webs expressed as a percentage of the maximum bolt capacity. Columns where the avoidance of stiffening may impose limits on bolt loads or may require tensile stiffening are indicated.

Table 4.5 estimates UC compression zone bearing/buckling and shear. The capacities are given in kN and are based on various combinations of beam flange and endplate thickness.

Table 4.6 estimates the minimum effective lever arm expressed as a percentage of the beam depth to be used to estimate the force in the compression flange of a given serial size UB. This table is derived from Table 4.2. Using the effective depth, a compressive flange force can quickly be estimated and compared with the column capacities in Table 4.5 to determine if any column compressive stiffening is required.

Worked examples utilising these tables follow the tabulated information in this section.

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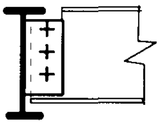
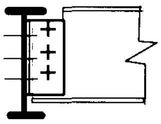
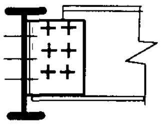
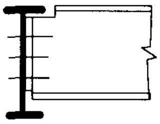
Table 4.1 Shear connection capacities expressed as a% of UB section shear capacity

For Design Grade 43 material				
U.B. Serial Range	**Fin Plate**	**Double Angle Web Cleat Single Bolt Row**	**Double Angle Web Cleat Double Bolt Row**	**Flexible End Plate**
203	25 %	25 %	35 %	70 - 100 %
254	35 %	35 %	50 %	70 - 100 %
305	30 %	30 %	45 %	70 - 100 %
356	40 %	40 %	60 %	70 - 100 %
406	50 %	50 %	75 %	70 - 100 %
457	50 %	55 %	90 %	70 - 100 %
533	45 %	60 %	90 %	70 - 100 %
610	25 - 40 %	60 %	75 %	70 - 100 %
686	N/A	65 %	80 %	70 - 100 %
762	N/A	65 %	80 %	70 - 100 %
838	N/A	60 %	75 %	70 - 100 %
914	N/A	55 %	70 %	70 - 100 %

Notes.

1. All capacities quoted assume M20 grade 8.8 bolts are used.

2. All capacities assume that the supporting ply is of sufficient strength and thickness to develop the full single shear capacity of the bolts, or double shear capacity if used for two sided connections.

3. For end plate connections to develop greater than 75% capacity this would usually require the endplate to be the full depth of the beam and welded on full inside profile.

Table 4.2 Maximum moment end plate connection capacity (un-haunched) expressed as a percentage of section moment capacity Mcx

For Design Grade 43 material

SERIAL REF	EXTENDED ENDPLATE		NON-EXTENDED ENDPLATE	
	M20 BOLTS	M24 BOLTS	M20 BOLTS	M24 BOLTS
914 UB 388	25%	30%	20%	25%
914 UB 289	35%	45%	25%	35%
914 UB 201	50%	60%	40%	45%
838 UB 226	40%	50%	35%	40%
838 UB 176	50%	60%	45%	45%
762 UB 197	40%	55%	35%	40%
762 UB 147	50%	65%	40%	45%
686 UB 170	45%	55%	35%	45%
686 UB 125	50%	70%	40%	50%
610 UB 238	30%	40%	20%	30%
610 UB 149	45%	50%	30%	35%
610 UB 140	50%	60%	35%	40%
610 UB 101	65%	70%	45%	50%
533 UB 122	50%	65%	35%	40%
533 UB 82	70%	80%	45%	50%
457 UB 98	55%	70%	35%	45%
457 UB 67	70%	85%	45%	55%
457 UB 82	65%	85%	45%	50%
457 UB 52	80%	80%	55%	65%
406 UB 74	60%	85%	40%	45%
406 UB 54	80%	90%	45%	60%
406 UB 39	80%	80%	60%	70%
356 UB 67	65%	85%	35%	45%
356 UB 45	85%	90%	50%	60%
356 UB 33	80%	80%	60%	65%
305 UB 54	65%	95%	40%	45%
305 UB 40	85%	95%	45%	55%
305 UB 37	85%	90%	50%	75%
305 UB 25	75%	75%	60%	75%
254 UB 43	80%	100%	40%	55%
254 UB 31	95%	95%	55%	70%
254 UB 22	80%	80%	65%	75%
203 UB 30	100%	100%	50%	65%
203 UB 23	100%	100%	65%	70%

Notes

1. All values are expressed as a percentage of the beam Mcx capacity.
2. All connections assume 2 vertical rows of grade 8.8 bolts.
3. Capacities are limited by bolt force, flange stresses (tensile or compressive) and web tension capacities relating only to the beam member.

Table 4.2 Maximum moment end plate connection capacity (un-haunched) expressed as a percentage of section moment capacity Mcx

For Design Grade 50 material

SERIAL REF	EXTENDED ENDPLATE		NON-EXTENDED ENDPLATE	
	M20 BOLTS	M24 BOLTS	M20 BOLTS	M24 BOLTS
914 UB 388	15%	20%	10%	15%
914 UB 289	25%	35%	20%	25%
914 UB 201	35%	50%	30%	40%
838 UB 226	30%	40%	25%	30%
838 UB 176	40%	55%	35%	45%
762 UB 197	35%	45%	25%	35%
762 UB 147	45%	60%	35%	45%
686 UB 170	35%	45%	30%	35%
686 UB 125	50%	60%	40%	45%
610 UB 238	20%	30%	15%	20%
610 UB 149	35%	40%	25%	30%
610 UB 140	40%	50%	30%	35%
610 UB 101	55%	65%	40%	45%
533 UB 122	40%	50%	30%	35%
533 UB 82	55%	70%	40%	45%
457 UB 98	45%	55%	30%	35%
457 UB 67	60%	75%	40%	45%
457 UB 82	55%	70%	35%	40%
457 UB 52	70%	80%	45%	55%
406 UB 74	50%	65%	35%	40%
406 UB 54	65%	80%	40%	50%
406 UB 39	80%	80%	55%	60%
356 UB 67	50%	70%	30%	35%
356 UB 45	80%	100%	45%	55%
356 UB 33	80%	80%	55%	65%
305 UB 54	55%	75%	30%	35%
305 UB 40	70%	95%	35%	45%
305 UB 37	80%	80%	45%	55%
305 UB 25	70%	70%	55%	60%
254 UB 43	65%	85%	30%	35%
254 UB 31	95%	100%	45%	60%
254 UB 22	75%	75%	55%	55%
203 UB 30	85%	100%	35%	45%
203 UB 23	95%	95%	45%	60%

Notes

1. All values are expressed as a percentage of the beam Mcx capacity.
2. All connections assume 2 vertical rows of grade 8.8 bolts.
3. Capacities are limited by bolt force, flange stresses (tensile or compressive) and web tension capacities relating only to the beam member.

Table 4.3 Un-stiffened column flange and web capacities expressed as a percentage of bolt force

Design Grade 43 material		M20 Grade 8.8 Bolts — Tensile Capacity of Bolt = 110 kN — Vertical Pitch assumed = 80mm			M24 Grade 8.8 Bolts — Tensile Capacity of Bolt = 159 kN — Vertical Pitch assumed = 90mm		
UC REF	c/c	% Bolt Capacity — Flange	% Bolt Capacity — Web	Comments	% Bolt Capacity — Flange	% Bolt Capacity — Web	Comments
356x406x634	130	100	100		100	100	
	140	100	100		100	100	
	150	100	100		100	100	
356x406x551	120	100	100		100	100	
	130	100	100		100	100	
	140	100	100		100	100	
356x406x467	120	100	100		100	100	
	130	100	100		100	100	
	140	100	100		100	100	
356x406x393	120	100	100		100	100	
	130	100	100		100	100	
	140	100	100		100	100	
356x406x340	110	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
356x406x287	110	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
356x406x235	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
356x368x202	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		94	100	SEE NOTE 1
356x368x177	100	100	100		100	100	
	120	100	100		91	100	SEE NOTE 1
	140	92	100	SEE NOTE 1	71	100	SEE NOTE 1
356x368x153	90	100	100		99	85	SEE NOTES 1 AND 2
	100	100	100		90	95	SEE NOTES 1 AND 2
	120	86	100	SEE NOTE 1	67	95	SEE NOTES 1 AND 2
356x368x129	90	97	100	SEE NOTE 1	71	76	SEE NOTES 1 AND 2
	100	80	100	SEE NOTE 1	62	80	SEE NOTES 1 AND 2
	120	60	100	SEE NOTE 1	47	80	SEE NOTES 1 AND 2
305x305x283	110	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
305x305x240	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
305x305x198	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
305x305x158	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		80	100	SEE NOTE 1
305x305x137	90	100	100		100	91	SEE NOTE 2
	100	100	100		100	100	
	120	96	100	SEE NOTE 1	75	100	SEE NOTE 1
305x305x118	90	100	100		81	82	SEE NOTES 1 AND 2
	100	93	100	SEE NOTE 1	73	89	SEE NOTES 1 AND 2
	120	69	100	SEE NOTE 1	54	89	SEE NOTES 1 AND 2
305x305x97	90	76	99	SEE NOTES 1 AND 2	57	74	SEE NOTES 1 AND 2
	100	64	99	SEE NOTES 1 AND 2	49	77	SEE NOTES 1 AND 2
	120	48	99	SEE NOTES 1 AND 2	37	77	SEE NOTES 1 AND 2

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Table 4.3 Un-stiffened column flange and web capacities expressed as a percentage of bolt force (Continued)

Design Grade 43 material		M20 Grade 8.8 Bolts Tensile Capacity of Bolt = 110 kN Vertical Pitch assumed = 80mm			M24 Grade 8.8 Bolts Tensile Capacity of Bolt = 159 kN Vertical Pitch assumed = 90mm		
UC REF	c/c	% Bolt Capacity		Comments	% Bolt Capacity		Comments
		Flange	Web		Flange	Web	
254x254x167	90	100	100		100	100	
	100	100	100		100	100	
	120	100	100		100	100	
254x254x132	90	100	100		100	100	
	100	100	100		100	100	
	120	100	100		97	100	SEE NOTE 1
254x254x107	90	100	100		97	97	SEE NOTES 1 AND 2
	100	100	100		82	98	SEE NOTES 1 AND 2
	120	79	100	SEE NOTE 1	62	98	SEE NOTES 1 AND 2
254x254x89	90	85	100	SEE NOTE 1	66	79	SEE NOTES 1 AND 2
	100	72	100	SEE NOTE 1	56	79	SEE NOTES 1 AND 2
	120	55	100	SEE NOTE 1	43	79	SEE NOTES 1 AND 2
254x254x73	80	70	86	SEE NOTES 1 AND 2	48	59	SEE NOTES 1 AND 2
	90	58	86	SEE NOTES 1 AND 2	45	67	SEE NOTES 1 AND 2
	100	49	86	SEE NOTES 1 AND 2	38	67	SEE NOTES 1 AND 2
203x203x86	80	100	100		97	87	SEE NOTES 1 AND 2
	90	100	100		89	98	SEE NOTES 1 AND 2
	100	97	100	SEE NOTE 1	76	98	SEE NOTES 1 AND 2
203x203x71	80	94	99	SEE NOTES 1 AND 2	69	73	SEE NOTES 1 AND 2
	90	78	99	SEE NOTES 1 AND 2	61	77	SEE NOTES 1 AND 2
	100	67	99	SEE NOTES 1 AND 2	52	77	SEE NOTES 1 AND 2
203x203x60	80	64	93	SEE NOTES 1 AND 2	48	70	SEE NOTES 1 AND 2
	90	54	93	SEE NOTES 1 AND 2	42	72	SEE NOTES 1 AND 2
	100	46	93	SEE NOTES 1 AND 2	36	72	SEE NOTES 1 AND 2
203x203x52	80	48	80	SEE NOTES 1 AND 2	37	62	SEE NOTES 1 AND 2
	90	41	80	SEE NOTES 1 AND 2	32	62	SEE NOTES 1 AND 2
	100	35	80	SEE NOTES 1 AND 2	27	62	SEE NOTES 1 AND 2
203x203x46	80	37	73	SEE NOTES 1 AND 2	29	57	SEE NOTES 1 AND 2
	90	31	73	SEE NOTES 1 AND 2	24	57	SEE NOTES 1 AND 2
	100	27	73	SEE NOTES 1 AND 2	21	57	SEE NOTES 1 AND 2
152x152x37	70	45	81	SEE NOTES 1 AND 2	32	57	SEE NOTES 1 AND 2
	80	37	81	SEE NOTES 1 AND 2	29	63	SEE NOTES 1 AND 2
	90	32	81	SEE NOTES 1 AND 2	25	63	SEE NOTES 1 AND 2
152x152x30	70	29	66	SEE NOTES 1 AND 2	21	48	SEE NOTES 1 AND 2
	80	24	66	SEE NOTES 1 AND 2	19	51	SEE NOTES 1 AND 2
	90	21	66	SEE NOTES 1 AND 2	16	51	SEE NOTES 1 AND 2
152x152x23	70	15	61	SEE NOTES 1 AND 2	11	44	SEE NOTES 1 AND 2
	80	13	61	SEE NOTES 1 AND 2	10	47	SEE NOTES 1 AND 2
	90	11	61	SEE NOTES 1 AND 2	8	47	SEE NOTES 1 AND 2

Notes.

1. Local column flange flexure capacity less than bolt tensile capacity

2. Local column web tension capacity less than bolt tension capacity

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Table 4.4 Un-stiffened column flange and web capacities expressed as a percentage of bolt force

Design Grade 50 material		M20 Grade 8.8 Bolts, Tensile Capacity of Bolt = 110 kN, Vertical Pitch assumed = 60mm			M24 Grade 8.8 Bolts, Tensile Capacity of Bolt = 159 kN, Vertical Pitch assumed = 70mm		
UC REF	c/c	% Bolt Capacity Flange	% Bolt Capacity Web	Comments	% Bolt Capacity Flange	% Bolt Capacity Web	Comments
356x406x634	130	100	100		100	100	
	140	100	100		100	100	
	150	100	100		100	100	
356x406x551	120	100	100		100	100	
	130	100	100		100	100	
	140	100	100		100	100	
356x406x467	120	100	100		100	100	
	130	100	100		100	100	
	140	100	100		100	100	
356x406x393	120	100	100		100	100	
	130	100	100		100	100	
	140	100	100		100	100	
356x406x340	110	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
356x406x287	110	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
356x406x235	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
356x368x202	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		95	100	SEE NOTE 1
356x368x177	100	100	100		100	100	
	120	100	100		92	100	SEE NOTE 1
	140	90	100	SEE NOTE 1	72	100	SEE NOTE 1
356x368x153	90	100	100		100	96	SEE NOTE 2
	100	100	100		91	96	SEE NOTES 1 AND 2
	120	84	100	SEE NOTE 1	68	96	SEE NOTES 1 AND 2
356x368x129	90	94	100	SEE NOTE 1	76	81	SEE NOTES 1 AND 2
	100	78	100	SEE NOTE 1	63	81	SEE NOTES 1 AND 2
	120	58	100	SEE NOTE 1	47	81	SEE NOTES 1 AND 2
305x305x283	110	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
305x305x240	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
305x305x198	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		100	100	
305x305x158	100	100	100		100	100	
	120	100	100		100	100	
	140	100	100		81	100	SEE NOTE 1
305x305x137	90	100	100		100	100	
	100	100	100		100	100	
	120	94	100	SEE NOTE 1	75	100	SEE NOTE 1
305x305x118	90	100	100		89	90	SEE NOTES 1 AND 2
	100	91	100	SEE NOTE 1	74	90	SEE NOTES 1 AND 2
	120	68	100	SEE NOTE 1	55	90	SEE NOTES 1 AND 2
305x305x97	90	74	96	SEE NOTES 1 AND 2	60	77	SEE NOTES 1 AND 2
	100	62	96	SEE NOTES 1 AND 2	50	77	SEE NOTES 1 AND 2
	120	46	96	SEE NOTES 1 AND 2	37	77	SEE NOTES 1 AND 2

Table 4.4 Un-stiffened column flange and web capacities expressed as a percentage of bolt force (Continued)

Design Grade 50 material		M20 Grade 8.8 Bolts			M24 Grade 8.8 Bolts		
		Tensile Capacity of Bolt = 110 kN Vertical Pitch assumed = 60mm			Tensile Capacity of Bolt = 159 kN Vertical Pitch assumed = 70mm		
UC REF	c/c	% Bolt Capacity		Comments	% Bolt Capacity		Comments
		Flange	Web		Flange	Web	
	90	100	100		100	100	
254x254x167	100	100	100		100	100	
	120	100	100		100	100	
	90	100	100		100	100	
254x254x132	100	100	100		100	100	
	120	100	100		98	100	SEE NOTE 1
	90	100	100		99	99	SEE NOTES 1 AND 2
254x254x107	100	100	100		83	99	SEE NOTES 1 AND 2
	120	78	100	SEE NOTE 1	63	99	SEE NOTES 1 AND 2
	90	83	99	SEE NOTES 1 AND 2	67	80	SEE NOTES 1 AND 2
254x254x89	100	70	99	SEE NOTES 1 AND 2	57	80	SEE NOTES 1 AND 2
	120	54	99	SEE NOTES 1 AND 2	43	80	SEE NOTES 1 AND 2
	80	68	83	SEE NOTES 1 AND 2	55	67	SEE NOTES 1 AND 2
254x254x73	90	56	83	SEE NOTES 1 AND 2	45	67	SEE NOTES 1 AND 2
	100	47	83	SEE NOTES 1 AND 2	38	67	SEE NOTES 1 AND 2
	80	100	100		100	99	SEE NOTE 2
203x203x86	90	100	100		90	99	SEE NOTES 1 AND 2
	100	95	100	SEE NOTE 1	77	99	SEE NOTES 1 AND 2
	80	91	97	SEE NOTES 1 AND 2	74	78	SEE NOTES 1 AND 2
203x203x71	90	76	97	SEE NOTES 1 AND 2	61	78	SEE NOTES 1 AND 2
	100	65	97	SEE NOTES 1 AND 2	52	78	SEE NOTES 1 AND 2
	80	62	90	SEE NOTES 1 AND 2	50	73	SEE NOTES 1 AND 2
203x203x60	90	52	90	SEE NOTES 1 AND 2	42	73	SEE NOTES 1 AND 2
	100	44	90	SEE NOTES 1 AND 2	36	73	SEE NOTES 1 AND 2
	80	47	77	SEE NOTES 1 AND 2	38	63	SEE NOTES 1 AND 2
203x203x52	90	39	77	SEE NOTES 1 AND 2	32	63	SEE NOTES 1 AND 2
	100	34	77	SEE NOTES 1 AND 2	27	63	SEE NOTES 1 AND 2
	80	36	71	SEE NOTES 1 AND 2	29	57	SEE NOTES 1 AND 2
203x203x46	90	30	71	SEE NOTES 1 AND 2	24	57	SEE NOTES 1 AND 2
	100	26	71	SEE NOTES 1 AND 2	21	57	SEE NOTES 1 AND 2
	70	44	78	SEE NOTES 1 AND 2	35	63	SEE NOTES 1 AND 2
152x152x37	80	36	78	SEE NOTES 1 AND 2	29	63	SEE NOTES 1 AND 2
	90	31	78	SEE NOTES 1 AND 2	25	63	SEE NOTES 1 AND 2
	70	28	64	SEE NOTES 1 AND 2	23	52	SEE NOTES 1 AND 2
152x152x30	80	24	64	SEE NOTES 1 AND 2	19	52	SEE NOTES 1 AND 2
	90	20	64	SEE NOTES 1 AND 2	16	52	SEE NOTES 1 AND 2
	70	15	59	SEE NOTES 1 AND 2	12	48	SEE NOTES 1 AND 2
152x152x23	80	12	59	SEE NOTES 1 AND 2	10	48	SEE NOTES 1 AND 2
	90	10	59	SEE NOTES 1 AND 2	8	48	SEE NOTES 1 AND 2

Notes.

1. Local column flange flexure capacity less than bolt tensile capacity
2. Local column web tension capacity less than bolt tension capacity

Table 4.5 UC. Column minimum bearing or buckling capacity in kN

Design Grade 43

UC COLUMN SECTION SIZE	SHEAR CAP'TY kN	MINIMUM OF BEARING OR BUCKLING CAPACITY ASSUMING CONTINUOUS COLUMN OVER POINT OF LOAD FOR BEAM FLANGE THICKNESS AND ASSUMED ENDPLATE THICKNESS									
Beam Flange Thickness		8		10		15		20		30	
Endplate Thickness		15	20	15	20	20	25	25	30	25	30
356x406x634	3,320	5,819	5,936	5,843	5,959	6,018	6,134	6,193	6,309	6,309	6,426
356x406x551	2,812	4,646	4,749	4,667	4,769	4,821	4,924	4,975	5,078	5,078	5,181
356x406x467	2,398	3,698	3,790	3,717	3,808	3,854	3,946	3,991	4,083	4,083	4,174
356x406x393	1,961	2,809	2,887	2,825	2,903	2,942	3,020	3,059	3,137	3,137	3,215
356x406x340	1,648	2,220	2,287	2,233	2,301	2,335	2,402	2,436	2,504	2,504	2,571
356x406x287	1,414	1,776	1,836	1,788	1,848	1,878	1,937	1,967	2,027	2,027	2,087
356x406x235	1,120	1,299	1,348	1,309	1,358	1,383	1,432	1,456	1,505	1,505	1,554
356x368x202	1,000	1,109	1,153	1,117	1,162	1,184	1,229	1,251	1,296	1,296	1,340
356x368x177	849	895	934	903	941	961	999	1,018	1,057	1,057	1,095
356x368x153	725	726	760	733	766	783	816	833	866	866	900
356x368x129	605	571	600	577	605	620	648	662	690	690	719
305x305x283	1,503	2,295	2,363	2,308	2,377	2,411	2,480	2,514	2,583	2,583	2,651
305x305x240	1,288	1,844	1,905	1,856	1,917	1,947	2,008	2,039	2,100	2,100	2,161
305x305x198	1,037	1,379	1,430	1,389	1,440	1,465	1,516	1,542	1,593	1,593	1,643
305x305x158	816	994	1,036	1,003	1,044	1,065	1,107	1,127	1,169	1,169	1,211
305x305x137	703	814	850	821	858	876	912	931	967	967	1,004
305x305x118	595	654	686	661	692	708	739	755	787	787	818
305x305x97	503	520	547	525	553	566	594	607	634	634	662
254x254x167	882	1,323	1,374	1,333	1,384	1,409	1,460	1,486	1,537	1,537	1,587
254x254x132	685	943	984	951	992	1,013	1,054	1,075	1,116	1,116	1,158
254x254x107	551	703	737	710	744	761	796	813	847	847	882
254x254x89	434	523	551	529	557	570	598	612	640	640	668
254x254x73	360	408	432	413	436	448	472	484	507	507	531
203x203x86	459	660	694	667	701	718	753	770	804	804	839
203x203x71	353	479	506	484	512	525	553	566	594	594	621
203x203x60	322	409	435	414	440	453	478	491	517	517	542
203x203x52	272	333	355	338	360	371	393	404	426	426	448
203x203x46	245	289	309	293	313	323	343	353	373	373	393
152x152x37	216	297	320	302	324	335	358	369	391	391	413
152x152x30	171	223	241	227	245	254	272	281	299	299	318
152x152x23	153	185	201	188	205	213	230	238	255	255	272

Table 4.5 UC. Column minimum bearing or buckling capacity in kN

Design Grade 50

UC COLUMN SECTION SIZE	SHEAR CAP'TY kN	MINIMUM OF BEARING OR BUCKLING CAPACITY ASSUMING CONTINUOUS COLUMN OVER POINT OF LOAD FOR BEAM FLANGE THICKNESS AND ASSUMED ENDPLATE THICKNESS									
Beam Flange Thickness		8		10		15		20		30	
Endplate Thickness		15	20	15	20	20	25	25	30	25	30
356x406x634	4,404	7,720	7,874	7,750	7,905	7,983	8,137	8,215	8,369	8,369	8,524
356x406x551	3,731	6,163	6,299	6,190	6,327	6,395	6,532	6,600	6,736	6,736	6,873
356x406x467	3,150	4,859	4,979	4,883	5,003	5,063	5,183	5,244	5,364	5,364	5,484
356x406x393	2,576	3,690	3,793	3,711	3,813	3,865	3,967	4,018	4,121	4,121	4,223
356x406x340	2,165	2,916	3,005	2,934	3,023	3,067	3,156	3,200	3,289	3,289	3,378
356x406x287	1,841	2,312	2,390	2,327	2,405	2,444	2,522	2,561	2,639	2,639	2,717
356x406x235	1,459	1,691	1,755	1,704	1,768	1,800	1,864	1,896	1,959	1,959	2,023
356x368x202	1,302	1,443	1,501	1,455	1,513	1,542	1,600	1,629	1,687	1,687	1,745
356x368x177	1,105	1,166	1,216	1,176	1,226	1,251	1,301	1,326	1,376	1,376	1,426
356x368x153	944	945	989	954	998	1,019	1,063	1,085	1,128	1,128	1,172
356x368x129	787	744	781	751	788	807	844	862	899	899	936
305x305x283	1,974	3,014	3,104	3,032	3,122	3,168	3,258	3,303	3,393	3,393	3,483
305x305x240	1,677	2,400	2,480	2,416	2,496	2,535	2,615	2,654	2,734	2,734	2,813
305x305x198	1,350	1,795	1,861	1,808	1,875	1,908	1,974	2,007	2,073	2,073	2,140
305x305x158	1,062	1,295	1,349	1,305	1,360	1,387	1,441	1,468	1,522	1,522	1,576
305x305x137	915	1,059	1,107	1,069	1,116	1,140	1,188	1,212	1,259	1,259	1,307
305x305x118	774	852	893	860	901	922	963	983	1,024	1,024	1,065
305x305x97	649	671	706	678	713	731	766	784	819	819	854
254x254x167	1,149	1,722	1,788	1,735	1,802	1,835	1,901	1,934	2,000	2,000	2,067
254x254x132	892	1,227	1,281	1,238	1,292	1,319	1,372	1,399	1,453	1,453	1,507
254x254x107	717	915	960	924	969	991	1,036	1,058	1,103	1,103	1,148
254x254x89	566	681	717	688	725	743	779	797	833	833	869
254x254x73	465	527	557	533	563	579	609	624	655	655	685
203x203x86	598	859	904	868	913	935	980	1,002	1,047	1,047	1,092
203x203x71	460	624	659	631	666	684	720	737	773	773	808
203x203x60	415	528	561	535	568	584	617	634	667	667	700
203x203x52	351	430	459	436	464	479	507	521	550	550	578
203x203x46	316	373	399	378	404	417	443	456	482	482	508
152x152x37	279	384	413	390	418	433	462	476	505	505	533
152x152x30	221	288	312	293	316	328	351	363	387	387	410
152x152x23	198	238	260	243	264	275	297	308	329	329	351

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Table 4.6 **Effective lever arm as % serial depth for limiting values of un-haunched moment connections**

Design Grade 43

SERIAL DEPTH	EXTENDED ENDPLATE %D	NON-EXTENDED ENDPLATE %D
914	72	70
838	72	70
762	72	70
686	72	68
610	70	65
533	72	65
457	78	65
406	85	70
356	82	65
305	87	65
254	87	65
203	85	65

Design Grade 50

SERIAL DEPTH	EXTENDED ENDPLATE %D	NON-EXTENDED ENDPLATE %D
914	74	69
838	80	80
762	72	67
686	70	63
610	68	62
533	71	61
457	75	61
406	86	65
356	81	62
305	85	61
254	85	61
203	83	60

Design For Manufacture Guidelines

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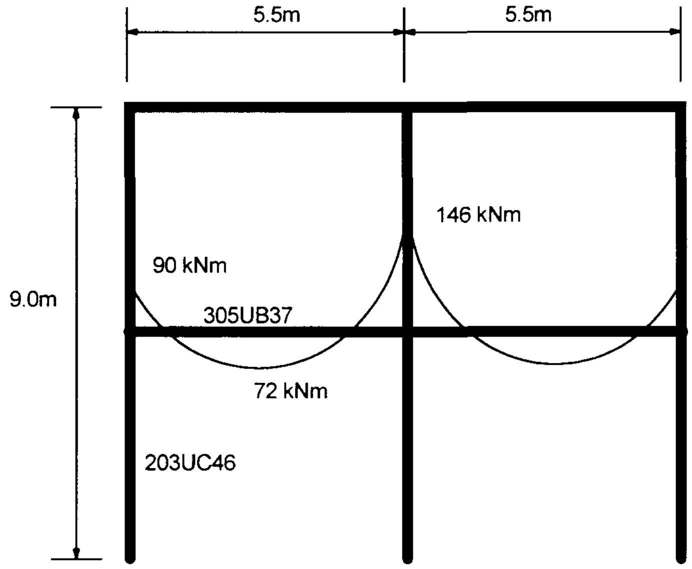
Use of connection design check tables

Example 1

Consider a two bay by two level portion of a continuous spanning frame, having a moment diagram from factored vertical loads as shown, and using 203 UC 46 columns and 305 UB 37 beams, all in Design Grade 43 material.



CHECK CONNECTION STYLES USING TABLES.

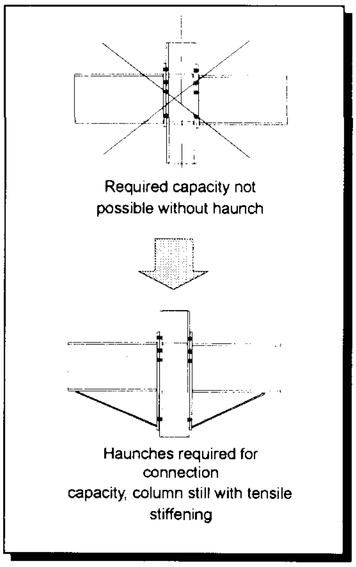
CENTRAL COLUMN/BEAM

Consider beam section. - 305 UB 37 Mcx = 149 kNm (Design Grade 43) > 146 kNm

CHECK Beam connection capacity

Table 4.2 - M24, extended = 90% Mcx = 134 kNm < 146 kNm applied.

Beam end requires a short haunch connection to sustain applied load



Taking haunch as 500 mm o/all deep by 400 long.

Assuming 4 bolts in tension - bolt force = $\frac{146}{4\text{x}0.5}$ = 73 kN i.e. 66% M20

Compression force = $\frac{146}{0.5}$ = 292 kN

CHECK Column tension zone capacity.

Table 4.3 - 203 UC 46 Flange capacity = 37% M20 - Stiffening required.

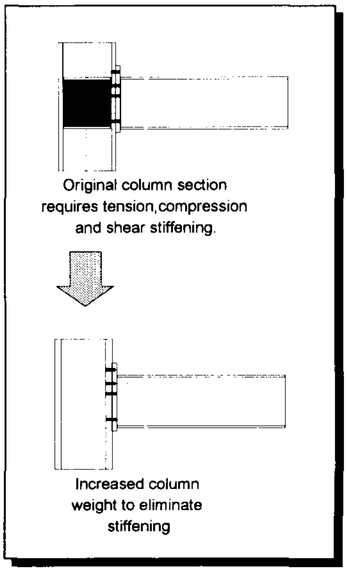
Web capacity = 74% M20 - OK.

Column is likely to require flange stiffening, even if more than 4 bolts were supplied.

CHECK Column compression zone capacity.

Table 4.5 - 203 UC 46 ; 10 mm UB flange ; 15 mm Endplate.
Minimum of buckling/bearing capacity = 293 kN > 292 kN - OK.

No compression stiffening required.



OUTER COLUMN/BEAM

Applied Moment = 90 kNm

CHECK Beam connection capacity.

Table 4.2 - M20, extended = 85% Mcx = 127 kNm >> 90 kNm - OK.

No haunch required for beam capacity.

Estimate compressive force.
Table 4.6 - At maximum connection moment effective lever arm = 87%D

$$\text{Compressive force} = \frac{90}{0.87\text{x}0.305} = 340 \text{ kN.}$$

However as Applied moment << connection capacity, effective lever arm will tend to increase to say 95%D

$$\text{Reduced Compressive force} = \frac{90}{0.95\text{x}0.305} = 311 \text{ kN.}$$

CHECK Column tension zone capacity.

Table 4.3 - 203 UC 46	Flange capacity =	37% M20 - Stiffening required.
	Web capacity =	74% M20 - Stiffening required.

Complete flange flexure and web tension stiffening required.

CHECK Column compression zone capacity.

Table 4.5 - 203 UC 46 ; 10 mm UB flange ; 20 mm Endplate.
Minimum of buckling/bearing capacity = 313 kN > 311 kN - OK.- If lower lever arm is correct - hence may require compressive stiffening.
BUT also shear capacity = 245 kN << 311 or 340 kN - Shear stiffening required.

Column requires shear stiffening, and may require compressive stiffening.

Consider an alternative column selection for outer columns to avoid such stiffening requirements.

Table 4.5 - 203 UC 71 -	Shear capacity =	353 kN
	Minimum Buckling/bearing capacity =	484 kN
Table 4.3 -	Flange capacity =	94% M20 bolt.
	Web capacity =	99% M20 bolt.

With 203 UC 71 no stiffening required to column & no haunch to beam at this connection.

Discuss me ...

Design For Manufacture Guidelines

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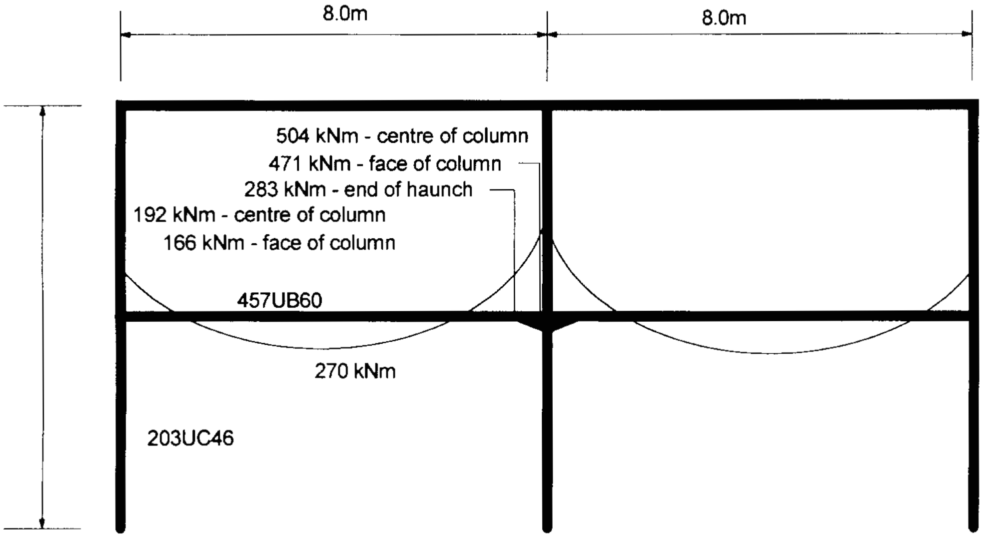
Example 2

Consider a similar portion of frame as in example 1, but with each bay spanning 8m and the frame having 456 UB 60 beam members and 203 UC 46 columns. The beams were selected after assuming that the central connection would have a short haunch style. Factored load bending moments are shown below :



CHECK CONNECTION STYLES USING TABLES.

CENTRAL COLUMN/BEAM

Consider beam section. - 457 UB 60 Mcx = 352 kNm > 283 kNm end of haunch

Moment at face of column = 471 kNm

Assuming haunch 750 mm deep overall, 600 mm long

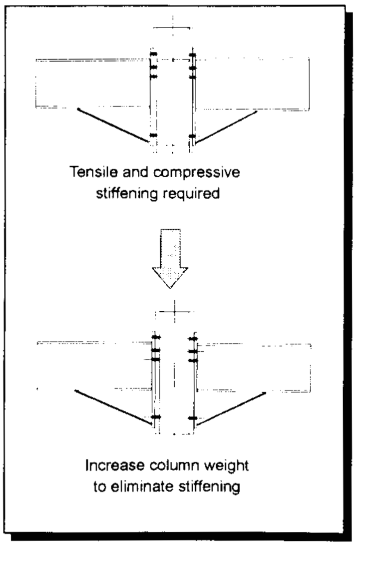
Assuming 4 bolts in tension - bolt force = $\frac{471}{4\text{x}0.75}$ = 157 kN i.e. 99% M24

Compression force = $\frac{471}{0.75}$ = 628 kN

CHECK Column tension zone capacity.

Table 4.3 - 203 UC 46 — Flange capacity = 29% M24 - Stiffening required.
Web capacity = 57% M24 - Stiffening required.

Column is likely to require flange & web stiffening, even if more than 4 bolts were supplied.



CHECK Column compression zone capacity.

Table 4.5 - 203 UC 46 ; 10 mm UB flange ; 20 mm Endplate.
Minimum of buckling/bearing capacity = 313 kN << 628 kN - Stiffening required

Full compression stiffening required.

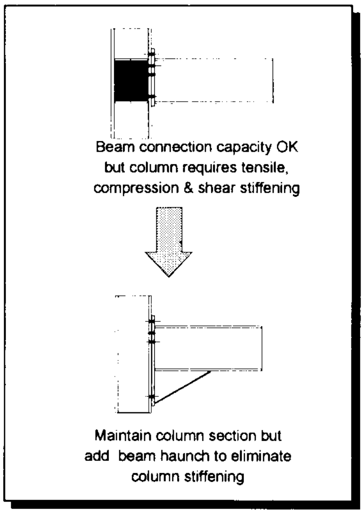
Alternative column section to avoid this stiffening -

Table 4.5 - 203 UC 86 - Shear capacity = 459 kN
Minimum Buckling/bearing capacity = 701 kN
Table 4.3 - Flange capacity = 89% M24 bolt. OK If 6 bolts used in tension.
Web capacity = 98% M24 bolt. OK If 6 bolts used in tension.

With 203 UC 86 no stiffening required to column.

OUTER COLUMN/BEAM

Applied moment at face of column = 166 kNm



CHECK Beam connection capacity.

Table 4.2 - M20, extended = 70% Mcx = 246 kNm
>> 166 kNm - OK.

No haunch required for beam capacity.

Estimate compressive force.

As applied moment << connection capacity, effective lever arm will tend to be the actual depth D

$$\text{Compressive force} = \frac{166}{0.457} = 363 \text{ kN.}$$

CHECK Column tension zone capacity.
Table 4.3 - 203 UC 46 Flange capacity = 37% M20 - Stiffening required.
Web capacity = 74% M20 - Stiffening required.

Complete flange flexure and web tension stiffening required.

CHECK Column compression zone capacity.

Table 4.5 - 203 UC 46 ; 10 mm UB flange ; 20 mm Endplate.
Minimum of buckling/bearing capacity = 313 kN < 363 kN - Stiffening required.

BUT also shear capacity = 245 kN << 363- Shear stiffening required.

Column requires shear stiffening and compression stiffening.

Alternative column selection to avoid stiffening.

Table 4.5 254 UC 132 or 305 UC 118 would be required to avoid shear stiffening - this increase in column weight and plan size - would probably not be economic.

Use a haunched beam connection to avoid shear stiffening in the column.

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Keeping the 203 UC 46 would require a minimum lever arm of $\frac{166}{246}$ = 0.67m.

However use 750 deep haunch as on centre column.

If 4 bolt connection - tension/bolt = $\frac{166}{4 \times 0.75}$ = 55 kN. i.e.. $\frac{55}{110}$ = 50% M20

CHECK Column tension zone capacity.

Table 4.3 - 203 UC 46	Flange capacity =	37% M20 - Stiffening required.
	Web capacity =	74% M20 - OK.

If 6 bolt connection -max. tension/bolt = $\dfrac{166}{4*0.75+\frac{2*0.63*0.63}{0.75}}$ = 40.9 kN

i.e.. 37 % M20 - OK.

Compression force = 4*41 + 2*41*0.63/0.75 = 233 kN. < 245 kN shear - OK.

Use of 750 haunch on beam at outer column connection eliminates stiffening requirements.

5. FABRICATION CLASSIFICATION AND COSTING

FABRICATION CLASSIFICATION AND COSTING

Introduction

The previous section showed that connection design requirements should be investigated at time of frame design to indicate the likely style of connections required. It was also shown that the style of connections can be modified by revisions to the member thickness instead of providing additional stiffening. This tends to increase the overall material weight and hence material cost, but simplifies the fabrication content and reduces manpower costs in drafting, fabrication and erection. The designer thus has a choice to make as to which is most effective. This choice may involve considerations beyond the scope of the steel frame, but this section of the Guidelines addresses the relative order of costs of the fabricated elements including material and fabrication labour costs. The actual costs of erection of the frame are not included, as these will depend greatly on the overall nature of the project, site location, craneage availability, laydown areas available etc. However, as a general statement, a frame which has easy access to the connections, un-hindered by additional stiffening which complicates the location of members framing to webs of columns, would generally be more easily erected.

Fabrication Classification

This section shows a classification of fabricated members related to the connections associated with those members. Two main areas of fabrication have been addressed ;

i) Beam and column frames (excluding portal frames).
ii) Lattice structures.

Guidance on costs has been limited to the beam and column category. Costing of lattice frames is more difficult to formulate as it depends greatly on the actual geometry of the frame, the cost of associated jigging and numbers of repeated frames per jig, as well as the nature of the sections used and the complexity of the connections within the frame. However the principle of adoption of simple connection details still applies, and anticipated connection complexity should still be investigated and indicated on tender and construction drawings.

Three levels of complexity have been identified; low, medium and high. The cost information given only addresses the end connections of the beams and no allowance has been made for intermediate connections.

As in the previous section, cost information in this section is presented in tabular form to assist manual calculations and is illustrated by worked examples. However such information may also be incorporated into frame analysis and design software, allowing a more rapid cost benefit assessment of frame choices to be examined. These aspects are being considered for incorporation within software being developed by other collaborators in the CIMsteel project.

BEAM & COLUMN CLASSIFICATION

"Low" complexity

This category is typified by braced frames, simply supported connections, pinned base columns with shear splices. Typical structure may be 1-5 storeys, with beams say 5-9m of span. Floors may be composite or non-composite design.

Assumptions

Connections consist of angle cleats, end plates or fin plates.
All main units pass through CNC saw and drill lines.
All secondary beams will be fittings free.
All welds 6/8 mm fillets.
Fittings made by shearing and punching for 8.8 bolts.

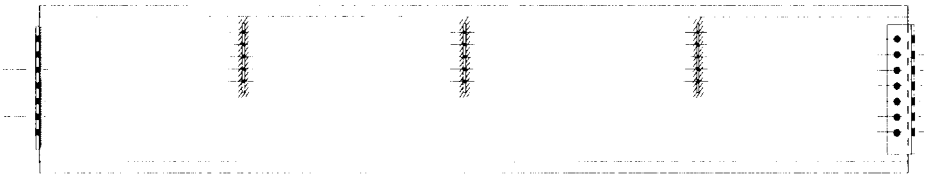
"Medium" complexity

Similar geometry buildings to the "low" category but using moderate capacity moment connections.

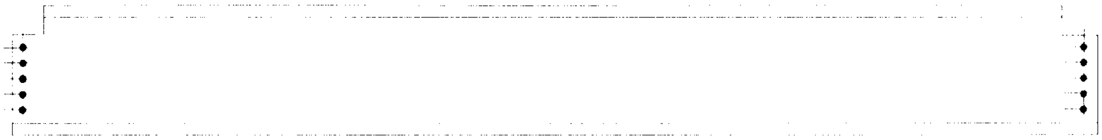
The resulting moments are generally catered for by full depth or extended endplates without stiffening or haunches. The associated columns require little or no stiffening, which causes no problems for beams framing into the column webs. Endplates are typically 20-25 mm thick, flame/plasma cut and drilled for M20 or M24 grade 8.8 bolts. Welds are generally 10/12 fillets requiring multiple passes. Secondary connections are as for "low" category although of greater capacity.

Assumptions

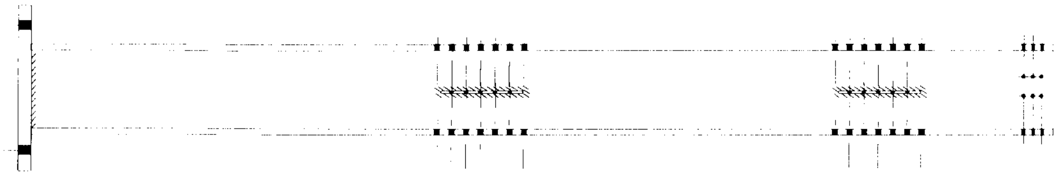
Main units still processed by CNC preparation equipment.
Fittings to primary beams require flame/plasma cutting and drilling.
Welding of endplates have greater consumable requirements and take longer.
Columns not greatly affected ; some stiffeners assumed but allow simple web connections and ease of erection.



Primary Beams - End cleats or part-depth endplates
Intermediate connections use fin plates

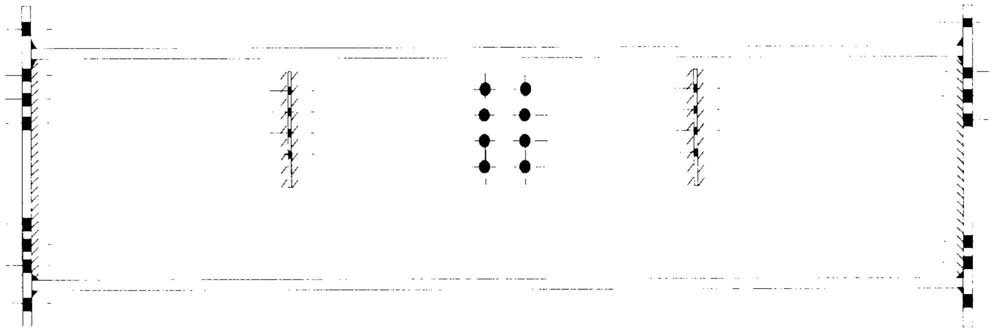


Secondary Beams - Notched and drilled only
No fittings required for a fin plate connection to Primary

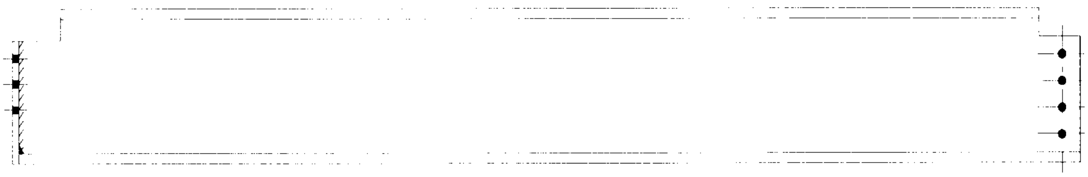


Columns - Simple baseplate and splice details
Flanges drilled for Primary beams, fin plates for Secondary to web

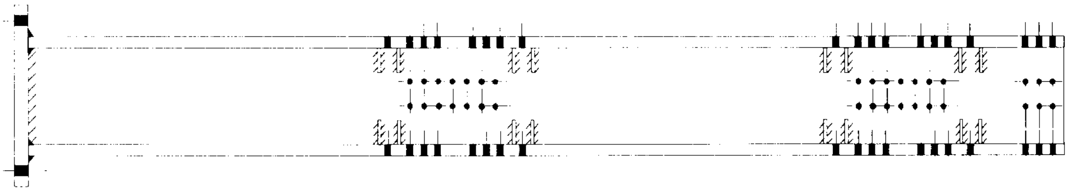
Figure 5.1
Fabrication Classification - Beam & Column
"Low" Fabrication Complexity



Primary beams - Moment endplates - No haunches
If non-extended, butt weld may be required to top flange
(butt welds not included in cost data)
Intermediate connections are fin plates or drilled.



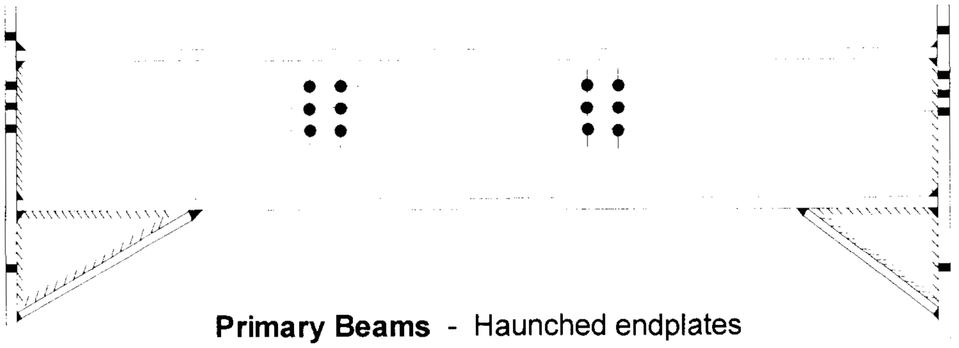
Secondary beams - notched and drilled for fin plates,
if loading requires use endplates or double row angle cleats



Columns - "Nominal" moment base and splice details.
Primary beam connections require only tensile stiffening

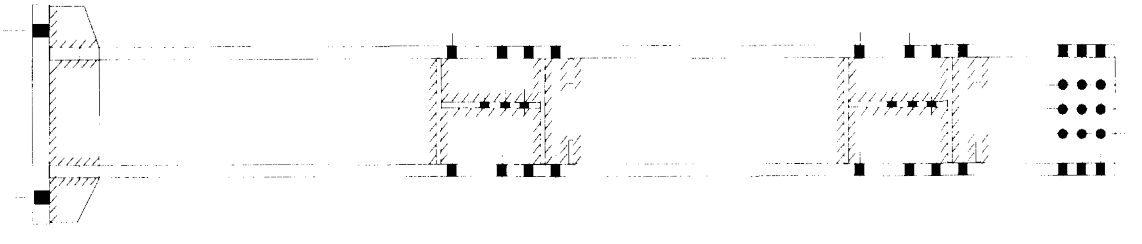
Figure 5.2
Fabrication Classification - Beam & Column
"Medium" Fabrication Complexity

Discuss me ...



Primary Beams - Haunched endplates
Intermediate connections drilled
if fin plates are not adequate

Secondary Beams - Notched and endplates or
double row bolted angle cleats if fin plates not adequate



Columns - Moment baseplate and splice
Drilled flanges for Primary beams but with tensile &
compressive stiffeners. Fin plates between stiffeners
for web connections - use T-stubs if endplate
connection required

Figure 5.3
Fabrication Classification - Beam & Column
"High" Fabrication Complexity

"High" complexity

Similar geometry buildings to the "low" category but using high capacity moment connections. Moment capacity requirements placed on connections typically exceeds 0.6*Mcx of the associated beam. Substantial moment resistance is required of the column bases.

Moment connections are of such a magnitude as to require the use of haunched ends to primary beams. Columns require tension and compression stiffening. Such stiffening requires web connections to be detailed to avoid erection problems. Secondary beams are sufficiently heavily loaded to require double row angle cleats or flexible endplate connections. Column baseplates are stiffened by additional vertical plates ; splices have some moment carrying requirement as well as axial and shear.

Use of haunched beams implies 25-30 mm thick endplates probably with M24 grade 8.8 bolts. "Short" haunches are usually formed from plate material of equivalent thickness to those of the parent beam flange and web ; the flange plates are normally rectangular and are typically greater than 15 mm thick hence requiring flame cutting and possibly edge preparation. Welds to the UB section would typically be 10-12 mm fillets each side of the top flange. Web and other welds would typically be 8-12 mm fillets each side.

Assumptions

Primary beams with haunches require more labour and welding.
The haunch materials are often flame cut.
Part penetration butt welds are often required to the haunch flange plate ends.

Secondary members if required to use double bolt row double angle web cleats need additional drilling times, together with the fittings fabrication and shop bolting. If endplates are used for the secondary members this will place additional demands on the welding resources.

Columns have substantial welding requirements at base plates and fitted stiffeners at primary beam connections.
Column splices have substantial flange and web plates.

TRUSSES & LATTICE GIRDERS

"Low" complexity

Typical mono-planar roof truss ; two booms plus internal bracing. All internal joints assumed "pinned" ; booms may be assumed as continuous.

L1 Bolted Internals

Booms from UB, UC, RSJ, ST, RSC, RSA or RHS with gusset plates at intersections bolted or welded as appropriate. Internals from RSA, RSC, RSJ, UC or UB using bolted connections to gusset plates. RSJ, UB and UC internals require flange stripping to allow connections to be formed.

L2 Welded Internals

Booms from UB, UC, RSJ, ST or RHS with internals from SHS, RSA, RSC, UB or UC sections. Boom faces must be wider than internals to allow fillet welds all round.

"Medium" Complexity

Typical mono-planar roof truss where magnitude of axial forces or moments is such as to require the introduction of local stiffening within or around the boom members at intersection points.

M1 Bolted Internals

Section selection as for "Low" above.

M2 Welded Internals

Section selection as for "Low" above.

"High" complexity

H1 All trusses requiring CHS throughout, all welded connections

Such trusses often require part or full profile preparation for internals with associated welding preparations. If joint capacity has not been satisfied by selection of adequate member section or grade, some connections may require sections of thickened walls by "canning" or use of "saddles" to prevent over stress in the CHS walls. Alternatively use of through fitted and welded gussets could be considered. The conceptual designer must consider these local effects during his preliminary sizing of the boom members (refer to CIDECT publications for more information and British Steel SHS joint design software and literature). If the truss is multi-planar the provisions of H2 will also apply.

H2 All welded, 3 or 4 boom lattice truss

All welded condition generally leads to higher costs of jigging in three dimensions as well as general handling problems and possible complications of multi-positional welding. For larger fabrications both shop handling and general transportation must be considered as well as corrosion protection provisions.

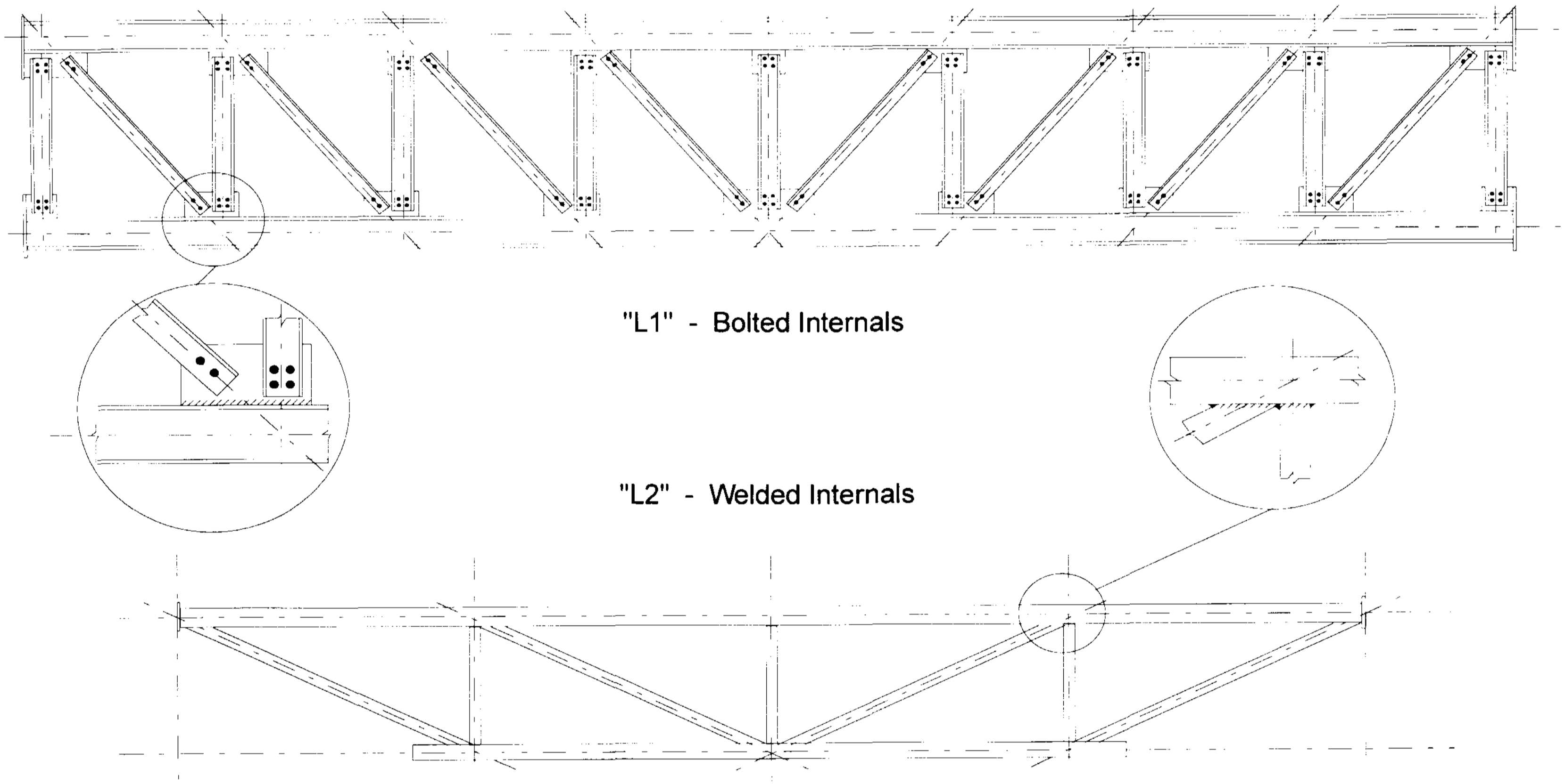


Figure 5.4

Fabrication Classification - Trusses and Lattice Frames

"Low" Fabrication Complexity

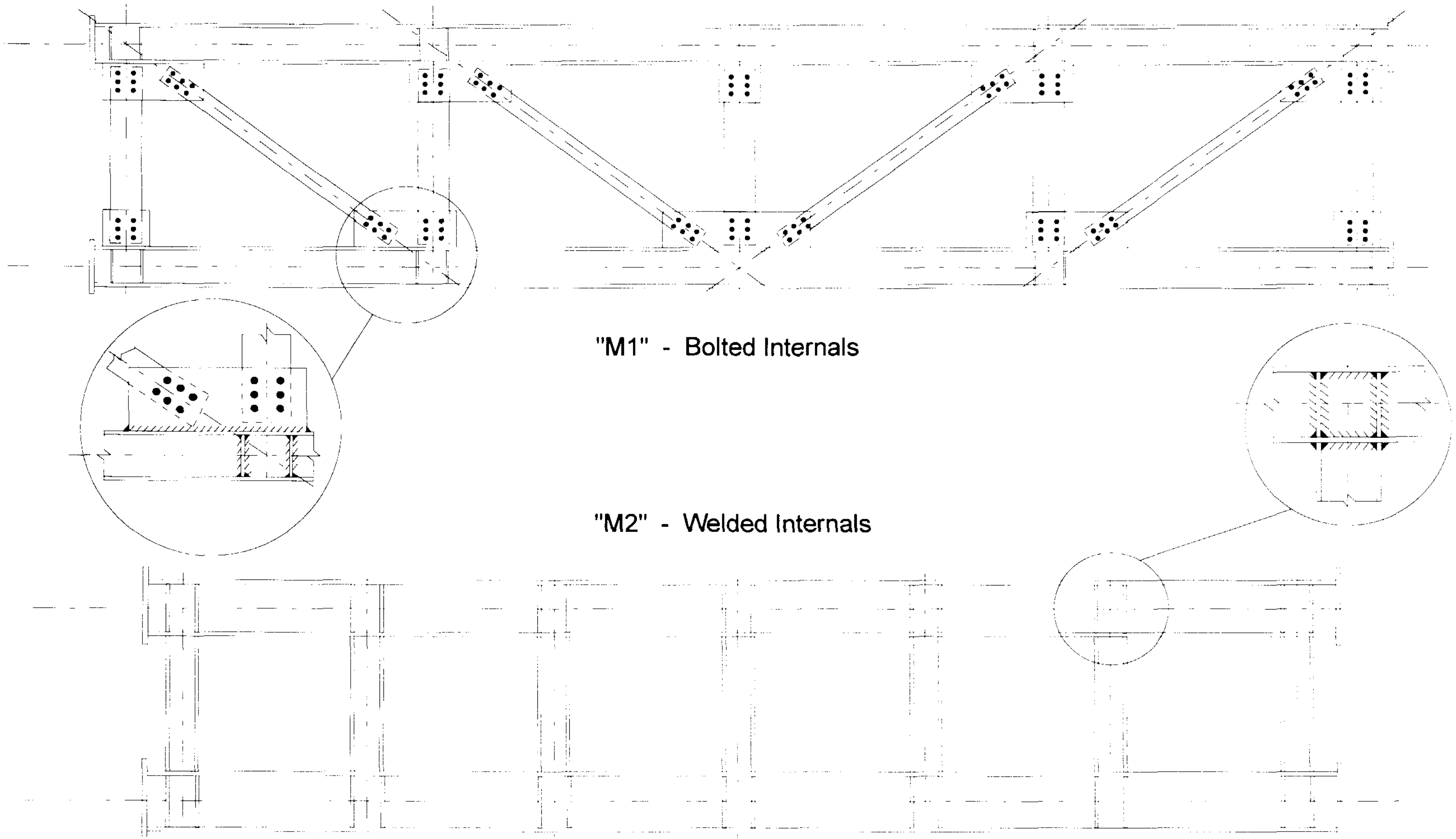
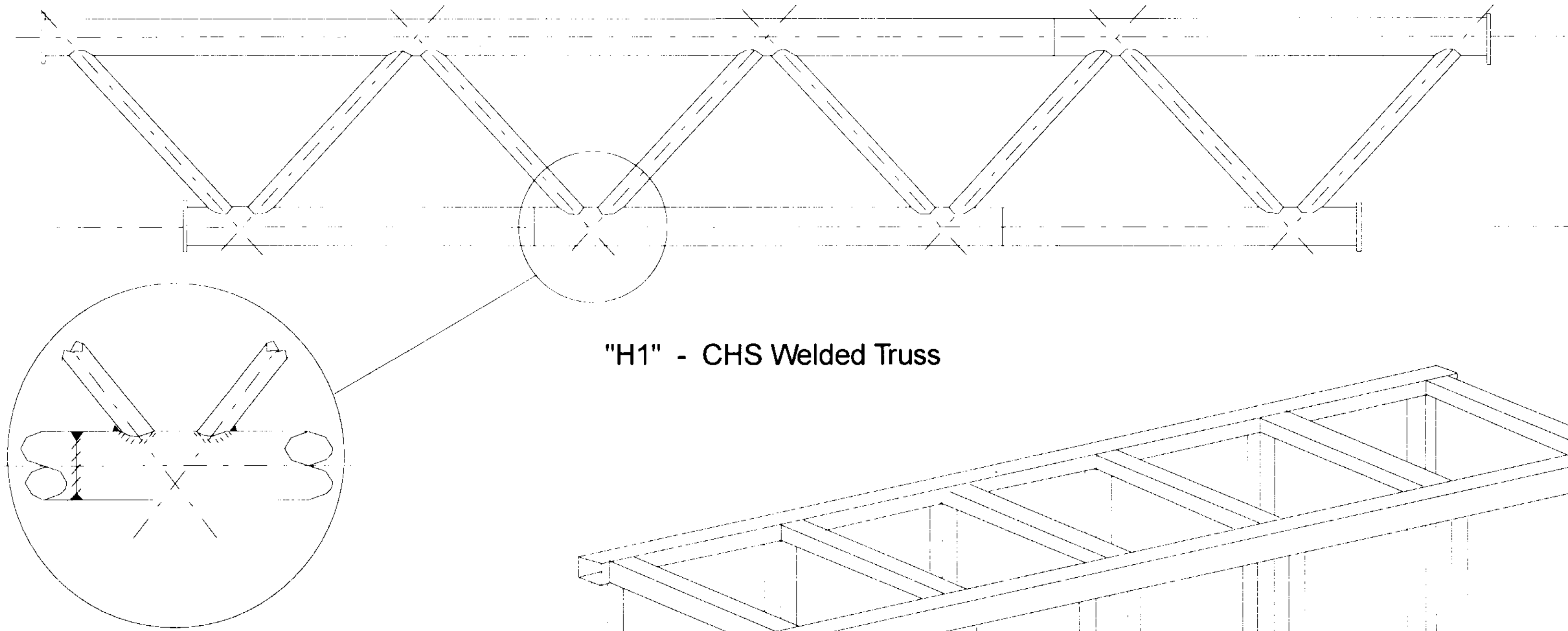
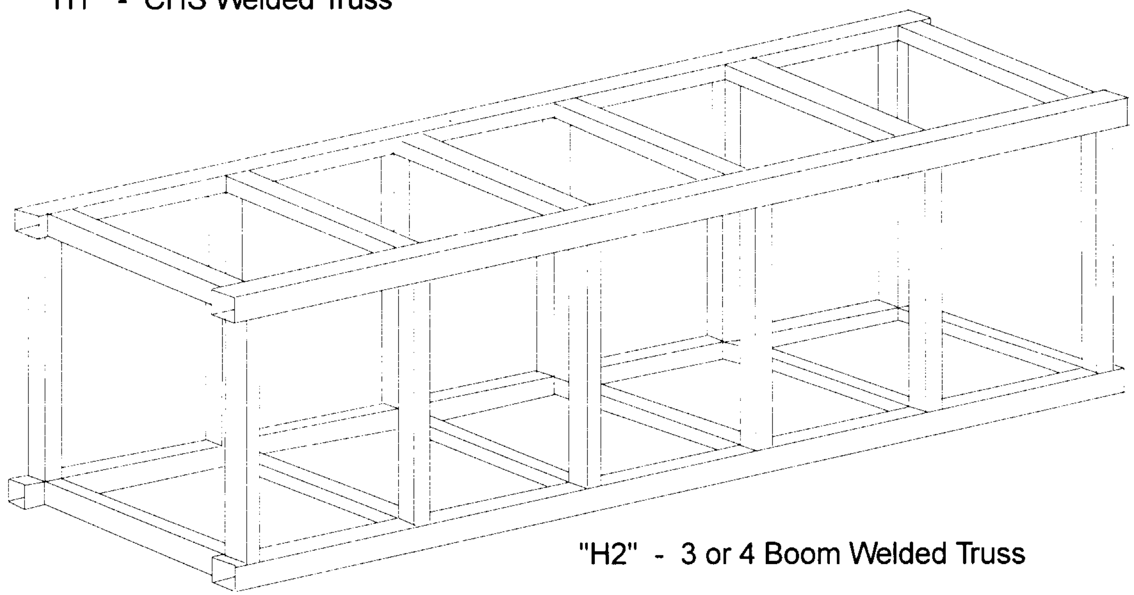


Figure 5.5
Fabrication Classification - Trusses and Lattice Frames
"Medium" Fabrication Complexity



"H1" - CHS Welded Truss



"H2" - 3 or 4 Boom Welded Truss

Figure 5.6
Fabrication Classification - Trusses and Lattice Frames
"High" Fabrication Complexity

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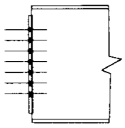
Comparative fabrication costing

For purposes of comparative costs of the fabrication classifications, the following Tables 5.1 & 5.2 have been prepared. The tables have been based on the assumptions of fabrication details as outlined below. Variations such as using butt welds to achieve a completely flush top flange detail have not been allowed for and would tend to increase the fabrication costs significantly. Worked examples of the tables usage are given at the end of this section.

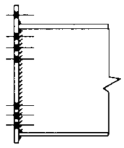
PRIMARY BEAM ENDS - COSTINGS - QUALIFYING ASSUMPTIONS

"LOW"



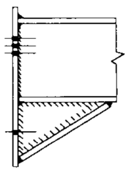
PLATES UPTO 10mmTHK,200 WIDE
TO WEB DEPTH ONLY
WELDING 6FW E/S WEB DEPTH ONLY
HOLING SAY O.6D @ 70mm VERT C/C

"MEDIUM"



ENDPLATE 25 THK,B+30 WIDE AND D+100mm DEEP. HOLING OVER SAY 0.75D @ 90mm VERT C/C. WELDING ASSUME FW E/S =0.7*THK OF WEB OR FLANGE RESPECTIVELY, BUT MIN 6mm - MAX 12mm LEG SIZE

"HIGH"



HAUNCH GEOMETRY 0.5D VERTICAL & 1.0D HORIZONTAL. ENDPLATE 25 THICK, HAUNCH PLATES EQUIVALENT TO UB FLANGE AND WEB THICKNESSES. ENDPLATE HOLED SAY 0.5D @ 90mm VERT C/C. WELDING FW E/S = 0.7*THK OF ASSOCIATED PLATE MATERIAL, 6mm MIN - 12mm MAX LEG SIZE. WELDS AT END OF HAUNCH PLATES LEG TAKEN AS FLANGE PLT THICKNESS.

COLUMN CONNECTIONS - COSTINGS - QUALIFYING ASSUMPTIONS

"LOW"

SPLICE - ASSUME 2 ROWS, 2 HOLES PER ROW
FIN PLATES 10 THK, 2D LONG, 100 WIDE, HOLED AT 70 C/C VERT. WELDED 6FW E/S

"MEDIUM"

SPLICE - 3 ROWS 2 HOLES PER ROW TO FLGS & WEB

TENSILE STIFFENERS - 10mm THK, B/2 TRIANGULAR

WELDING - 6FW E/S

"HIGH"

SPLICE - 3 ROWS 2 HOLES PER ROW TO FLGS & WEB

TENSILE STIFFENERS AS PER "MEDIUM" BUT LOCATED AS SHOWN ON "HIGH"COMPLEXITY DIAGRAM
COMPRESSIVE STIFFENERS - THICKNESS AND WELDING AS TABULATED BELOW, ASSUMED FITTED ENDS HENCE WELDING TO ENDS ASSUMED ONLY 6FW E/S

STIFFENERS AND WELDING

FLANGE THK mm	STIFF THK mm	STIFF WELD LEG mm
0	10	6
6	10	6
20	12	8
25	15	10
35	25	12
50	35	15
100	50	15

Table 5.1 Comparative costs of beam fabrication as varied by connection complexity

Total cost of fabricated member = Length (m) * Cost /m + 'Main' + (Either 'L' or 'M' or 'H') All figures 1994 based.
From Table 3.1

BEAM REF	U.B. SERIAL SIZE	FABRICATION COSTS (FOR TWO ENDS SIMILAR) PRIMARY BEAMS MAIN	"L"	"M"	"H"	SECONDARY BEAMS MAIN	"L"	"M"	"H"
1	914x419x388	70	92	239	748	102	9	92	92
2	914x419x343	64	92	239	704	97	9	92	92
3	914x305x289	59	92	217	622	91	9	92	92
4	914x305x253	54	93	217	596	86	8	93	93
5	914x305x224	50	93	217	573	83	8	93	93
6	914x305x201	47	93	197	483	80	7	93	93
7	838x292x226	49	89	202	545	80	8	89	89
8	838x292x194	45	89	183	454	76	7	89	89
9	838x292x176	43	89	183	441	74	7	89	89
10	762x267x197	45	85	169	429	75	7	85	85
11	762x267x173	42	85	169	413	72	6	85	85
12	762x267x147	39	85	169	397	69	6	85	85
13	686x254x170	41	80	156	387	69	6	80	80
14	686x254x152	39	81	156	376	67	6	81	81
15	686x254x140	37	81	143	325	66	5	81	81
16	686x254x125	36	81	143	316	64	5	81	81
17	610x305x238	49	76	167	473	75	7	76	76
18	610x305x179	41	76	154	384	68	6	76	76
19	610x305x149	37	77	143	327	64	5	77	77
20	610x229x140	37	77	142	342	64	6	77	77
21	610x229x125	35	77	131	295	62	5	77	77
22	610x229x113	33	77	131	288	60	5	77	77
23	610x229x101	32	77	126	271	59	5	77	77
24	533x210x122	34	72	128	304	59	5	72	72
25	533x210x109	33	72	119	263	58	5	72	72
26	533x210x101	32	73	119	259	57	5	73	73
27	533x210x92	30	73	114	246	56	4	73	73
28	533x210x82	29	73	114	241	54	4	73	73
29	457x191x98	31	68	107	237	54	4	68	68
30	457x191x89	30	68	107	233	53	4	68	68
31	457x191x82	29	68	107	229	52	4	68	68
32	457x191x74	28	69	97	195	51	4	69	69
33	457x191x67	27	69	97	192	51	4	69	69
34	457x152x82	29	68	102	221	53	4	68	68
35	457x152x74	28	68	102	218	52	4	68	68
36	457x152x67	27	68	92	185	51	4	68	68
37	457x152x60	26	69	92	182	50	4	69	69
38	457x152x52	25	69	89	174	49	4	69	69

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Table 5.1 Comparative costs of beam fabrication as varied by connection complexity (Continued)

Total cost of fabricated member = Length (m) * Cost /m + 'Main' + (Either 'L' or 'M' or 'H'). All figures 1994 based

From Table 3.1

BEAM REF	U.B. SERIAL SIZE	FABRICATION COSTS (FOR TWO ENDS SIMILAR)							
		PRIMARY BEAMS				SECONDARY BEAMS			
		MAIN	"L"	"M"	"H"	MAIN	"L"	"M"	"H"
39	406x178x74	27	66	100	212	50	4	66	66
40	406x178x67	26	66	91	181	49	4	66	66
41	406x178x60	25	66	91	179	48	4	66	66
42	406x178x54	25	66	87	170	48	4	66	66
43	406x140x46	24	66	84	162	47	4	66	66
44	406x140x39	23	66	82	155	46	3	66	66
45	356x171x67	26	63	84	171	48	4	63	63
46	356x171x57	25	63	84	167	47	4	63	63
47	356x171x51	24	63	81	158	46	4	63	63
48	356x171x45	23	64	81	156	45	4	64	64
49	356x127x39	23	63	77	149	45	4	63	63
50	356x127x33	22	64	76	143	44	3	64	64
51	305x165x54	24	60	79	156	45	3	60	60
52	305x165x46	23	60	76	148	44	3	60	60
53	305x165x40	23	61	76	146	43	3	61	61
54	305x127x48	24	60	75	148	45	4	60	60
55	305x127x42	23	60	73	141	44	3	60	60
56	305x127x37	23	61	73	140	43	3	61	61
57	305x102x33	22	61	71	135	43	3	61	61
58	305x102x28	21	61	70	131	42	3	61	61
59	305x102x25	21	61	70	130	42	3	61	61
60	254x146x43	23	58	73	141	43	3	58	58
61	254x146x37	22	58	70	134	42	3	58	58
62	254x146x31	22	58	68	128	41	3	58	58
63	254x102x28	21	58	67	126	41	3	58	58
64	254x102x25	21	58	66	122	41	3	58	58
65	254x102x22	21	58	66	121	40	3	58	58
66	203x133x30	21	55	64	122	40	3	55	55
67	203x133x25	21	55	62	117	39	3	55	55
68	203x102x23	20	55	60	114	39	3	55	55
69	178x102x19	20	54	58	108	38	3	54	54
70	152x89x16	19	52	56	102	37	3	52	52
71	127x76x13	19	51	53	96	36	3	51	51

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Table 5.2 Comparative costs of column fabrication as varied by detailing complexity

Total cost of fab'ted member = Length (m)* Cost/m + Main + (1or 0)*Base(L,M,or H) + (1,2 or 0)*Splice (L,M or H) + (1,2 or 3)*Beam (L,M or H)
From Table 3.2

All figures 1994 based

COL REF	U.C. SERIAL SIZE	FABRICATION COST COMPONENT									
		MAIN	BASE DETAIL			SPLICE DETAIL [note1]			BEAM CONNECTION		
			"L"	"M"	"H"	"L"	"M"	"H"	"L"	"M"	"H"
1	356x406x634	119	136	159	609	51	66	75	72	120	457
2	356x406x551	103	128	148	587	47	61	70	65	114	436
3	356x406x467	88	95	112	525	43	56	65	61	110	405
4	356x406x393	74	75	90	357	40	51	60	55	106	306
5	356x406x340	65	73	87	346	37	47	57	52	104	298
6	356x406x287	55	70	85	336	35	44	53	50	100	289
7	356x406x235	46	57	71	242	32	40	50	47	98	233
8	356x368x202	40	54	67	234	30	38	47	46	93	224
9	356x368x177	35	53	66	177	29	36	45	44	92	187
10	356x368x153	31	53	66	174	28	34	44	43	91	184
11	356x368x129	28	45	58	124	27	33	42	41	89	158
12	305x305x283	55	61	73	307	34	44	53	50	90	256
13	305x305x240	47	59	70	298	32	41	50	48	88	249
14	305x305x198	39	49	60	215	30	38	47	46	86	202
15	305x305x158	32	48	59	207	28	35	44	43	82	195
16	305x305x137	29	47	59	157	27	33	43	41	81	162
17	305x305x118	27	41	52	113	26	32	41	40	80	139
18	305x305x97	24	40	51	111	25	31	40	39	79	137
19	254x254x167	34	44	53	189	22	27	31	43	77	173
20	254x254x132	28	43	53	181	20	25	28	40	75	167
21	254x254x107	25	42	52	138	19	23	27	39	72	138
22	254x254x89	23	37	46	100	18	22	25	38	71	118
23	254x254x73	21	36	46	98	18	21	25	37	71	116
24	203x203x86	23	38	46	121	18	22	26	37	65	117
25	203x203x71	21	33	42	90	17	21	25	36	64	101
26	203x203x60	20	33	41	88	17	20	24	35	64	99
27	203x203x52	19	33	41	87	17	20	24	35	63	98
28	203x203x46	18	33	41	86	17	20	24	35	63	97
29	152x152x37	17	30	37	76	16	19	23	33	56	81
30	152x152x30	16	30	37	75	16	19	23	33	55	79
31	152x152x23	15	29	36	74	16	19	22	32	54	78

Notes

1. Splice costs allow only for main unit drilling operations and preparation of splice plates plus material costs of splice plates and bolts, but does not include the labour costs of bolting operations (normally an erection function).

Notes on the following Comparative Cost Examples

1. The following pages show examples of comparative costing of building frames using the tabulated fabrication cost information in this Section and material costs from Section 3 of this document. These cost figures are **relative** costs of steel frames, **not absolute** costs.

2. The costs in Section 3 are based on British Steel list prices current at the time of preparation of this document (1994), but these do not allow for additional costs of transportation, tonnage variations or particular discounts that may be offered in the market at any particular time.

3. The fabrication costs were derived by the collaborating fabricators as a reasonable comparative estimate of labour and fitting materials costs associated with each connection style, compared with the basis of the materials costs used in the calculations.

4. The frame costs derived in these examples will **not** therefore reflect an **absolute** price for the steel frame presented, but are believed to show the relative merits of increased material use against reduced fabrication content. The frame costs derived should not therefore be used to compare steel framed construction with alternative forms.

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Comparative costing examples

Using the worked examples used to examine connection styles these frames can now be costed using the fabrication costing tables and material cost tables of Section 3.

Example 1

For Beams

305 UB 37	material cost	5.5*14	=	77.00
	main unit cutting etc.			23.00
	1 end extended endplate	0.5*73	=	36.50
	1 end short haunched	0.5*140	=	70.00
				206.50 / beam

For Central Column

203 UC 46	material cost	9.0*17	=	153.00
	main unit cutting etc.			18.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, tension stiffened	2*63	=	126.00
				347.00 / column

For Outer Columns

203 UC 46	material cost	9.0*17	=	153.00
	main unit cutting etc.			18.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, comp. & tension stiff'd	2*97	=	194.00
	and shear stiffened	2*0.5*97	=	97.00
				512.00 / column

Alternative Heavier outer column

203 UC 71	material cost	9.0*27	=	243.00
	main unit cutting etc.			21.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, no stiffening	2*36	=	72.00
				386.00 / column

Frame costs

Original	4No beams	4* 206.5	=	826.00
	Central column	1* 347	=	347.00
	Outer columns	2* 512	=	1024.00
				2197.00 / frame
Alternative	4No beams	4* 206.5	=	826.00
	Central column	1* 347	=	347.00
	Outer columns	2* 386	=	772.00
				1945.00 / frame - ***11% Saving.***

Note - ***savings do not include additional benefits of secondary members framing into un-stiffened columns.***

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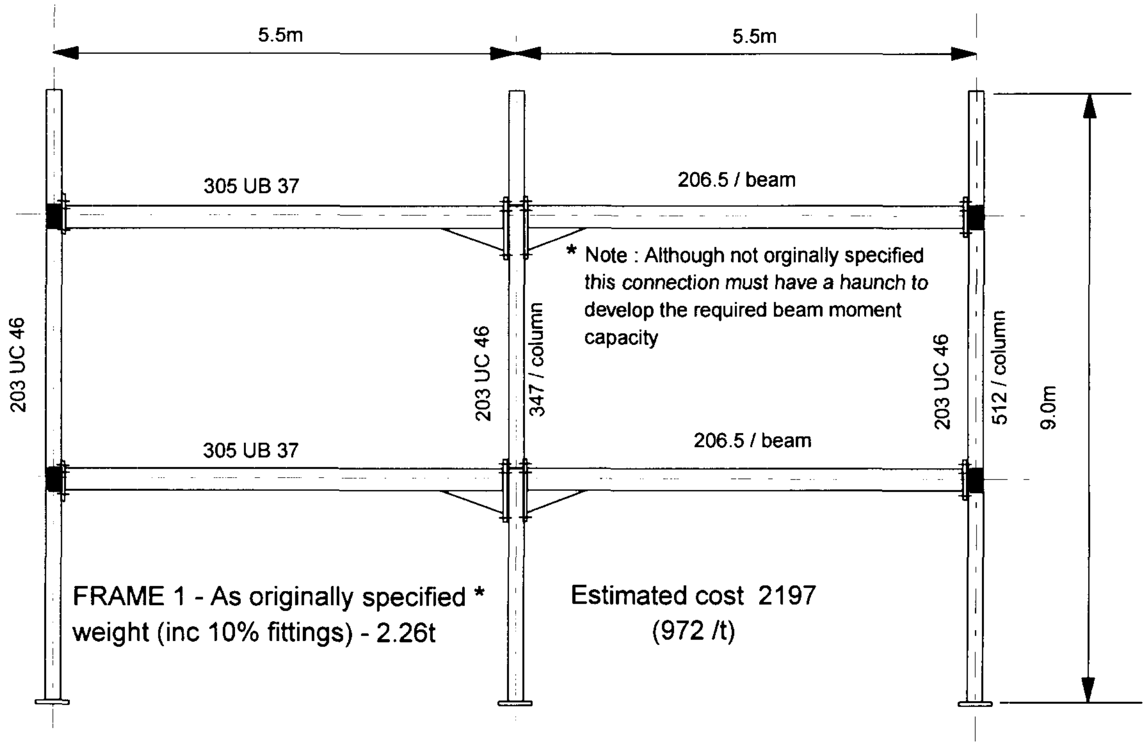
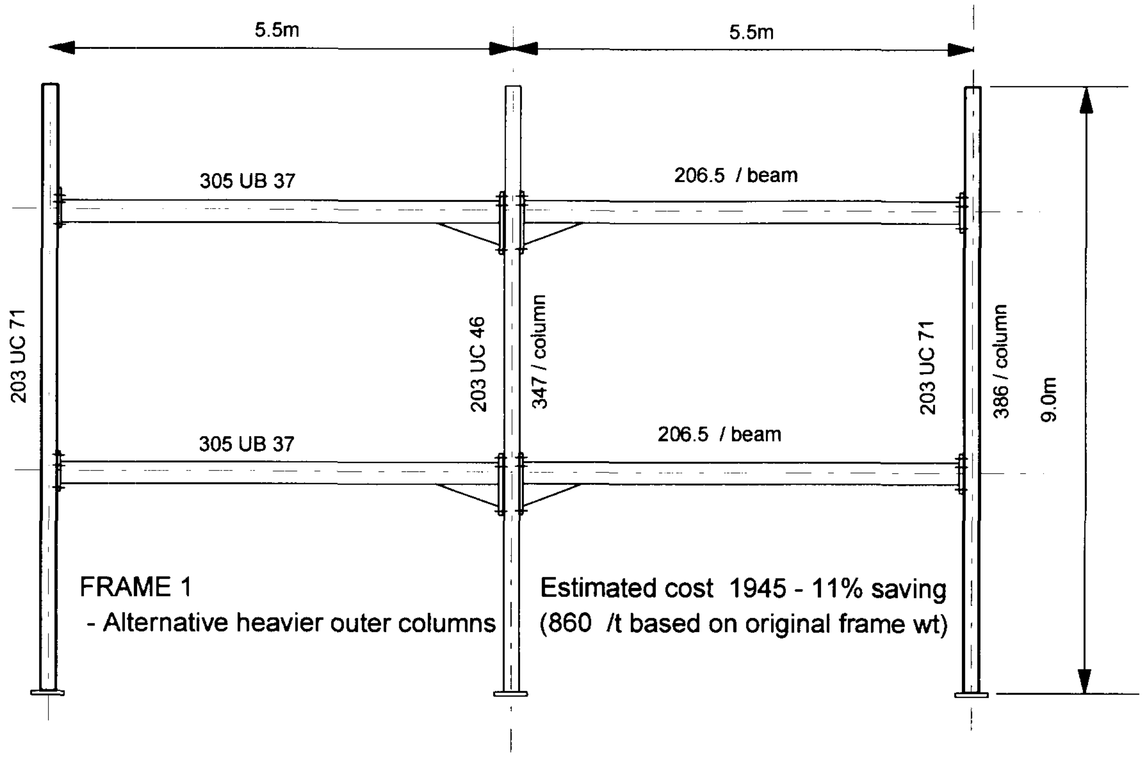
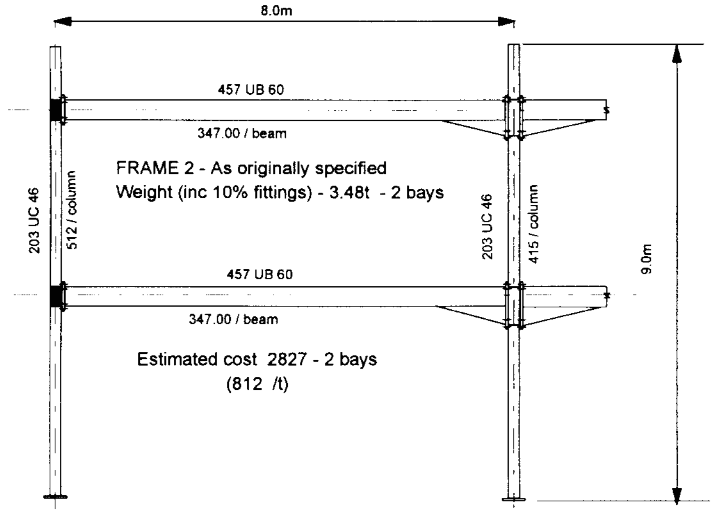




Example 2



For Beams

457 UB 60	material cost	8.0*23	=	184.00
	main unit cutting etc.			26.00
	1 end extended endplate	0.5*92	=	46.00
	1 end short haunched	0.5*182	=	91.00
				347.00 / beam

For Outer Columns

203 UC 46	material cost	9.0*17	=	153.00
	main unit cutting etc.			18.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, comp. & tension stiff'd	2*97	=	194.00
	and shear stiffened	2*0.5*97	=	97.00
				512.00 / column

For Central Column

203 UC 46	material cost	9.0*17	=	153.00
	main unit cutting etc.			18.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, comp. & tension stiff'd	2*97	=	194.00
				415.00 / column

Frame costs

4No beams	4* 347	=	1388.00
Central column	1* 415	=	415.00
Outer columns	2* 512	=	1024.00
			2827.00 / frame

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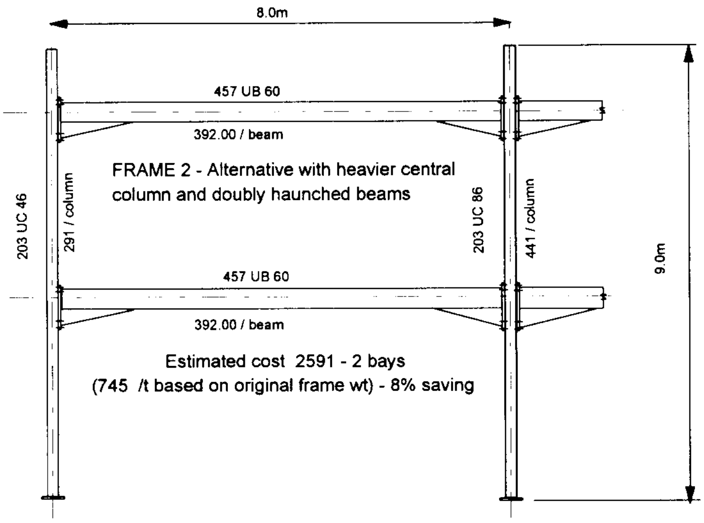

Alternative Heavier central column

203 UC 86	material cost	9.0*32	=	288.00
	main unit cutting etc.			23.00
	pin base			38.00
	nominal splice			18.00
	2No beam conns, no stiffening	2*37	=	74.00
				441.00 / column

Small penalty for increasing central column weight.

Alternative double haunched beams to eliminate external column stiffening.

For Beams

457 UB 60	material cost	8.0*23	=	184.00
	main unit cutting etc.			26.00
	2 ends short haunched			182.00
				392.00 / beam

For Outer Columns

203 UC 46	material cost	9.0*17	=	153.00
	main unit cutting etc.			18.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, no stiffening	2*35	=	70.00
				291.00 / column

Alternative frame cost

4No beams	4* 392	=	1568.00
Central column	1* 441	=	441.00
Outer columns	2* 291	=	582.00
			2591.00 / frame - ***8% saving.***

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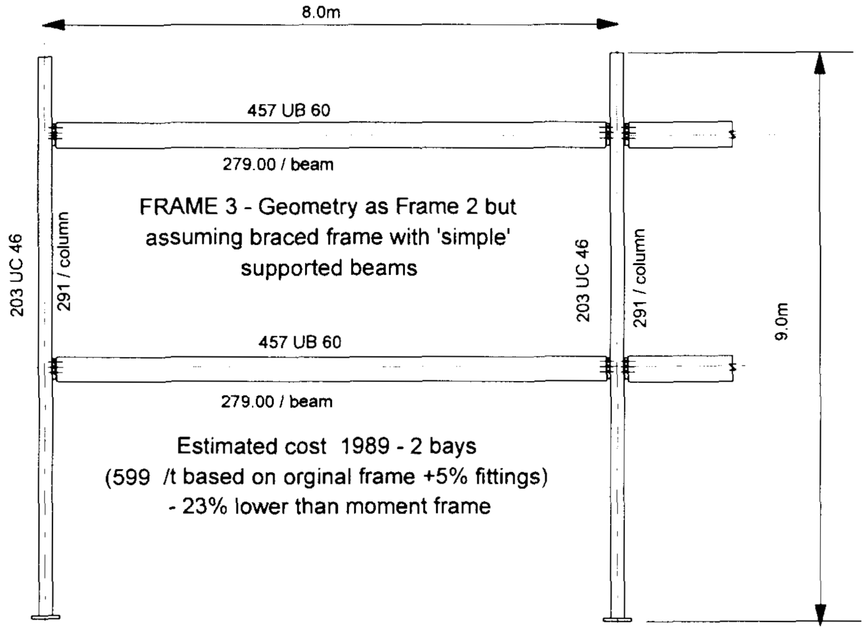
By way of illustration only, using the same frame geometry as example 2 i.e. 203UC46 throughout with 457 UB 60 beams, but assuming a braced frame with simply supported beams. (It should be remembered that such a frame would not have as large a vertical loading capacity as the equivalent moment frame and that the following costs do not allow for bracing fabrication.)

Example 3



For Beams

457 UB 60	material cost	8.0*23	=	184.00
	main unit cutting etc.			26.00
	2 ends simple endplates			69.00
				279.00 / beam

For Columns

203 UC 46	material cost	9.0*17	=	153.00
	main unit cutting etc.			18.00
	pin base			33.00
	nominal splice			17.00
	2No beam conns, no stiffening	2*35	=	70.00
				291.00 / column

Simple frame cost

4No beams	4* 279	=	1116.00
Columns	3* 291	=	873.00
			1989.00 / frame

- 23% less than moment frame.

6. BOLTS AND BOLTING

BOLTS AND BOLTING

Introduction

It is preferred to have some form of standardisation in the most commonly used general structural bolts. At present bolts vary by grade, diameter, length, thread length, type and finish. This involves time consuming bolt listing, onerous store keeping and costly site operation. It complicates purchasing and from a safety aspect there is more risk of fixing bolts in the wrong locations. The fact that standardisation would lead to overall economies in the industry is indisputable. The main obstacle to the implementation of such standardisation is the specifier. There are still many specifications currently being issued which require no threaded parts in the shear planes.

A general acceptance of threaded parts in shear and bearing, as permitted in BS5950 and EC3, is required, together with the recognition that there will be a larger bolt projection through the nut. (Fully threaded bolts are the subject of a paper by G W Owens in The Structural Engineer - Volume 70 - No. 17 - 1 September 1992).

Standardising bolts as suggested below will produce worthwhile cost savings :

Grade and Dimensions

Mechanical properties to BS 3692 : 1967
Bolts to Grade 8.8
Nuts to Grade 8 minimum
Dimensional properties and tolerances to BS 4190 : 1967

Diameters

M20 and M24

Thread Length

Fully threaded up to 70 mm long

Washers

Not required for strength when used in normal clearance holes except to preserve the surface finish.

Length

There has been much discussion on the subject of length. The specific lengths chosen for standardising may vary slightly from company to company, depending on the throughput of differing types of fabrication work.

The actual lengths chosen are not important as the quantities involved would be large enough to ensure economical purchasing.

Note that for fully threaded bolts the maximum economical length is 70 mm.

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High Strength Friction Grip (HSFG) Bolts (Non-slip connections)

These bolts are more expensive to purchase and much more costly to install and inspect. There are further costs involved in masking off contact surfaces and touching up after erection. Only where joint slip is unacceptable should the following types of bolt be used :-

1. HSFG Bolts to BS 4395 : Part 1 (bolt material equivalent to grade 8.8)
2. HSFG Bolts to BS 4395 : Part 2 (bolt material equivalent to grade 10.9)
3. Torsion control bolts (TC) (bolt material equivalent to grade 10.9)

Various tightening methods may be used to achieve the clamping force in the HSFG bolt i.e.

a) Use of load indicating washers.
b) Part turn method.
c) Torque control method.

Careful consideration needs to be given to an appropriate choice of tightening method compatible with the bolt and nut finishes and careful use of lubricants during installation. The following table indicates such combinations and preferences.

	HSFG Tightening Method		
Bolt Finish	**Load Indicator**	**Part Turn**	**Torque Control**
Black	Preferred	Can be used	First alternative
Coated (Galv, Sher, Pltd)	Preferred	First alternative	Do <u>not</u> use

The use of Torsion Control bolts is seen as equally preferable to the use of load indicating washers with HSFG bolts, although particular care is required with coated TC bolts to ensure that original lubrication conditions which existed on the black bolt is maintained at the time of installation. For this reason also the use of coated Torsion Control bolts is not recommended.

Finishes

The choice of finish required on fasteners should be compatible with and comparable to the performance required of the finish called for in the connected components. This could mean that 'black' finish bolts are perfectly acceptable or that one of a number of possible surface coatings could be used. If a company has already chosen to standardise on the grade, diameter, lengths and threading to minimise bolt types and gain the advantages of such standardisation, it is natural that they will have standardised on a finish also. This is usually set at the 'higher' level of protection requirements to cater for all possible locations, additional costs of surface finish where not actually required being outweighed by the benefits of standardisation.

Commonly chosen finishes are as follows:

a) Zinc plated to BS 7371:Part 3

This produces a coating of zinc which can be considered as giving a reasonable level of protection to bolt assembly during the erection period, but is not suitable for longer duration exposure. Typical coating thickness is 7.5 microns, although some 'commercially plated' fasteners may only have coatings of 4 micron.

This is a suitable finish for grade 8.8 bolts.

b) Spun galvanized to BS 729

This produces a durable coating of zinc/iron alloy and zinc to a minimum thickness of 43 microns.

The components are acid pickled or blast cleaned, placed in a perforated container, dipped in a zinc bath, then spun in a centrifuge to remove excess zinc.

The temperature of the bath varies between approx. 450°C - 550°C, considerably lower than the temperatures used for heat treatment of the bolts, consequently the mechanical properties are not impaired by the process.

Nuts are usually galvanized as pierced blanks and then tapped 0.4 mm oversize to allow for the zinc coating on the bolt.

This is a suitable finish for grade 8.8 bolts. Note that if grade 10.9 or HSFG bolts to BS4935:Part 2 are to be galvanized, these bolts must be blast cleaned not acid pickled prior to coating.

c) Sherardized to BS 4921

Class 1 : gives a minimum coating thickness of 30 micron.
Class 2 : gives a minimum coating thickness of 15 micron.

The components are blast cleaned, placed in a container with zinc dust and an inert filler. The container is placed inside a furnace and rotated at a temperature of about 350°C. The coating is a layer of zinc/iron alloy and zinc. The nuts are then tapped or re-tapped as for the galvanized process.

This is a suitable finish for grade 8.8 and grade 10.9 bolts.

d) Mechanical Plating and Galvanizing to ASTM B695-83 (BS 1971 under preparation)

Gives a minimum coating thickness of 30 micron.

Components are cleaned in alkali solution and placed in a rotating barrel with an impact media. (i.e. glass beads in differing diameters).

Further chemical treatment adds a 'flash' coating of copper after which zinc dust is added and a zinc layer is built up by impact peening onto the surface.

This is a suitable finish for 8.8 and 10.9 bolts and is the only finish allowed for use on Torsion Control bolts.

All the above finishes can be further treated by passivation i.e. immersion in a phosphate or chromate bath. This reduces the risk of the formation of 'white rust' and gives a better finish. Such passivation is an essential requirement for zinc plated finish, and may be of benefit to sherardized coatings. Passivation is rarely used with galvanized finishes and may not be available at most galvanizing operations

Costs (1994 Prices)

By way of indication of relative costs of ordinary and preloaded bolts the following material costs may be considered per 100 No. bolts. Costs will vary by say +/- 5% depending on shank length. Quoted costs include for a nut and bolt assembly, including flat round washers for ordinary bolts and a hardened plus a load indicating washer for the pre-loaded bolts. Prices do not include labour or small tools costs for installation and inspection. Installation labour costs would be approximately three times greater for preloaded bolts compared to the equivalent size ordinary bolt.

Ordinary bolt assembly	M20	£ 32.00 / 100 No.
	M24	£ 62.00 / 100 No.
HSFG bolt assembly	M20	£ 55.00 / 100 No.
	M24	£107.00 / 100 No.

All costs shown are for black finish bolts ; typically add 30% for spun galvanised finish.

Holding Down Bolts

The preferred bolts are square headed, square necked bolts in grade 4.6 or grade 8.8, to BS 7419 : 1991, normally supplied 'black'. It should be noted that specification of both strength grades on the same project should be avoided, if this is not possible then different grades should not be specified with the same diameters to avoid confusion during installation.

The preferred diameters and lengths are as follows:-

M20	x 450 x 600
M24	x 450 x 600 x 750
M30	x 450 x 600 x 750
M36	x 450 x 600 x 750

The above bolts are readily available, specifiers should endeavour to keep to these sizes, as Holding Down bolts are usually required at short notice.

Anchor flats are provided for each bolt, with square holes to suit the square necks and to prevent bolt rotation. These flats should be of adequate net size and thickness to safely distribute the tensile load into the concrete. It should be noted that even 'nominal' Holding Down bolts can have appreciable tensions applied during erection and that the safety of the structure can depend on these bolts during this stage; specify minimum of 4 bolts per base.

The practise of welding nuts and bolt heads is not permissible; if a welded 'nut' is required this should be formed from a tapped boss of weldable grade steel. A welded threaded stud may be formed using threaded bar of weldable grade material.

7. WELDING AND INSPECTION

WELDING AND INSPECTION

The Fusion Welding Process

By far the most widely used connection technique in welded construction today is fusion welding; otherwise referred to as arc welding.

The heat source which melts and fuses the parent material is provided by an electric arc. This is a high current, low voltage discharge in the range of 10-2000A and 10-50V, depending on the welding process. The arc is formed between the electrode and the workpiece or parent metal. The electrode can be either consumable or non-consumable.

The heat generated by the arc forms a melt in the parent metal, into which a flow of liquid metal droplets of filler material are transferred across the arc. The depth of penetration of the melt into the parent metal is controlled by the arc energy (normally expressed in kJ/mm), which is a combination of arc voltage, amperage and welding speed.

The deposited molten metal needs to be protected from the atmosphere until solidification to prevent oxidation and embrittlement. This can be achieved by the use of either flux or gas shields.

The most common fusion welding processes are as follows :

a) **Shielded Metal Arc Welding (SMAW)**
(Also referred to as MMA - Manual Metal Arc welding)

This is one of the oldest forms of fusion welding but is still widely used. The solid electrode or 'stick,' which is consumed, consists of a core of filler wire coated with flux which consists of various silicates, metals and metal oxides. During welding the flux melts to form a viscous slag which provides a protective layer between the molten weldpool and the atmosphere. In addition the flux is used to stabilise the arc and can transport alloying additions to the weldpool.

b) **Gas Metal Arc Welding (GMAW)**
(Also referred to as MIG - Metal Inert Gas or MAG - Metal Active Gas welding - the gas being a gas shield not a heat source)

The basic physics of this process is similar to that of SMAW, using an arc struck between a consumable electrode and the workpiece. However in GMAW the electrode is a continuous and flexible spool of wire, automatically fed through the welding gun as it is consumed in the weldpool. The arc is protected from oxidation by a gaseous shield also supplied via the welding gun nozzle. The gas may be inert, such as carbon dioxide, or have active components to assist in the welding process. The gaseous shield is still suitable for welding in all positions, assuming due attention is paid.

A further development of this process is Flux Cored Arc Welding (FCAW), where the centre of a hollow wire electrode is filled with flux to assist in solidifying weld and allow alloying additions.

Both of these processes have greater productivity levels compared to that of SMAW, as the operator does not need to periodically stop welding to replace electrodes. Similarly the welders skill requirement is reduced as the welding gun remains at a constant distance from the workpiece rather than a varying level as in SMAW with ever shortening electrode stick.

c) Submerged Arc Welding (SAW)

Submerged arc welding is similar to those of GMAW or FCAW in that a continuos wire feeds the electrode. It differs in that the weldpool is protected by being submerged under a granular flux. The flux is normally deposited from a hopper just in front of the arc, the heat of the arc melts the flux to form a slag and any surplus flux is recovered by suction and refills the hopper. This entire process is normally undertaken via an automatic self-propelling machine. The process is only suitable for welding in the downhand position and is normally only undertaken on substantial continuos runs of weld, such as occur on the flange to web welds of plate girders. The process also has the advantage of the enclosed arc not requiring adjacent operators to require safety protection from arc light flashes.

d) Stud Welding

This form of welding occurs mainly in composite construction where headed studs are to be fixed to the top flange of a beam either directly to the flange or through associated metal decking. Threaded studs can be attached in a similar manner. In this process the stud is placed into a special welding gun to act as the electrode. The energy source is a capacitor stored charge which is discharged as the gun is placed above the beam flange. The discharge melts the end of the stud shaft and the molten metal is contained by a surrounding porcelain collar fitted to each stud during the loading operation.

Whilst there are other forms of fusion welding these are more specialised in nature and do not normally find application in normal steel-framed structures and so are not discussed in this document.

Choice Of Process

The choice of welding process depends on a number of factors and in factory conditions these are:-

a) the plant available.
b) the suitability of the process to the particular application.

Most factories will have plant to carry out SMAW, GMAW, and stud. SMAW and GMAW can be used for all purposes but are better suited to shorter runs of weld i.e. fittings or where, particularly in the case of FCAW welding, heavy rates of deposition are required. Naturally the easiest position for welding together with the best deposition rates is downhand, the other positional welds require more skill.

Submerged arc welding is most suitable for long continuous runs and therefore is commonly used in the forming the web to flange welds of plate girders.

Stud welding can be carried out in the factory when through-deck studding is not required, although this is becoming less common other than for bridge beams etc.

Site welding

Although site welding has not traditionally been favoured by many UK fabricators due mainly to concerns over lack of control in an external environment, it should not be neglected as a possible means of joining members. It is used extensively by the gas and oil industries and is in common use outside the UK.

If structures are designed with simple repetitive site welds, executed and tested by specialists, the resulting structure can be continuous, elegant and cheaper than the bolted alternative.

Site welded joints require even more consideration with regard to location for access and equipment to give protection and to carry out the welding and inspection. Also forms of temporary fixing to secure the joint in position and provide adjustment prior to welding need investigation. These may need to take the erection loads and whilst providing sufficient space to allow adequate welding to secure the joint prior to their removal prior to completion of the joint.

Weld Details

General

The amount of welding has a significant influence on the overall cost and fabrication time of any project. The main message is **don't over specify**. Problems arising from poor fit-up causing large gaps can often be avoided by consideration of alternative details. Similarly good access for welding not only assists in forming good welds but also in maintaining cleanliness whilst welding is in progress.

The secrets of good welding are standardisation and repetition, cleanliness of materials and proper fit-up, inspection after fit-up as well as after welding, use of qualified welders with familiar procedures, consumables and equipment.

For companies that have neither the technical resources nor the desire to produce their own welding procedures the Weldpro package produced by TWI may be of advantage.

Fillet welds

A fillet weld is a fusion weld approximately triangular in cross section which joins two faces of steel surface which are not (normally) the cross sectional cut surface of the material.

Fillet welds should not be less in leg length or throat thickness than specified. The throat thickness should be taken as 0.7 times the leg length (for fillets joining faces at 90°). Fillet welds should be returned around the corners for a distance of not less than twice the weld size, or if this is not done then the effective length of the weld should be reduced by twice the weld leg size (in accordance with BS 5950).

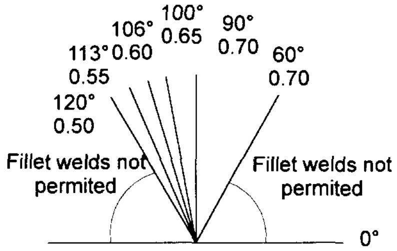
To avoid the possibility of hydrogen cracking when welding certain thicknesses of material minimum sizes of weld should be used or the parent metal pre-heated. The requirements for

Discuss me ...

pre-heating depend upon the carbon equivalent of the parent metal, the hydrogen potential of the welding process and are given in BS 5135 appendix E.

Where fillet welds are used to connect fusion faces which vary from 60 to 120 degrees the effective throat thickness relative to leg length should be as shown in the following table :-

60 to 90 degrees	0.70
91 to 100 degrees	0.65
101 to 106 degrees	0.60
107 to 113 degrees	0.55
114 to 120 degrees	0.50



Intermittent fillet welds

These are used where the loading is low enough not to require continuous welding. They must not be used in locations where there is a danger of ingress of moisture to cause corrosion. Furthermore, the fabricator should be allowed to offer a similar sized or slightly smaller continuos fillet weld as an alternative as this may in fact be a cheaper solution for him than the marking requirements of the intermittent weld and the possible problems caused by numerous stop/starts in the weld. One advantage of intermittent welds is the lower likelihood of distortions on long runs as there is a reduced heat input compared to the equivalent continuous fillet.

Butt welds

A butt weld is a weld in which the cross section of the member being welded is fully or partially joined usually by preparing one or both faces to provide a suitable angle for welding.

A **full penetration butt** weld is that which fully welds the entire cross section of the welded member and usually develops the same load carrying ability.

A **partial penetration butt** weld, as its name suggests, does not fully weld the cross section. In BS 5950 if the whole section is partial penetration butt welded then 3 mm of the section is neglected and the preparation must be at least $2\sqrt{}$ t, where t is thickness of the thinner plate.

Butt welds, both partial and full penetration, have the advantage of higher permissible stress levels than fillet welds, but are more susceptible to welding defects.

Weld Defects

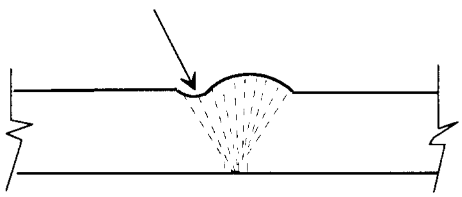
It is necessary to point out that in all welding some defects will occur. Provided that those defects do not exceed the specified limits this is perfectly acceptable.

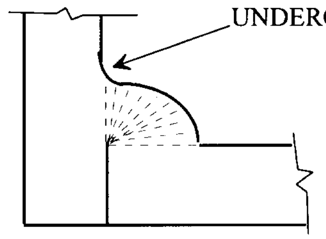
The following defects are the most common that a designer may encounter.

Discuss me ...

Undercut

Sketches of this defect are given below. The most common reason for this defect is when insufficient weld metal is incorporated into the weld pool as a result of poor welding technique or incorrect welding parameters and disturbance prevents deposition at the edges.





The effects of undercut are not particularly serious in the case of statically loaded structures provided they are within the limits given in the specification but particular attention should be given where this occurs in a fatigue condition.

Slag Inclusions

Slag inclusions are non-metallic particles trapped in the solidified weld and usually come from the flux. In multi-pass welds they may occur from incomplete removal of slag from previous runs of weld or from inadequate access due to poor joint preparation.

As these defects can be, relatively, quite large, particular care should be taken to adhere to the correct preparation, fit and procedure.

Incomplete penetration

Incomplete penetration occurs in two ways, firstly by failing to backgouge a root run back to sound weld metal or using too large an electrode for the geometry of the joint and secondly by not achieving an adequate welding angle or sufficient current in the weld pool.

Provided that this defect does not occur too deep in the weld it can be rectified fairly easily by gouging and rewelding. If it occurs deep in the weld an assessment of the effect of the defects needs to be carried out to see if the weld is fit for purpose prior to carrying out any repairs.

Lack of Fusion

This defect occurs when the runs of weld have not fully fused with the parent metal or other runs of weld. The reasons for this are similar to those for incomplete penetration i.e. lack of cleanliness, incorrect welding parameters and poor approach angle. Attention to good practice should eliminate or, at least, minimise their effect.

Discuss me ...

Porosity

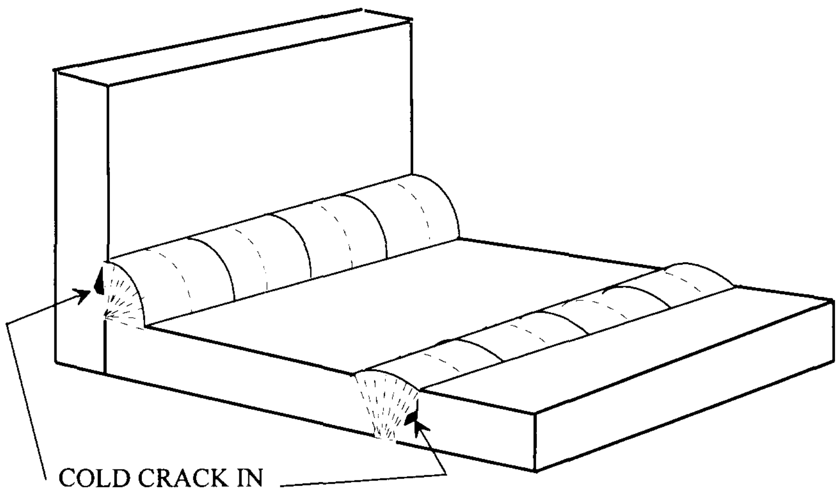
This defect is caused by the trapping of gas in the solidified weld giving rise to small cavities which can be spherical or 'pipes or worm holes'. Contamination of the parent material in the weld fusion zone can produce more gasses and exacerbate the problem.

Start porosity is caused when an arc is first struck and before the shielding properties of the coating or inert gas have had the chance to be fully effective.

All the above defects are dependent upon the skill of the welder to avoid or minimise there occurrence. This highlights the requirement to ensure adequately trained and certificated are employed for structural welding and that these welders are regularly re-tested.

Cold cracking, hydrogen or heat-affected zone cracking.

This type of defect usually occurs in the heat-affected zone as indicated in the following sketch.

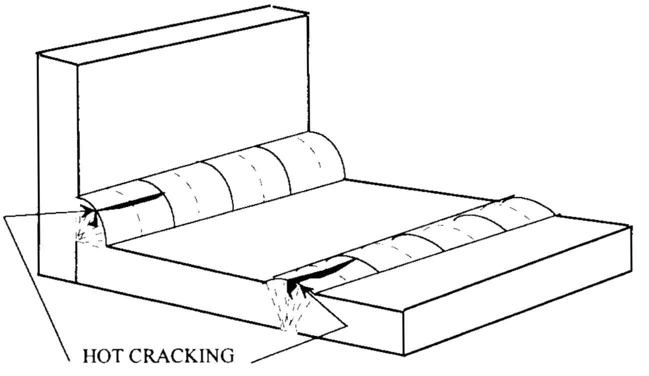


The cause of this defect depends upon the composition of the material, the cooling rate of the weld, the amount of restraint and the level of hydrogen.

The material being welded should have a known Carbon Equivalent so that if a material is susceptible to hydrogen cracking the proper precautions can be taken. The slower the rate of cooling the less chance there is of cold and hydrogen cracking because slow cooling lessens the hardening of the heat-affected zone and assists the diffusion of hydrogen. If the weld is in a highly restrained joint as it cools cracks will be more likely to form. Hydrogen levels can be minimised by pre-drying the electrodes and flux.

Hot cracking, solidus or weld metal solidification

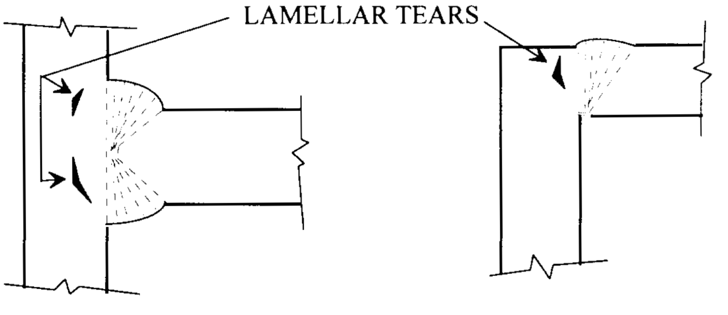
A sketch of a typical hot crack is shown below. As can be seen, this crack generally occurs in the centre of a weld shortly after solidification. It is formed by impurities in the weld pool which congregate in the centre which, being the last part of the weld to cool, forms a plane of weakness which during shrinkage causes the crack.



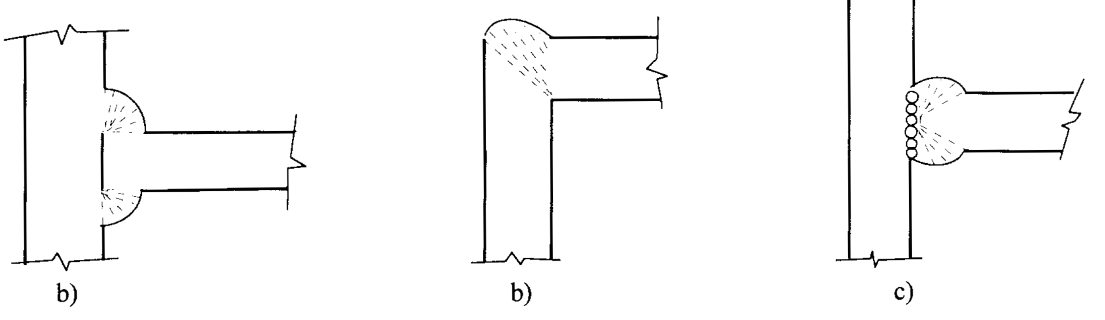
Lamellar tearing

If the material being welded to has poor 'through thickness' tensile properties, due to impurities in the steel elongated in the rolling process, tearing of the parent metal occurs when attachments are welded to it.

Typical sketches showing where lamellar tearing can occur are shown below :-



There are three common ways of overcoming this problem :-



a) to use through thickness tested material. This is material which has fewer impurities such as sulphur and from which tensile specimens have been taken to ascertain the ductility of the material.

b) to change the detail to avoid lamellar tearing as shown above

c) to butter the face of the material receiving the attachment with weld metal of high ductility to absorb the strain, as shown in the sketch below :-

Weld Repairs

Despite the care taken in the welding of joints there will be occasions when imperfections occur which are outside the specified limits. Before proceeding with a deep seated or difficult repair the magnitude and effect of the defect on the behaviour of the joint should be examined and its fitness for purpose ascertained. It could happen that the defect was in a non-critical joint or in a location where the member/joint was not fully stressed, in these cases it is better to leave the defect alone as more harm than good often occurs when trying to effect repairs.

Inspection Methods

There are five fundamental forms of inspection and non-destructive testing of welds.

a) visual	
b) dye penetrant inspection (DPI)	Inspection
c) magnetic particle inspection (MPI)	
d) ultrasonic flaw detection (UFD)	Testing
e) radiography.	

Visual

Visual checks should be carried out at all stages of the welding process to ensure that the welding procedure is followed and that the finished weld is free from surface defects.

The checks during production should verify the accuracy of 'fit-up', cleanliness of welded parts and preparation, and examination for cracking in both tacks and root runs, interpass cleanliness and the final weld geometry.

The completed weld should also not exhibit surface cracking, porosity or undercut greater than stated acceptable in the specification.

Visual weld inspection is one of the most cost efficient forms of NDT and it can also be one of the most effective if conducted by properly trained and motivated personnel.

Dye Penetrant Inspection (DPI)

Penetrant methods consist of a range of techniques in which a liquid is put on the surface of the specimen and given time to be drawn into any surface breaking cracks and cavities. The surplus liquid is removed from the surface and any liquid which has entered cracks etc. is made visible by developer, fluorescence or seepage. Normally penetrants are applied to one surface, but leakage defects can also be traced by checking for signs of the penetrant on the opposite side to that of application. In principle penetrant methods can be applied to all weldments to detect surface breaking cracks, but in practice magnetic particle methods are often preferred as the technique is more sensitive.

Magnetic Particle Inspection (MPI)

Magnetic particle inspection is a relatively inexpensive and simple system for detecting surface and some sub-surface cracks in ferro magnetic materials. It should be noted however that this system cannot be used when welding high carbon steels with stainless steel filler as the resulting weld metal is non-magnetic.

The principle of the method is that the specimen is magnetised such that magnetic lines of force are produced in the material. If these lines of force meet a discontinuity, such as a crack cutting the lines of force, secondary magnetic poles are produced at the faces of the cracks, and if these are near the surface they can be revealed by application of magnetic particles such as fine iron in powder or liquid suspension form. The technique is at its most sensitive when the cracks are at right angles to the magnetic flux.

Ultrasonic Flaw Detection (UFD)

As the name implies, ultrasonic waves are mechanical vibrations having the same characteristics as sound waves, but at frequencies above those audible to the human ear. For weld examination in metals the ultrasonic waves typically have frequencies of 1-5MHz. The most important type of waves for flaw detection in welds are compressional and shear waves. Most UFD methods use the pulsed echo technique in which a short ultrasonic pulse is emitted from a transmitter probe through a coupling medium to the material under test. During its travel this pulse is partially reflected from any discontinuities in its path and the 'echoes' produced are picked up by a receiving probe. This may be the same as the transmitter or be a second separate probe. The usual method of display is the 'A-scan' via an oscilloscope display screen.

All NDT systems rely on the skill and integrity of the operator. In the case of UFD this is of even greater importance as the level of manual and technical skill required to perform and interpret the instrumentation is very high.

Radiography

X-rays and gamma rays are of the same form of electro-magnetic radiation as that of visible light. However they have wavelengths that are so small that it enables them to penetrate all materials to some extent. The rays are progressively absorbed as they pass through the material, travelling in straight lines. They affect photographic emulsions in a similar manner to visible light. Hence if a radiation source is placed on one side of a specimen, and a sheet of radiographic film is placed on the other side of the specimen, because more radiation will pass through regions of lower density, as with a crack or cavity, this difference will register on the film. Once developed the film will show such cavities and cracks as darker areas with approximately the same size as the cross section of the cavity.

In the case of X-rays the radiation source is produced electrically either from mains supply or generator via a transformer. The radiation source for gamma-rays is a radio-isotope of limited strength, usually therefore requiring longer exposure times to produce a radiograph. As the gamma-ray source emits radiation continuously, it is kept in a heavy shielded container when not in use. The radiograph is produced by opening a small aperture in the container once in position.

Both X-ray and gamma-ray techniques require greater care of execution than other inspection methods to ensure the safety of personnel in the area; only trained and approved personnel should operate the equipment. The main advantage of radiography is the permanence of the inspection record film produced.

None of the inspection methods have the capability of detecting all the possible defects that can occur in welded construction. The type position and orientation of possible defects should be considered when selecting the inspection method.

General guidance on inspection methods, location and frequency of testing as well as defect acceptance criteria can be found in the National Specification for Structural Steelwork for Building Construction - BCSA & SCI publication No. 203/91.

Residual Stresses

After any welding, due to shrinkage, there will always be residual stresses. This should not normally give rise to any concern as time and flexing of the structure will relieve these stresses.

The exception to this are highly restrained joints. Here particular care should be taken in the sequence in which the welds are laid to avoid high residual stresses.

The most popular method of stress relieving by placing the item in a relieving furnace is not an option usually open to the structural fabricator.

Choice Of Materials

The most common materials used in structural steelwork are carbon manganese steel supplied to BS EN 10025 or BS 4360.

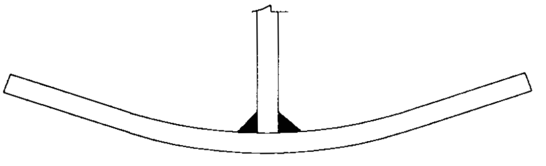
In choosing the steel grade it is necessary to consider the material properties such as tensile strength, yield strength, percentage of elongation and the toughness required (normally expressed as an impact energy J at a test temperature). Further considerations are the weldability of the material. BS 5135 Process of arc welding of carbon manganese steels, specifies these requirements for arc welding of these steels.

Some steels used in structural engineering, are quite unsuitable for welding. This particularly applies to nuts & bolts. The practice of welding these where access is difficult to apply the nut is to be deprecated and tapping or similar solutions should be sought.

Control of welding distortion

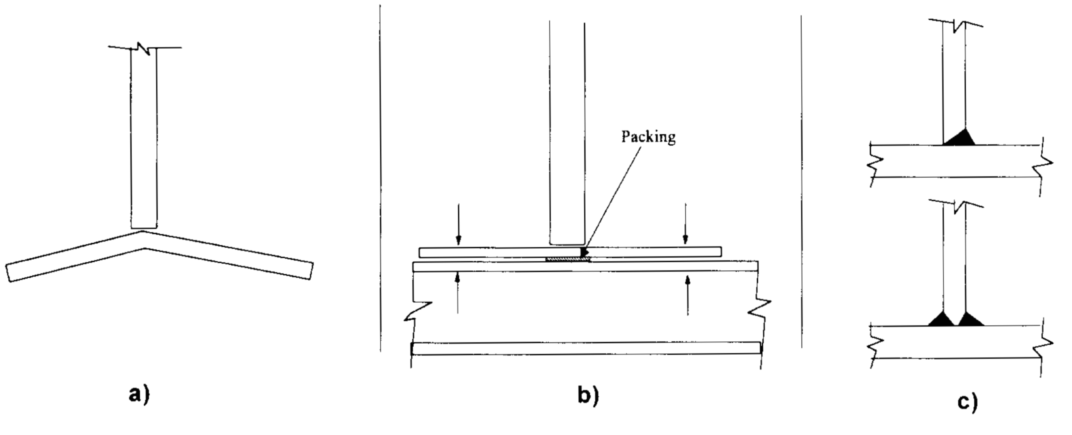
As was stated in the case of residual stresses you cannot weld and not have shrinkage. This naturally gives rise to distortion. There are some steps that can be taken in detailing to minimise the effect of welding distortion. The amount of distortion is dependent upon the volume and location of the weld.

As can be seen from the sketch below if there are heavy welds either side of an end plate the plate will become convex because when the welds (which are some distance apart) shrink they will pull the plate in a convex manner.



There are four possible ways of minimising this distortion :-

a) by prebending the plate
b) by clamping a 'strongback' to the plate
c) by butt welding or partial penetration butt welding
d) by increasing the thickness of the plate.



In the same way, but less predictable, is longitudinal shrinkage. When constructing plate girders, for instance, it is advisable to create the I section and trim to length afterwards. When butt welding the ends of plates together, depending upon the geometry of the preparation, it is advisable to tilt the plates in relation to each other to compensate for the movement.

Choice and cost comparison of types of welds

There is no doubt that the cheapest and most effective weld is the fillet, particularly in the downhand position. It is not usually plagued with fit up problems, is easy to examine before and afterwards by

simple NDT. It follows that wherever possible fillet welds should be adopted up to a suggested maximum of 12 - 15 mm leg length.
The next most cost effective weld is the partial penetration butt weld. It has the advantage of higher permissible stress levels than fillet welds but has the additional cost of preparation.

Finally, if there is no other sensible alternative, full-penetration butt weld. This is the most expensive and carries the additional burdens of additional material preparation and usually combined with a stringent non-destructive testing regime.

To try to make a comparison of the relative costs if we take the laying down of a 6 mm fillet weld in the downhand position as 1 then the other forms are given as multiples of this :-

6 mm fillet weld in downhand position		1.0
6 mm fillet weld in vertical position		2.0
6 mm fillet weld in overhead position		3.0
For each additional run to the above	multiply by 1.75	

Note :	6 mm weld	1 run
	8 mm weld	1/2 runs
	10 mm weld	2/3 runs

Open square butt weld in 10 mm plate	4.0
Single V butt weld in 10 mm plate	6.0
Double V butt weld in 20 mm plate	12.0
Single U butt weld in 20 mm plate	10.0
Double U butt weld in 40 mm plate	20.0
Single J butt weld in 20 mm plate	9.0
Double J butt weld in 40 mm plate	18.0
Single bevel butt weld in 10 mm plate	5.0
Double bevel butt weld in 20 mm plate	10.0

These factors are exclusive of preparation or testing times as many fabricators, not having the specialised machinery available, may opt for having the preparation carried out by others. Similarly, inspection is frequently carried out by outside agencies and the time taken for inspection depends upon their availability.

Manufacturing

If UFD or MPI is called for it is important to note that for certain welds (see National Structural Steelwork Specification for Building Construction) there are mandatory hold periods, varying from 16 to 40 hours, before UFD or MPI examination can be carried out.

The disruption to the fabrication process that these tests require are considerable and if possible details should be adopted to avoid this delay.

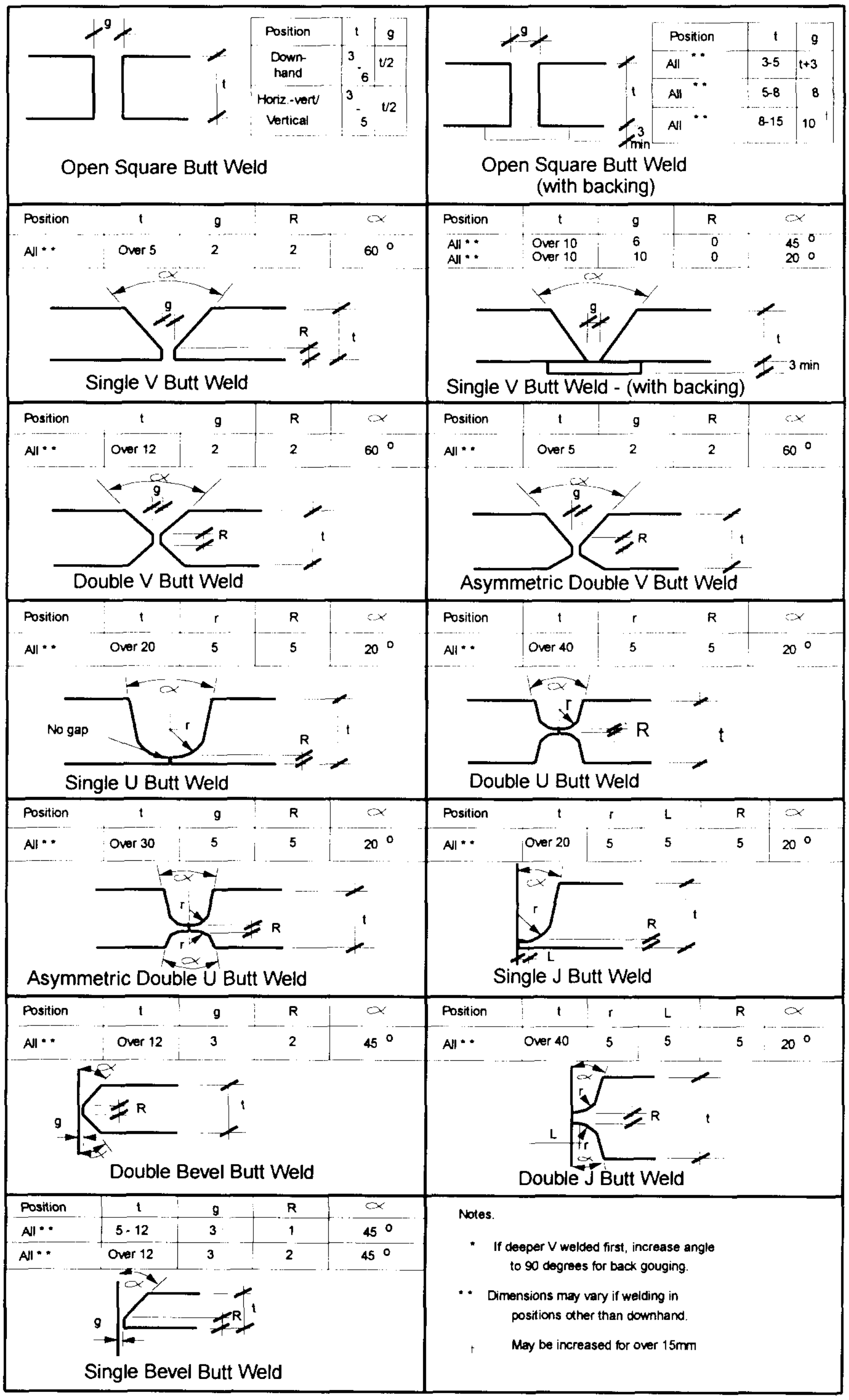
If common details are adopted for a number of similar items, manipulators can be used to avoid the use of cranes in turning the items and cost savings achieved.

Discuss me ...

Design For Manufacture Guidelines

CIMsteel

Commonly Used Weld Preparations



Open Square Butt Weld

Position	t	g
Down-hand	3 - 6	t/2
Horiz.-vert/ Vertical	3 - 5	t/2

Open Square Butt Weld (with backing)

Position	t	g
All **	3-5	t+3
All **	5-8	8
All **	8-15	10 †

Single V Butt Weld

Position	t	g	R	α
All **	Over 5	2	2	60 °

Single V Butt Weld - (with backing)

Position	t	g	R	α
All **	Over 10	6	0	45 °
All **	Over 10	10	0	20 °

Double V Butt Weld

Position	t	g	R	α
All **	Over 12	2	2	60 °

Asymmetric Double V Butt Weld

Position	t	g	R	α
All **	Over 5	2	2	60 °

Single U Butt Weld

Position	t	r	R	α
All **	Over 20	5	5	20 °

Double U Butt Weld

Position	t	r	R	α
All **	Over 40	5	5	20 °

Asymmetric Double U Butt Weld

Position	t	g	R	α
All **	Over 30	5	5	20 °

Single J Butt Weld

Position	t	r	L	R	α
All **	Over 20	5	5	5	20 °

Double Bevel Butt Weld

Position	t	g	R	α
All **	Over 12	3	2	45 °

Double J Butt Weld

Position	t	r	L	R	α
All **	Over 40	5	5	5	20 °

Single Bevel Butt Weld

Position	t	g	R	α
All **	5 - 12	3	1	45 °
All **	Over 12	3	2	45 °

Notes.

* If deeper V welded first, increase angle to 90 degrees for back gouging.

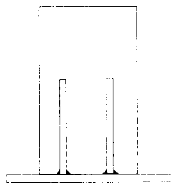
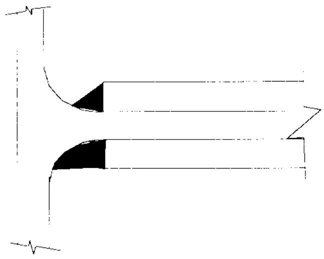
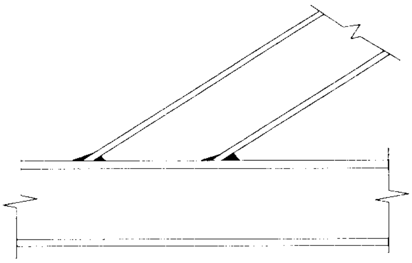
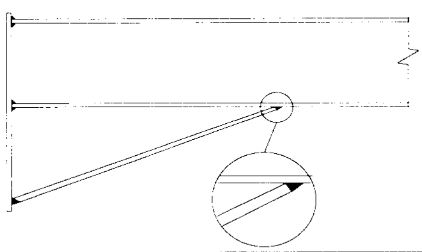
** Dimensions may vary if welding in positions other than downhand.

† May be increased for over 15mm

Design For Manufacture Guidelines

CIMsteel

In all welding it is essential to have good access if satisfactory welds are to be obtained and illustrated below are some of the most common faults to be avoided:-

DETAIL	COMMENT
	Problem : Welding to base of closely spaced gussets. Comment : Increase spacing to allow access for MIG or MMA or Use single sided butt weld with access from outer face
	Problem : Welding plates to web of rolled sections. Comment : Both of the welds shown would tend to have reduced side wall fusion caused by poor access - should not be used to transmit shear or tension, may be used as part of all round weld to web doubler plate
	Problem : Angle of intersection leads to poor access to accute fillet and small throat to obtuse fillet welds. Comment : Below 30 degree angle, or where additional strength is required, prepare flanges to allow single sided butt weld.
	Problem : How should the weld at the end of a long haunch be treated ? Comment : This weld can be treated as a part penetration butt weld and is usually sized to allow half the beam flange force to be transmitted.

8. CORROSION PROTECTION

CORROSION PROTECTION

Introduction

The purpose of this chapter is to guide the designer in specifying appropriate corrosion protection systems for structural steelwork. It will point out situations in which uneconomic specifications are commonly used. There is scope for economy at all stages of the construction process from design through fabrication and coating application to erection on site.

The format of this guide follows stages of the design process. At each stage, particular issues which influence the parties involved, i.e. designer or fabricator, are highlighted and specific guidance given.

It has been assumed that those using the guide are involved in the construction of general building structures rather than special structures, such as bridges or process plant. The principles outlined here can be readily adapted to a wide range of buildings from simple portal framed warehouses through to complex modern office developments.

Although the guide generally follows stages of the construction process, it is initially necessary to consider two additional areas; the methods available for protection and a definition of environments. Thereafter the construction process is more closely followed.

Finally tabulated guidance is given to the protection specifications for each environment identified, together with indicative costs of such systems.

Corrosion Prevention By Coatings

It is often thought that coatings prevent corrosion by acting as a physical barrier between the steel and its environment. Thus the fuels required in the corrosion reactions, oxygen and water, are prevented from reaching the steel surface. Even with modern coating formulations this ideal is not approached.

It has been shown that for unpigmented polymers the diffusion rate of both oxygen and water is sufficient to sustain corrosion at rates equivalent to uncoated steel. Yet, even under these conditions corrosion of coated steel is very slow.

The corrosion of steel can be represented by the following chemical reactions:

$4\,Fe \rightarrow 4\,Fe^{2+} + 8e^-$	Anodic
$2O_2 + 4H_2O + 8e^- \bullet\ 8OH^-$	Cathodic
$4\,Fe + 3O_2 + 2H_2O \rightarrow 2\,Fe_2O_3.H_2O$	Overall

The anodic and cathodic reactions are simultaneous but occur at microscopically different locations on the surface. The reactions can only proceed if current flows from anode to cathode, i.e.. elections can move. The principal role of coatings is to prevent this current flow between different sites. This is achieved because the coating represents a high electrical resistance, making electron transfer difficult.

Coatings can also be pigmented to provide additional means of preventing corrosion by directly affecting either the anodic or cathodic reactions. In particular some primers are pigmented with either sacrificial metals, such as zinc or inhibitors such as zinc phosphate. Inhibitors act to stop the cathodic reaction and sacrificial metals act to stop the anodic reaction.

The cathodic reaction can be retarded by decreasing the rate at which oxygen and water diffuse through the coating. This can be achieved by using materials with high cross linking density which reduces microporosity. This can be further enhanced by inert pigments, such as micaceous iron oxide (MIO), which block the pores, and make the path through the film more tortuous.

The third way in which corrosion can be prevented, or at least retarded, is by polarisation. To some degree this is assisted by the previously mentioned inhibitors. However, it can be understood on a more fundamental level. Coatings are relatively permeable to molecules, such as oxygen and water, but relatively impermeable to ions, such as Fe^{2+} or OH^-. Therefore, any corrosion products formed cannot easily move away from the surface. They therefore become concentrated at the interface, stifling further reactions.

From the brief discussion above, it should be clear that the prevention of corrosion by coating involves complex processes. However, an understanding of the points given above should allow an appreciation of why different materials are used to fulfil different functions.

Coating Formulations And Systems

An appreciation of the important constituents of coatings and the components within a given system, should enable the designer to understand more clearly how coating systems are specified. It will also assist in selecting an appropriate system from the apparently bewildering range of products available.

Coating Formulations

Coatings are made up of pigments dispersed in a solution of binding medium and solvent. The binding medium, or binder, is usually organic and determines the basic physical and chemical properties of the coating. Examples of binders are alkyds, epoxies and polyurethane, although there are many others.

Pigments are added to the binder for a variety of reasons depending on the coatings function. For finishes pigments are added to produce the required colour or decorative appearance. In primers the pigments may be more functional and act as corrosion inhibitors. In barriers, inert pigments, for example micaceous iron oxide, are added to reduce the coatings permeability.

The solvent is added to ease manufacture, application and control the coating viscosity. On application this solvent evaporates and is lost; it therefore has no role in preventing corrosion.

In addition to these basic components the manufacturer will probably also include a range of additives to improve storage, application and improve basic mechanical properties.

Coating Classifications

Coatings can be conveniently grouped into four classifications depending on how the wet applied coating cures. Curing can be achieved by:

(a) Air Drying: the coating film is formed by the oxidation of the liquid oils. These are usually one component materials such as the oil/alkyd type products.

(b) Polymerisation: the film is formed by the cross linking of organic molecules. Typical examples are epoxies and polyurethane. These are usually two pack materials comprising resin and curing agent or hardener. They can also be single pack materials which require atmospheric moisture to initiate the curing process.

(c) Solvent release: the coating is formed by the evaporation of solvent leaving an inert film. Usually these are single pack materials such as chlorinated rubbers.

(d) Water evaporation: water evaporates allowing a polymer emulsion to coalesce.

Coating Systems

For corrosion protection of structural steelwork, traditional coating systems are built up from a number of components. Each component within the system has a specific function in relation to corrosion prevention. These functional components can be conveniently placed in three groups; primers, barriers and finishes.

Primers

These are the first coats applied to correctly prepared steel. They are in intimate contact with the surface and are therefore the only component that can have a direct influence on the corrosion reactions. Primers must therefore show good adhesion to the substrate and provide a foundation for subsequent coats.

Primers can be pigmented with active inhibitor to prevent corrosion. The most common pigments found in primers, in the UK, are zinc phosphate and metallic zinc. These pigments can be carried in a wide range of binding media.

Metallic zinc is used in the so called "zinc rich primers", which are most commonly based on epoxy resins. In order to be capable of controlling the anodic reaction these coatings need to contain a high level of zinc in the film. The exact percentage will vary between manufacturers and the best approach is to specify that these primers comply with BS 4652.

The function of zinc rich primers, and the need for high zinc contents is often misunderstood. When first applied these coatings are microporous and on exposure the zinc starts to corrode, protecting the steel. The pores then become blocked with zinc corrosion products and the film acts as a barrier. If the film is subsequently damaged, zinc will again protect the substrate by sacrificially corroding. This can only occur if the zinc is electrically continuous with itself and the substrate. This is the main reason for insisting on high zinc contents.

For inhibitive primers, the inhibitor content is also of great importance. The most common inhibitor now used is zinc phosphate and this can be carried in a wide variety of binding media. The most common being epoxy and oil/alkyd. The way in which inhibitors work is complex and involves a variety of mechanisms that are outside the current scope. However, an important point to note is that all inhibitors must be sparingly water soluble, to allow slow release on exposure to water. If they were insoluble they could not influence the corrosion reactions and if too soluble they would be leached from the film. Because of this solubility of the inhibitor, in a corrosive environment these primers should not be used without overcoating.

A major requirement of the fabricator is that the paint system be fast drying, to allow handling and transportation as soon as possible to free the area for following work. There are now a number of quick drying zinc-phosphate primers on the market which can be applied in coatings of 75 μm or greater. The specifier should ensure that an adequate level of inhibitor is still maintained in such primers

Many manufacturers sell products called Red Oxide primers. These products are pigmented with red iron oxide which has no inhibitive properties and is added as an inert filler. For good anti-corrosion performance these products should also contain an inhibitive pigment such as zinc phosphate.

Barrier Coats

The purpose of barrier (build) coats is to increase film thickness, impermeability and resistance of the coating. They are often pigmented with materials that assist in these aims most notably with micaceous iron oxide (MIO) although sometimes with aluminium flakes. Barriers may be based on many binding media although the most common are alkyds and epoxies.

Plate type pigments, such as MIO increase impermeability in two ways. First the plates tend to orientate parallel to the steel surface. Therefore, any water and oxygen passing through the film must go around the plates. This presents a far more tortuous route. In this sense the MIO can effectively be thought of as increasing film thickness.

Secondly, air and water can only pass through the film via pores and voids. All films will contain large numbers of micropores which permit molecular water and oxygen transport. The MIO effectively blocks these pores preventing molecular transport through the primer.

As barrier coats do not usually contain inhibitors they should not be applied directly to steelwork. If they are, a reduction in performance must be anticipated. Indeed some barriers may fail quite rapidly due to poor adhesion to the substrate.

Finishes

Finishes can be required for a variety of reasons the most obvious being for decorative appearance. However, they may also be required for chemical or slip/abrasion resistance.

Decorative finishes are pigmented to give the required colour and gloss level. In terms of corrosion protection they add little to the overall system. However as they tend to be smooth and glossy they promote water run off reducing time of wetness. Finishes may be based on many binding media; although they are normally based on single pack materials for ease of application.

Where chemical resistance is important the finish will need to be a two pack material such as an epoxy or polyurethane. The exact choice being dependent on the exposure conditions and manufacturer's advice should be sought.

For slip/abrasion resistance conventional two pack materials may be used at an increased thickness. These materials will also contain a specified weight of fine aggregate (sand) to improve slip and wear performance.

Environments

In order to select the most appropriate coating scheme for a particular structure it is first necessary to define the working environment. If this is poorly defined it will result in either under or over specification, both of which have serious consequences for performance or cost respectively.

External environments can usually be sub-divided into three categories:

(i) Rural
(ii) Urban/Polluted
(iii) Coastal

Internal environments may also be placed into three categories :

a) Controlled and dry e.g.. Air conditioned or normally heated office space.
b) Un-controlled frequently damp or wet - e.g.. Un-heated roof spaces prone to condensation, perimeter steelwork in cavities or behind vapour barrier.
c) Special considerations - e.g.. Swimming pools, kitchens, vehicle loading/access bays etc.

Within a project it is possible for all environments discussed above to be present. However, this need not mean that many variations of specification need to be applied. For smaller projects in particular the specification requirements should be rationalised. One way of effectively achieving this is to select systems which have a common factory applied primer, with limited variations of site applied coats to accommodate the changes of protection specifications for the more severe environments.

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Design

The design and detailing stage of a project can have a significant influence on avoiding corrosion problems. It is impossible to cover all the good and bad versions, but typical examples of good and bad practice are shown diagrammatically at the end of this chapter.

In determining a corrosion protection scheme at the design stage it is essential to consider how it will interact with other materials. In particular, compatibility with the fire protection system must be resolved. Failure to do so could result in major problems when steel arrives at site, not to mention the possibility of large claims.

Problems in this area often result because of unclear definitions of whether the engineer or architect is responsible for fire protection. Having resolved such responsibility, the type of fire protection needs to be established before the corrosion protection is specified. It is important to remember that only concrete encasement provides both corrosion and fire protection. All other methods of fire protection require additional corrosion protection, as for the non fire protected environment.

In controlled internal environments there should be no need to provide corrosion protection if using either cementitious spray or dry lining. If the steel is exposed and requires a decorative finish intumescent paints might be used. In this case a primer will be required to promote adhesion. Ideally this should be one manufactured by the intumescent supplier, if not then an epoxy primer should be specified.

In external environments the fire protection could be either cementitious spray or intumescent paint. In both cases the corrosion protection should be based on epoxies and should be the full scheme minus the decorative finishes. The coating needs to be an epoxy to avoid problems with either the alkaline cementitious spray or the powerful solvents found in intumescents.

Another issue that sometimes causes confusion and leads to unnecessary coating specification is composite construction with through-deck stud welding. The top flanges of composite beams require to be left un-painted to allow for the site through-deck stud welding operations. painting the top flange where exposed by the decking troughs after the decking is installed is difficult and probably an impossible task to perform effectively. This method of construction should only be used where corrosion protection is not required.

Specification

As with any other specification those for surface preparation and coating should be clear, and concise. Poorly prepared, confusing and unwieldy specifications do nothing for the overall smooth running of a project.

Coating schemes are best presented in a tabular form which indicates the required preparation, number and type of coats, coating thickness and where each coat should be applied. Example tabulation is given at the end of this chapter.

Unwieldy specifications can be avoided by stating that all coating operations should be carried out in accordance with the manufacturers instructions. Coating manufacturers supply application data sheets which contain all the relevant information required to achieve a satisfactory coating. Paint manufacturers are often asked to guarantee the system, encompassing the application of the paint at the fabricators premises or work site. This necessitates the manufacturer visiting the point of application on a regular basis to ensure correct preparation and application methods are being adhered to. Often the manufacturer will prepare inspection reports for the Engineer, obviating the need for additional inspection by him.

There are a number of areas where specifications are often ambiguous and lead to conflict. It is therefore suggested that the following issues are clearly defined from the very outset.

(1) *Surface Preparation*: This is that level of cleanliness required at the time of coating.

(2) *Coating Thickness*: To avoid ambiguity at time of tender pricing a "target" dry film thickness (DFT) for each coat applied and an overall minimum thickness for the total scheme should be specified. A "target" DFT is defined by stating that the average of DFT readings for each coat shall equal or exceed the target value and that no DFT reading shall be less than 75% of the target thickness value. The use of wet film thickness measurements is essential for testing during application. The overall minimum DFT for the total scheme is important..

(3) *Bolted Connections*: There is no reason why bolts should be treated any differently from the members they join. However, bolts are difficult to coat and are easily overlooked. Therefore, if corrosion protection is to be provided this is best achieved using galvanised bolts which need only be overcoated if so required for decorative finish.

(4) *Single Supplier Specifications*: It is still quite common to find specifications that are based on a specific manufacturer's products, rather than a generic coating type. This can rarely be justified except in special circumstances. Simply specifying a product can prevent a fabricator from obtaining the most competitive price for a given material. Coatings are sold on a volume basis with high discounts for large volumes. These discounts can be based on the annual quantities purchased. Obviously a fabricator can obtain significant savings if one or two suppliers are used regularly as opposed to using all suppliers infrequently.

At a practical level there is a lot to be said for allowing fabricators to use what they are used to. They will then be aware of a particular products' difficulties and how to avoid them. If a fabricator has a regular supplier it is more likely that problems will be easily overcome with technical back up from the supplier.

Surface Preparation

Providing the correct surface preparation prior to coating is probably the single most important stage in achieving successful coating of steel.

The purpose of surface preparation is to improve adhesion between the steel and the primer. Adhesion is maximised when the steel is completely free of corrosion products, millscale, oil, grease and soluble salts. The most efficient method of removing millscale and rust is undoubtedly by abrasive blast cleaning. Excessive deposits of oil, grease, cutting fluids and soluble salts must be removed.

Specifications for surface preparation are often based on Swedish Standard SIS 055900, although this now is identical to British Standard BS 7079 Part A1. These standards are often the source of confusion because the specifier is unaware of the contents. BS 7079 Part A1 deals adequately with the visual assessment of surface preparation to be achieved. Surface roughness is a function of the blasting material and must be compatible with the nature of coating to be subsequently applied; some paints require a 'rounded' surface to avoid run-off from peaks, others require an 'angular' surface to ensure adequate adhesion of the coating.

Traditionally, the preferred fabrication route was fabricate, blast then coat, with the additional requirement that coating took place within four hours of cleaning. Many fabricators now prefer to blast, fabricate, coat; some may use a blast or holding primer and some may not. Clearly under these conditions the designer may have concerns that the steel will have started to corrode before it is coated. However, if conditions prevail in the shop that will initiate corrosion it will have happened within the four hour period anyway.

Environmental conditions in modern workshops are much better than in the past and in order to allow the fabricator greater flexibility and to avoid unnecessary costs of reblasting, it is more appropriate to specify that the required surface cleanliness is that which must prevail immediately prior to coating.

Application

The application of coatings are the responsibility of the fabricator or the coating subcontractor and the designer has little control over this area of the project. However, fabricators are finding it increasingly difficult to meet specification requirements and comply with recent legislation, often because the specifications are not prepared with the legislative requirements in mind. In this area the designer has considerable influence, and should therefore be aware of the fabricators problems.

The biggest single impact on coating application will probably come from the Environmental Protection Act (EPA). The long term aim of EPA is to significantly reduce the emission of solvents from coatings to the atmosphere. The objective may be achieved by:

(a) Demonstrating that volatile organic compounds (VOC) emission rates are below a threshold value. This is achieved by abatement equipment such as incineration or solvent recovery.

(b) Use VOC compliant coatings which contain high levels of solids (> 70% by volume).

At the present time it is unclear which of these two options industry will choose, although the more likely option is (b). If this is the case, designers will have to recognise that many traditional products will not be usable. However, coating manufacturers are now able to offer a range of products that are VOC compliant. These are either high solids versions of traditional products or water based materials which contain no harmful solvents. In the short term water based products are unlikely to be used in any great volume as they are largely unproven and need further development. The only immediate option therefore will be to use VOC compliant coatings; the designers should aim to specify these products.

Fabricators may also try to lessen the impact of the EPA by proposing methods which offer other practical advantages. The obvious means of reducing solvent emissions is to reduce the number of coats applied. Multi-coat schemes have always been a problem for fabricators because they slow down the throughput of steel. They also require large areas to be reserved for curing.

Manufacturers also now offer a range of single coat primer/finishes which are claimed to combine the properties of both products in one coat. The materials may be used in appropriate environments and the advice of manufacturers should be sought. Although these products are applied in a single coat, the coating thickness required is much greater than conventional primers and the increase in curing time should also be considered.

One alternative is a single coat of zinc rich primer. For functional buildings this will offer satisfactory protection on its own. As these materials cure quickly and are a single coat application the applied costs are comparable to other schemes. If zinc rich primer is used alone, future maintenance may be more onerous as there will be a need to remove zinc salts prior to overcoating because these are detrimental to intercoat adhesion.

Where decorative finishes are required these should not be specified as shop applied. Finishes are easily damaged during transport, handling and erection and this may be sufficiently extensive to necessitate an entire extra coat being site applied anyway.

Galvanizing

Galvanizing is a practical and often cost effective alternative to paint corrosion protective systems. If this form of protection is specified there are a number of design details which must be recognised to allow effective coatings to occur and, in the case of hot-dip galvanizing, practical considerations to allow the safe execution of the galvanizing process. In general terms these details require the provision of drainage and ventilation holes at corners, low points and hollow sections, preclusion of sealed volumes caused by welding and avoidance of capillary spaces which can trap the pickling acid solution. Specific details to be used and to be avoided, together with detailed advice on the nature of galvanizing and the various options which exist, can be obtained in literature from the Galvanizers Association. Geometry and weight limits on hot-dip galvanizing depend upon the specific capabilities of the galvanizer chosen, but in general terms a maximum piece weight of 10t and piece length of 18m could be accommodated at some location in the UK but with restrictions on piece widths also applying.

Scheme Selection

A simple set of corrosion protection schemes is proposed to cover the environments defined earlier. The schemes are categorised as Internal - controlled, Internal - un-controlled, Special and External are as follows:

Note : ***The following table costs include for materials and labour in the quoted figures, assuming application is site or shop applied as noted. The figures do not include for particular provisions of masked areas, stripe coats, extra-ordinary access provisions etc. The costs are quoted against a square metre area basis, but for simple conversion to cost per tonne, a section area per tonne of 15 - 25 m^2/t could be used as a reasonable figure for typical mid-range UB sections. All costs are at 1994 prices.***

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Internal: Controlled environment

No requirement for corrosion protection.	Cost NIL £/m^2

An *optional decorative* scheme may be chosen as follows:

Preparation:		Blast clean to Sa 2.5 of BS 7079 Pt A1		Cost Total £/m^2 £10.50/m^2 Includes materials and labour
Primer:	Shop Applied	Oil/resin zinc phosphate primer	40 μm min DFT	
Undercoat:	Site Applied	Normal oil/alkyd undercoat	40 μm min DFT	
Undercoat:	Site Applied	Normal oil/alkyd undercoat	40 μm min DFT	
Finish:	Site Applied	Oil/alkyd gloss paint	40 μm min DFT	

Preparation:		Blast clean to Sa 2.5 of BS 7079 Pt A1		Cost Total £3.40/m^2
Primer:	Shop Applied	Epoxy zinc phosphate	50 μm min DFT	

Internal: Un-controlled environment

A)	Cost Total £6.00/m^2
Galvanize to BS 729, 610g/m^2	

B)				Cost Total £4.40/m^2 Includes materials and
Preparation:		Blast clean to Sa 2.5 of BS 7079 Pt A1		
Primer:	Shop Applied	Epoxy zinc rich	75 μm min DFT	

C)				Cost Total £6.00/m^2 Includes materials and labour
Preparation:		Blast clean to Sa 2.5 of BS 7079 Pt A1		
Primer:	Shop Applied	Epoxy zinc phosphate	50 μm min DFT	
Barrier:	Site Applied	Epoxy micaceous iron oxide	75 μm min DFT	

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Special cases: e.g. Swimming pool , kitchen etc.

Preparation:		Blast clean to Sa 2.5 of BS 7079 Pt A1		Cost Total £12.10/m^2 Includes materials and labour
Primer:	Shop Applied	Two pack epoxy zinc phosphate	50 µm min DFT	
Barrier:	Site Applied	Two pack epoxy micaceous iron	75 µm min DFT	
Undercoat: *	Site Applied	Acrylated rubber undercoat	40 µm min DFT	
Finish: *	Site Applied	Acrylated rubber finish	25 µm min DFT	

* The finish coats may be omitted if there are no aesthetic requirements.

External:

Preparation:		Blast clean to Sa 2.5 of BS 7079 Pt A1		Cost Total £/m^2 £12.40/m^2 Includes materials and labour *£7.40/m^2 If these * omitted
Primer:	Shop Applied	Epoxy zinc rich	75 µm min DFT	
Barrier:	Site Applied	Two pack epoxy micaceous iron oxide	75 µm min DFT	
Undercoat: *	Site Applied	Silicone alkyd enamel	35µm min DFT	
Finish: *	Site Applied	Silicone alkyd enamel	35µm min DFT	

* The finish coats may be omitted if there are no aesthetic requirements.

The life to first maintenance which is achievable with this scheme depends on the environment in which it is used. The following lives should be achieved:

Environment	**Life to first maintenance**
Rural	15 years
Urban/polluted	10 to 15 years
Coastal	7 to 10 years.

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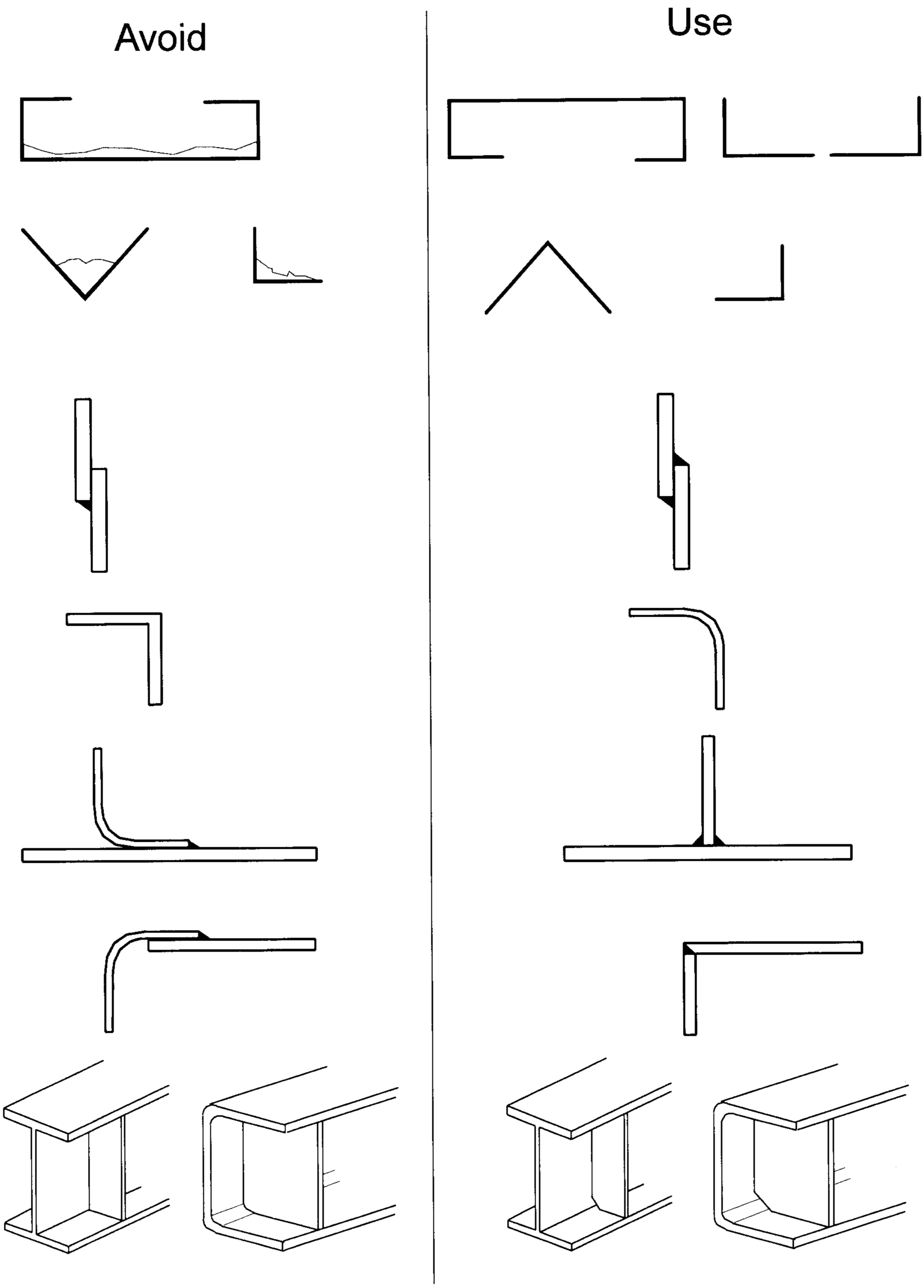

Details to be considered in an external exposed or other particularly corrosive condition

Discuss me ...

9. TRUSSES AND LATTICE GIRDERS

TRUSSES AND LATTICE GIRDERS

Introduction

This section indicates some of the features of materials and connections which should be considered by the designer and indicates where particular choices would simplify manufacture. As indicated in Section 5 of this document the provision of guidance on cost is not easy outside the context of a specific truss geometry, hence no relative cost data has been attempted. However the significance of simplified connections are equally important in truss and lattice members and the principles discussed in Section 5 still apply to these fabrications.

For medium or long spans and where conditions permit, lattice frames are often the most economical method of carrying loads. Trusses are invariably used for supporting roofs whereas lattice girders can be used for roof, floor, bridge and plant type structures.

Lattice frames come in a variety of shapes and layouts, the fundamental principle being to form triangles which are the most stable shape. A further principle is, where there is no major reversal of loads, to keep struts within the framework as short as practical as opposed to tension members which can be longer without detriment. When adopting this principle it must be borne in mind not to make the members too short and thus overcomplicate the frame.

Frames are fabricated in the factory by bolting or welding together and, if they have to be spliced for transport, by bolting and/or welding on site.

Member selection

In deciding how a frame is to be formed a number of points have to be considered :-

1) Size.

2) Choice of member.
   - a) Aesthetic considerations.
   - b) Cost of material.
   - c) Availability.
   - d) Lengths available.
   - e) Complexity of connections.
   - f) Corrosion protection.
   - g) Ease of handling in factory.
   - h) Ease of handling in transportation.
   - i) Ease of handling on site.

3) Choice of connection.

Taking these points in order :-

Size.

For ease of transport the size of frames should be kept below 5m wide and 27.4m long ; even at this size there is a premium in transport costs. More advice on this subject can be found in section 10 of this document.

Aesthetic considerations

Frames composed of structural hollow sections are generally considered more pleasing to the eye, although the use of UC chord members with RHS internals can have much the same appearance.

Cost of material

Frames formed from structural hollow sections may incur higher materials cost than those of equivalent strength open sections, refer to section 3 of this document for basic material cost information.

Availability

Most open sections are readily available but there are exceptions which require investigation before a choice is made. One such exception is the use of Design Grade 50 angles which are not normally available from stockholders.

Whilst hollow sections are available either from mills or stockholders, unless a considerable quantity of identical sections is required, hollow sections would normally be ordered from a stockholder.

Lengths available

The majority of open sections are readily available in lengths which can be easily handled by the average fabricator. Basic prices are for lengths upto 15m, with most sections available upto 18m long at a premium, upto 26m lengths are available in certain section sizes.

Hollow sections are available as standard mill lengths typically 7.5, 10 or 12m long, and in special mill lengths typically upto 14 or 15m long, dependent upon section, all lengths are supplied within a tolerance of ± 150mm. Special mill lengths are generally only supplied on loads over 10 tonnes of identical section, thus stockholder purchase is the more normal supply route. The limitations of available lengths generally means greater attention needs to be paid to member lengths during the frame development and/or greater levels of material "wastage" allowance will occur, possibly combined with increased provisions for butt welding of sections to obtain the required lengths, particularly for chord members, than would be needed for the equivalent open section truss.

Complexity of connections

The choice of member size has a considerable bearing on the complexity of connections. In principle, sections should be selected that do not require stiffening as this can be expensive to fabricate and result in distortion. In tubular construction a useful rule to adopt is for a given member cross section area requirement, provide relatively thick walled sections for chord members, and relatively thinner walled sections for internal members ; this will tend to increase the section dimensions of the internals and provide a better "footprint" where internals intersect with chords generally improving joint capacity. Internal section sizes should still be selected to be smaller in width than the face of the chord to which they will be connected, in order to simplify welding details.

Corrosion protection

In the choice of member this aspect requires great attention. It is common practice to specify double angle members separated only by gussets and washer plates making finishing coats after fabrication and subsequent maintenance extremely difficult. This form of construction should only be used where no corrosion protection is required and, even so, it can cause disruption in fabrication if finishing coats have to be applied before final bolting up.

Hollow section construction does offer advantages for corrosion protection, both in terms of reduced surface area to be covered for a given truss weight and a generally "cleaner" envelope to the final fabrication, with fewer possibilities of corrosion traps. Fabricators must be aware that some hollow sections can be supplied ready primed from the mills, and these would normally require pre-fabrication blasting to avoid contamination during welding and cutting processes.

Many fabricators would send larger trusses to sub-contract painters for corrosion protection application.

Ease of handling in factory

There are two main difficulties of handling frames within the factory :-

a) Handling of the long principal members before they are incorporated into the final frame.

b) Handling of a completed frame which has low out of plane stiffness.

In the first case the most common solution is to use a lifting beam designed to support the member frequently and in such a way that no distortion occurs.

In the second specific lifting devices need to be designed not only because the finished shape may present difficulties in lifting but also the problem of headroom needs to be addressed for deep frames. In frames made from open sections, where these are joined in the centre solely by a gusset, great care needs to be exercised as this is the point where bending occurs during lifting; this can also happen to a more limited extent in welded frames.

Ease of handling in transportation

It is common to have more limited craneage in finished goods yards than in the factory and as a consequence greater care needs to be exercised. One way of overcoming the problem is to stiffen identical or nearly identical frames by strapping them together for transportation and another is to fabricate 'toast rack' trestles and transport the frames to site vertically.

Ease of handling on site

Problems can occur on site if 'unstable' frames have to be stacked prior to erection. In this situation sufficient 'toast rack' trestles have to be provided and unloaded with the frames until they are erected.

When erecting 'flimsy' frames it is a good idea, craneage permitting, to erect at ground floor two frames braced together, then lift as a whole.

Choice of connection

In this section various types of connection are illustrated and comments added giving the advantages and disadvantages of each.

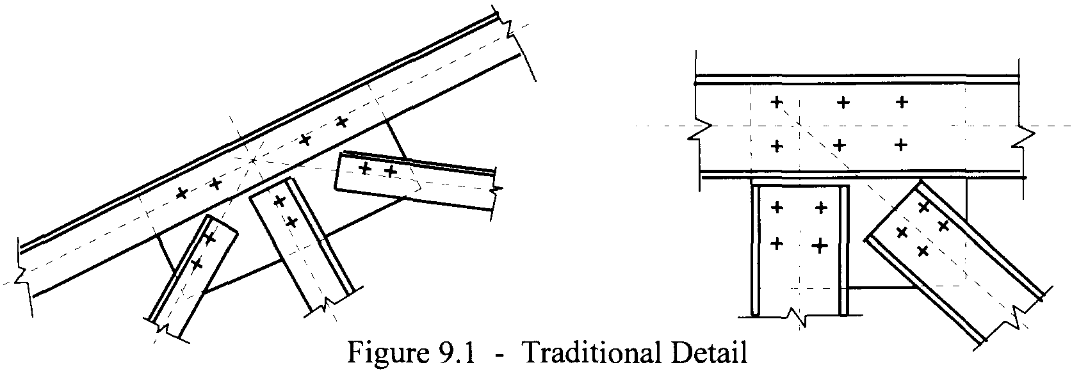

Figure 9.1 - Traditional Detail

The example illustrated above is typical of the traditional form of truss and lattice girder. When constructed with angles and gussets the material can be prepared on automatic cropping and punching machines. Subsequently, particularly with careful detailing, the truss can be assembled using unskilled labour, even introducing cambers where necessary. The frames are simple to design, dimensionally stable and members can be cut square. During assembly it is not usually necessary to turn the frame over.

This form of construction has the disadvantage of being difficult to paint if double angle members are used. In the case of large spans it can also be difficult to handle in the shop and on site due to its weakness about the minor axis of the frame.

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Alternative Details

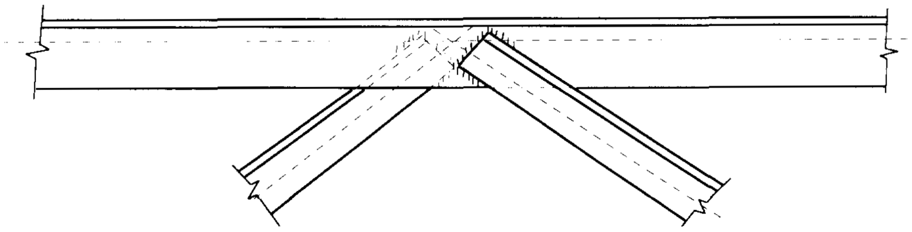

Figure 9.2 - Alternative Detail 1

1) The detail above is commonly adopted on welded trusses. The cost of splitting and straightening the Tee rafters and ties has to be added into the fabrication costs. As the apex is usually butt welded it has the advantage of being slightly stiffer than its bolted counterpart but has the disadvantage of having to be assembled in a jig and turned to fit the internals during assembly and again during final welding.

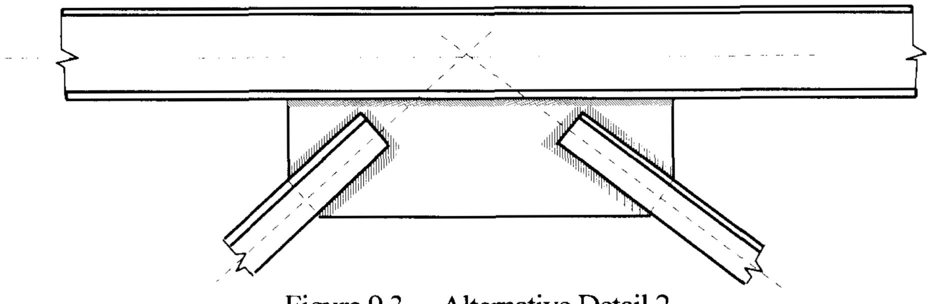

Figure 9.3 - Alternative Detail 2

2) This detail is frequently used in lattice girder construction. It has the advantage that some of the gusset to boom assembly can be carried out 'off jig' and if single angle internals are adopted may not require turning during fabrication. The welds are accessible and no difficulty should be encountered during welding. Almost invariably as the inertia of members out of plane of the truss is small, this leads to difficult handling.

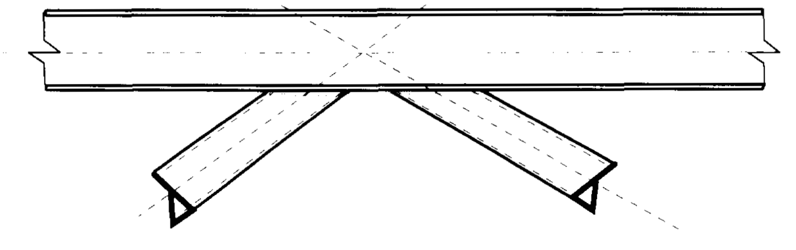

Figure 9.4 - Alternative Detail 3

3) This detail has many of the disadvantages of 2) and in addition has the problem of 'fit up'. Access for welding can be difficult, and this usually results in keeping the girder in the vertical position during welding to enable the welds to be made in the downhand position. The girder is then turned to carry out the similar operation on the other boom.

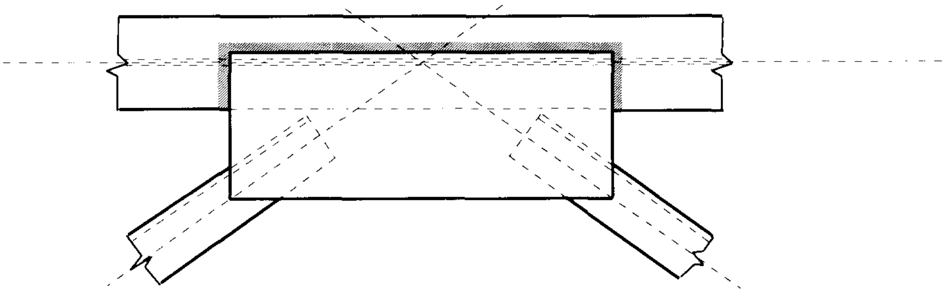

Figure 9.5 - Alternative Detail 4

4) Using the detail above gives better handling of the girder during fabrication but putting the internals on the outside of the booms is good for welding but not for aesthetics. If double angle members are used, a considerable cost has to be borne due to extended battens and the girder having to be turned during assembly and welding.

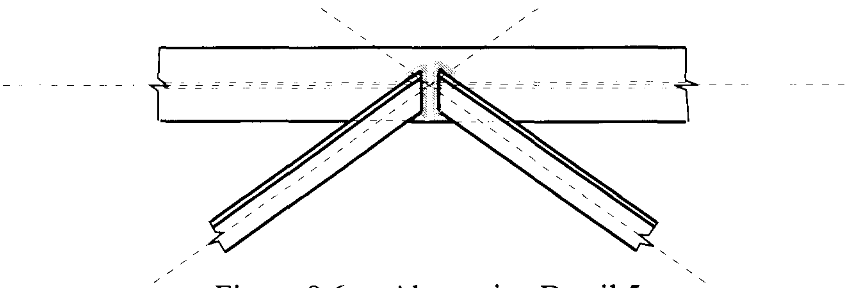

Figure 9.6 - Alternative Detail 5

5) The detail above has the advantage of 'off jig' assembly for the booms, battens are not as long as in 4) but access for welding of internals is more difficult.

6) Hollow section construction is increasingly popular for lattice work, especially where visual appearance is important. However the particular requirements for joint design and member selection have to be borne in mind.

a) Joint stiffening of hollow sections is particularly difficult; sections should always be selected such that this is un-necessary.

b) The best arrangement of internals is either fully gapped joint or if not achievable a 100% overlapped joint.

Both conditions simplify the preparation of internal member ends. The gap joint has the advantage of easier fit-up and welding access, the overlapped joint has greater strength if required.

c) Where possible use RHS sections for the booms in preference to CHS to make the end preparation of the internals simpler.

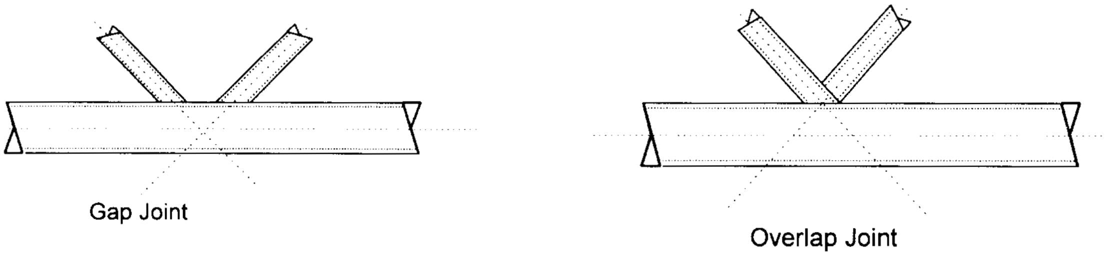

Figure 9.7 - RHS Truss Joints

10. TRANSPORTATION

TRANSPORTATION

In general the road transport of raw steel and of finished fabrication within the UK does not pose a great problem to fabricators on most structures. Simplified requirements for advance notification of Police and Department of Transport concerning long, wide or high loads are indicated in the following illustration. Transportation costs increase with each category due to both indirect administrative/planning effort costs and direct costs of vehicles used and driver plus drivers mate for larger loads. For exact rules concerning this subject reference should be made to the following regulations :

Motor vehicles (construction and use) regulations 1986
Motor vehicles (authorization of special types) general order 1979
Road traffic act 1972.

However there are some points which the designer should consider when structural members or frames become physically large:

a) The limitations of transportation are not the only limits on piece size that may have to apply. There may be smaller dimensional limits that apply to a particular fabricators workshop handling capacity, often this may be a piece weight or height limit rather than length. Similar restrictions may be imposed from offsite painting or galvanizing facility capacities.

b) The provision of temporary (or permanent) lifting points on the structure; if these are provided what provisions (if any) need to be specified for their removal and surface dressing. Positions of lifting points should allow appropriate orientation of the member during lifting with the centre of gravity of the lifted part noted as well as the piece weight. Consideration should also be given to the stresses in members during lifting and associated connection forces.

c) If a framed structure is to be erected using a number of smaller lattice frames, generally split at node points for transportation or handling requirements, the stability of the part frames needs to be considered. Any chord or internal member no longer connected to a node may need temporary support to ensure that member is not damaged during transport/erection.

Discuss me ...

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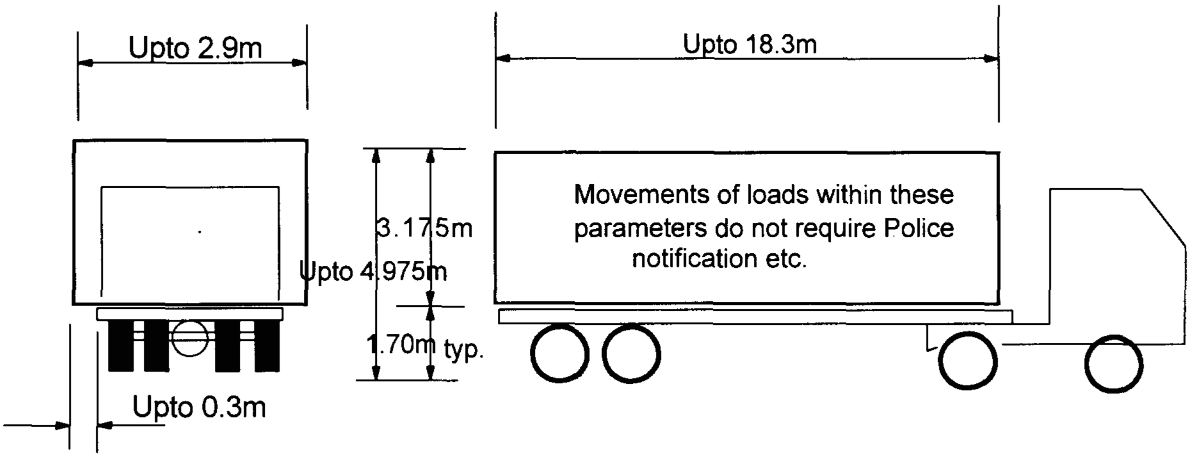

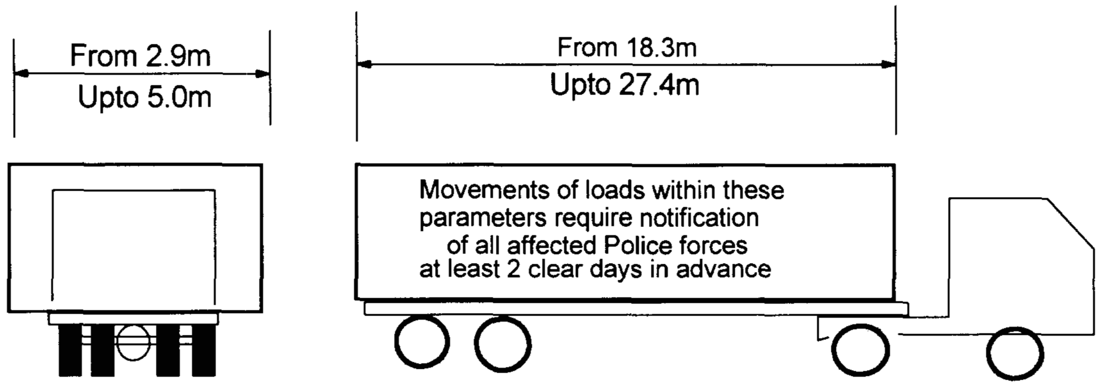

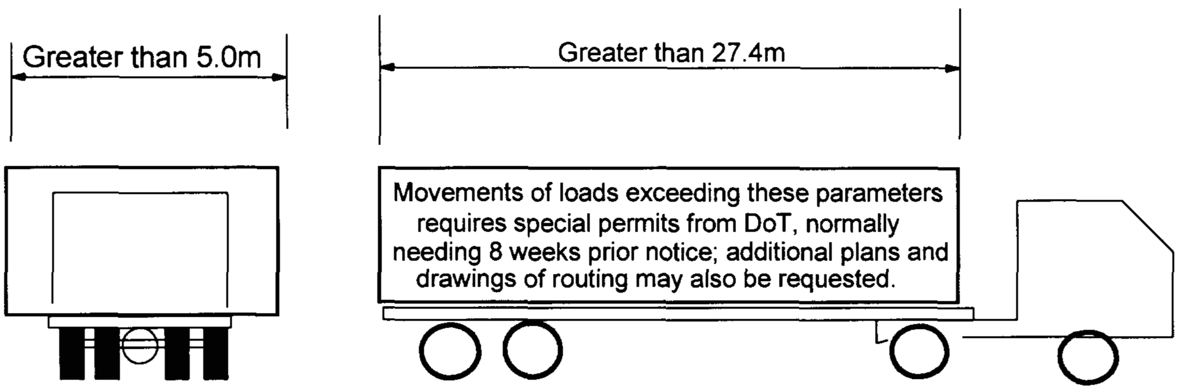

Figure 10.1 Road Transport Limitations (simplified)