SCI PUBLICATION 074

Parallel Beam Approach – A Design Guide

Parallele Träger – Ein Leitfaden für Entwurf und Berechnung

Approche en Poutres Paralleles – Un Guide de Dimensionnement

Método de la Viga Paralela – Guía de Diseño

Una Soluzione di Impalcato a travi Principali Parallele – Una Guida per la Progettazione

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FOREWORD

The parallel beam approach has been developed and used extensively by Peter Brett Associates over the last fifteen years in a variety of single and multi storey buildings.

To assist in the design of these buildings, a set of empirical guidelines and analytical methods have been developed. This document describes the approach, the empirical guidelines and the analytical methods used to date.

This publication has been prepared by Mr P Brett and Mr J Rushton of Peter Brett Associates with technical contributions from Dr G W Owens and Mr D L Mullett of The Steel Construction Institute and Ms N Molenstra of Peter Brett Associates. It is one of a series of publications being prepared by the Steel Construction Institute on the design of composite beams in buildings. Others in the series are:

Design of composite slabs and beams with steel decking.
Design of openings in the webs of composite beams.
Design of fabricated composite beams in buildings.
Design of haunched composite beams in buildings.

The design method presented in this publication is intended to be consistent with *BS 5950:* Part 1 and: Part 3.1 (the latter was in draft at the time of publication).

The following have been consulted or have commented on this publication:

Dr R M Lawson	Steel Construction Institute
Mr A R Mortlock	British Steel plc

The research and design studies leading to this publication were partially funded by British Steel plc.

Technical queries on this publication should be addressed in the first instance to Dr G W Owens and Mr D L Mullett of the Steel Construction Institute.

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SUMMARY

Parallel Beam Approach – A Design Guide

This publication presents a novel method of design using a parallel beam grillage system, in which continuity is developed in both secondary and primary beams. The secondary beams are generally designed to act compositely with the concrete slab and are made continuous by passing over primary beams; the latter are arranged in pairs and pass on either side of columns to which they are attached by shear resisting brackets.

The publication provides practical advice on preliminary sizing, constructional aspects and fabrication and erection details. Design criteria are explained in depth and a fully worked example is provided to illustrate the method of design.

Parallele Träger – Ein Leitfaden für Entwurf und Berechnung

Zusammenfassung

Diese Veröffentlichung stellt ein neues Verfahren zu Entwurf und Berechnung von durchlaufenden Trägern vor. Die Sekundärträger wirken im allgemeinen als Verbundträger mit der Betondecke zusammen und sind durchlaufend auf den Primärträgern angeordnet. Die Primärträger laufen paarweise auf beiden Seiten der Stützen vorbei und sind auf Konsolen aufgelegt.

Die Veröffentlichung vermittelt praktischen Rat, Aspekte der Konstruktion, Fertigung und Montage. Entwurfs- und Berechnungs kriterien werden ausführlich erläutert und ein Beispiel verdeutlicht die Entwurfs– und Berechnungsmethoden.

Approche en Poutres Paralleles – Un Guide de Dimensionnement

Résumé

Cette publication présente une nouvelle méthode de dimensionnement utilisant un système de grillage à poutres parallèles, dans lequel la continuité est développée tant pour les poutres primaires que pour les poutres secondaires. Les poutres secondaires sont, en général, dimensionnées en tenant compte d'un comportement composite avec la dalle de béton et sont rendues continues en passant au-dessus des poutres primaires; ces dernières sont disposées par paires et passent de chaque côté des colonnes, auxquelles elles sont attachées par des consoles résistant au cisaillement.

La publication fournit une aide pratique pour le dimensionnement préliminaire des éléments, les aspects constructifs et les détails de fabrication et de montage. Les critères de dimensionnement sont expliqués en détail et un exemple complet illustre la méthode de dimensionnement.

Método de la Viga Paralela – Guía de Diseño

Resumen

Esta publicación presenta un método nuevo de diseño usando un sistema de emparrillado con vigas paralelas. Generalmente las vigas secundarias se hacen trabajar conjuntamente con la losa de hormigón y se mantienen continuas haciendo que pasen sobre las primarias; estas últimas se organizan por parejas y pasan sobre las columnas a las que están unidas mediante correctores resistentes a corte.

La publicación suministra consejos prácticos sobre diseño preliminar aspectos constructivos y detailes de fabricación y montaje. Los criterios de diseño se explican cuidadosamente y se incluye un ejemplo completamente desarrollado para ilustrar el procedimiento de cálculo.

Una Soluzione di Impalcato a travi Principali Parallele – Una Guida per la Progettazione

Sommario

Questa pubblicazione presenta una soluzione strutturale nuova per le strutture di impalcato di edifici.

Tale soluzione adotta un sistema grigliato çon travi parallele, che consente di sviluppare la continuita' sia per le travi principali sia per quelle secondarie. Le travi secondarie, in generale collegate alla soletta in calcestruzzo e progettate come elementi composti, possono essere continue in quanto poste sopra le travi principali. Queste ultime disposte in coppie e passano ai lati delle colonne, alle quali sono collegate mediante apposite mensole dimensionate a taglio.

La pubblicazione fornisce gli elementi necessari alla soluzione dei problemi relativi al predimensionamento, agli aspetti costruttivi, alla preparazione di carpenteria e al montaggio. I criteri di progetto sono spiegati in dettaglio e viene altresi sviluppato un esempio completo allo scopo di una migliore illustrazione del metodo.

NOTATION

B	breadth of steel beam
D	overall depth of steel beam
I_g	span, second moment of area
I_s	support, second moment of area
I_c	cracked slab section, second moment of area
I_d	profile section, second moment of area
I_W	web of steel beam, second moment of area
I_Y	steel beam minor axis, second moment of area
L_t	Length over which buckling is to be checked
M_c	ultimate moment capacity of composite section
M_s	ultimate moment capacity of steel beam
M_{es}	elastic moment of resistance of a steel section
M_{pf}	moment of resistance of a steel section which corresponds with full plasticity of the flanges
M_{ps}	plastic moment of resistance of a steel section
T	thickness of flange of steel beam
W	loading in kN
b	B/2
d	steel beam web depth
l_b	bay width
l_s	nominal rib span, between column centrelines.
m	modular ratio
p_y	design strength of steel
t	steel beam web thickness
β	ratio of smaller to larger end moment
δ	deflection from restraint force
λ	minor axis slenderness ratio
λ_{LT}	equivalent slenderness

1. INTRODUCTION

We are in an information technology era. The effect of this is to require industrial and commercial buildings to have great flexibility in layout of accommodation, in communications and services. The speed of technological development and of information processing has led to a quickening of commercial life and a requirement for the construction industry to accelerate the translation of design into real buildings. This has encouraged the development of 'shell and core buildings', having flexibility in internal layout and services, with less internal columns, and having 'raised' floors for ease of cabling. These buildings are constructed by so called 'fast track' methods of construction. Steel frames have become the favoured form of construction.

Currently (1990), the basic cost of steel is approximately 35% of the total cost of each tonne of fabricated and erected steel. Steel designers have for many years sought to reduce the weight of steel in a given frame, but the same effort has not been given to reducing, through design, the fabrication and erection costs, some 40% of the total.

This guide describes a new approach to steel framing that makes use of the latest developments of composite and continuous construction to provide an economic solution for industrial and commercial structures. It is particularly advantageous for buildings with high service contents.

2. THE AIMS OF THE PARALLEL BEAM APPROACH

The aims of the parallel beam approach (P.B.A.) are:

- to reduce fabrication and erection complexities by reducing the total number of members in a steel frame;
- to reduce the weight of the steel beams by use of continuity;
- to reduce the complexities and costs that occur at intersections between structural members and between structural members and services.

The successful achievement of these aims produces a steel frame that can be constructed quickly, easily and cheaply. Most importantly, the resulting building has great flexibility of services and planning, allowing speedy erection of the frame and reducing the overall building cost.

A typical general arrangement of this method of framing is shown in Figure 1. To avoid clashes between services and/or structure, it has two parallel planning zones, one above the other. The services are then arranged 'parallel' to the structural members, permitting a high degree of servicing within the structural depth.

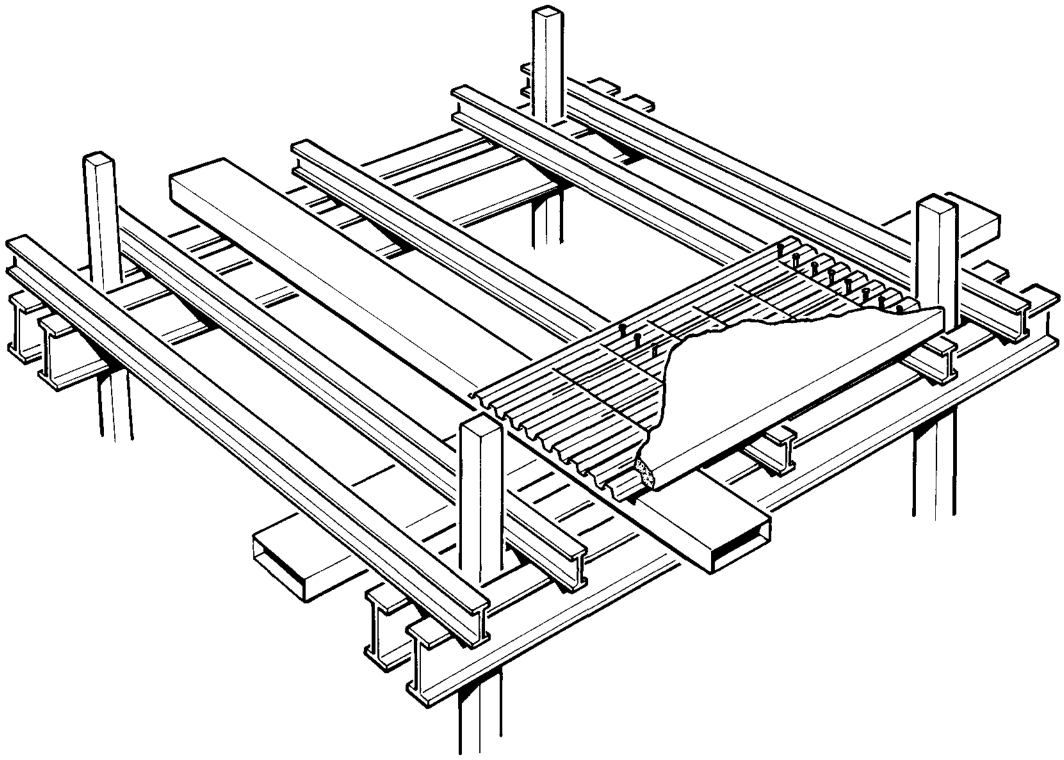


Figure 1 *Parallel grillage system showing service zones*

Discuss me ...

3. SKELETAL FRAMING

3.1 Conventional approach

In a three dimensional orthogonally framed structure, conventionally, beams in the X and Y directions (ie. along horizontal axes) are in the same XY plane (i.e. at the same level) and are supported by columns in the Z direction. The beams in the X or Y direction are generally located at centres which are a function of the Y and X dimensions of the column grid. The location of beams is therefore dictated to a large degree by the column layout.

3.2 Parallel beam approach

The parallel beam approach to framing also requires members in the X, Y and Z direction, but the beams in the X and Y direction are displaced in the Z direction relative to each other (i.e. not on the same level). With a beam in one direction passing over the beam in the other direction, beam intersections in the same plane are therefore avoided, which greatly simplifies the connections between the members (Figure 2).

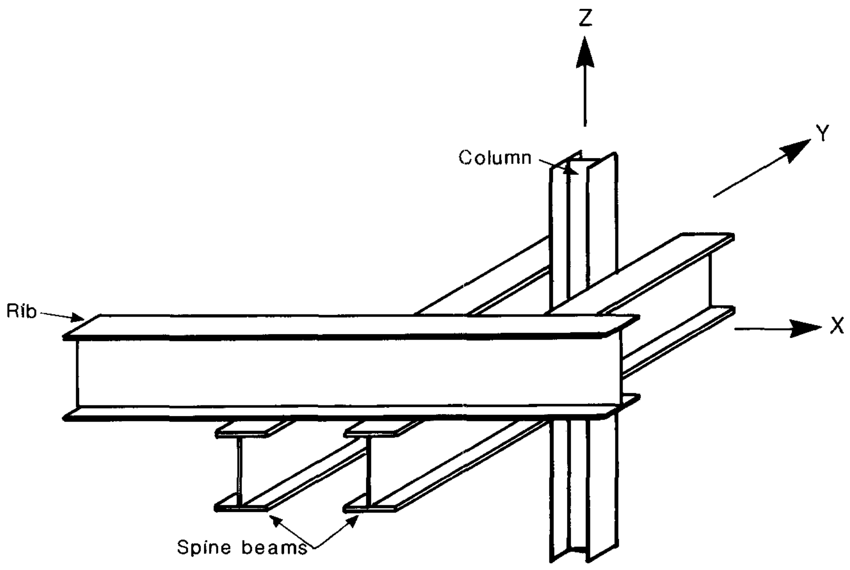


Figure 2 *Displacement of rib, spine beams and column to simplify connections*

3.2.1 Location of spine beams

The spine beams are displaced laterally so that they pass beside the columns thus avoiding an intersection with the column. They do not connect to the floor slab and are therefore designed non-compositely.

Internally, twin parallel beams are normally used whereas, externally, a single beam only is used. Spine beams are displaced either side of the column with 20–40 mm gaps between the face of the column and edge of the spine flange. The spine beams are connected via brackets to the columns.

Ideally, the distance between the inner faces of the webs of the twin spines should be equal to the overall depth of a standard Universal Beam Section such as 533× 210 UB. This allows diaphragms to be cut from the UB section and bolted between the spines, directly under the ribs located either side of the column as shown in Figure 3. Diaphragms may also be necessary between ribs at mid-span to stabilise the top flange under sagging moments.

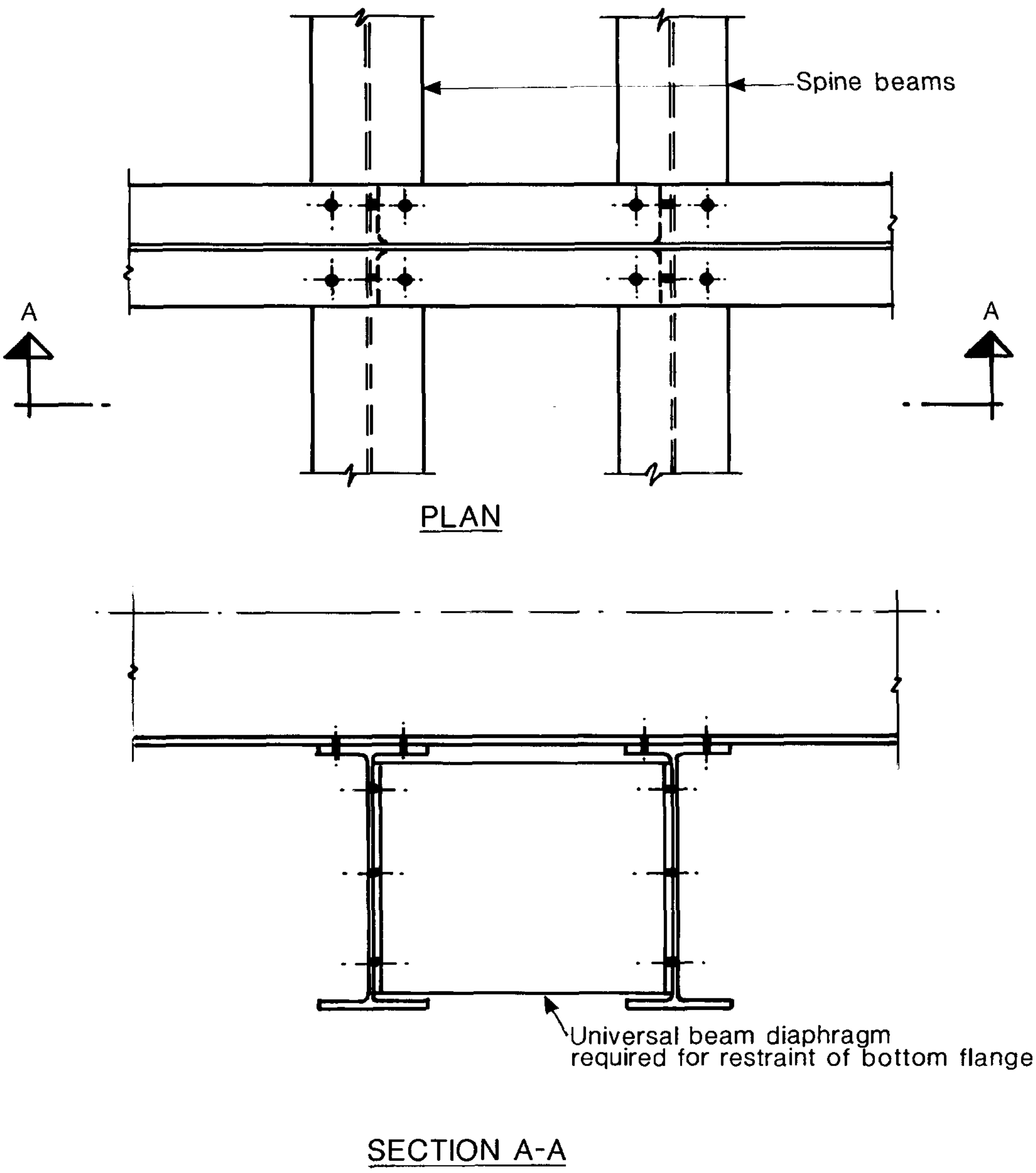


Figure 3 *Junction of rib and spine beams, showing diaphragm to stabilise the latter*

The spine beams are therefore braced together at the column and at an appropriate distance either side of the column to provide lateral and torsional restraint to the hogging moment region of the spine. The top flanges of the spines are braced together and held in position by bolting to the bottom flange of the ribs.

The structural efficiency of the spine beams is improved by making them continuous.

3.2.2 Location of ribs

Generally, it is preferable for the ribs to be of the same section, even when spans and/or loading vary. This rationalisation simplifies vertical setting out and detailing. It also minimises price/tonne and delivery times.

Whilst it is necessary for the spine beams to be located adjacent to the columns, the beams in the other direction (ribs) are displaced laterally to miss the column, as shown in Figures 1 and 2. Their lateral disposition is determined by economics or the planning requirements of the floor and not necessarily the column grid. Their spacing should be such that propping of the deck is not required.

By avoiding intersections, beams can now be more than one span in length without recourse to complex jointing, dependent only on the length that can be obtained and economically handled. Thus beams no longer need notching or end plating.

These rib beams are connected to the floor slab and are designed compositely. The structural efficiency is also enhanced by developing continuity.

3.2.3 Cantilevers

Cantilevered spine or rib beams are simply achieved by continuing the beam beyond the supporting column or spine, thus avoiding complex moment connections normally required in conventional construction (see Figure 8).

4. DETAILED ASPECTS OF PARALLEL BEAM FRAMING

4.1 Beams

As a general rule, it is better to let the composite and more lightly loaded rib beams span the greater distance with the more heavily loaded spine beams spanning the shorter distance. However, this arrangement may need to be reversed for very long span ribs if propping is to be avoided or to suit layout of services (see Section 4.12).

4.2 Column orientation

The major axes (XX) of the columns are normally aligned parallel to the spine beams, as shown in Figure 2. Thus any beam rotation at the supports, due to differential loading on adjacent spans, induces weak axis bending in the column. Column moments and shear forces on the supporting brackets are thereby minimized.

Where single spine beams are used around the perimeter of the building, as shown in Figure 4, the same orientation ensures that the column moment from the eccentric beam support is applied about the former's major axis.

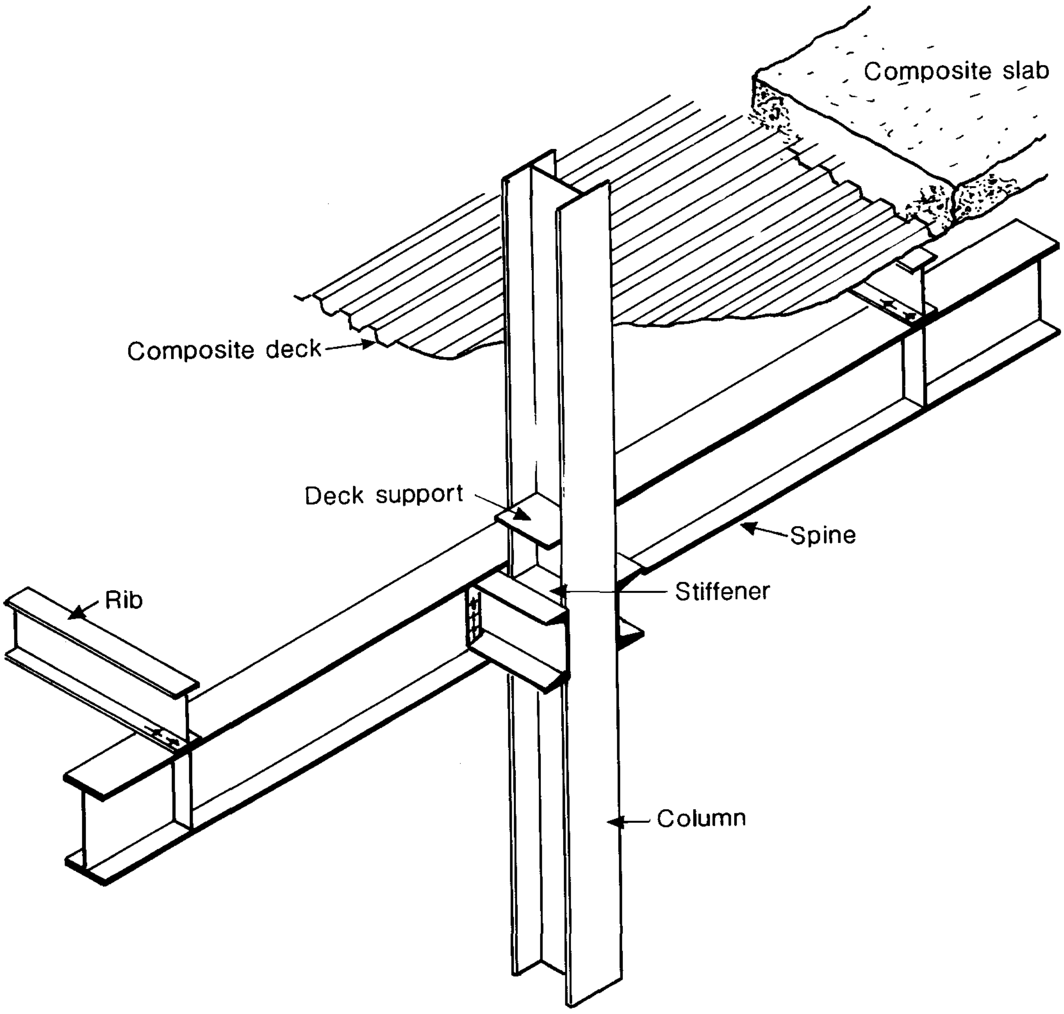


Figure 4 *Single spine beam at perimeter of building*

This orientation also produces the most favourable slenderness ratio for the column. However, in certain circumstances, for example where services are required to pass down between the column web and the spine beam, it may be necessary to orientate the columns so that their minor axes are parallel to the spine beams. It should be appreciated, however, that this orientation will attract larger column moments possibly requiring stronger support brackets. There will also be a reduction in the degree of column repetition.

4.3 Connections

Connections between the ribs and the spine beams are made by bolting the bottom flange of the upper beam to the top flange of the lower as shown in Figures 2 and 4. Connections between beam and column are made by bolting through the beam web to brackets welded to the columns, as shown for a single spine beam in Figure 4. (For a double spine beam the brackets are symmetric about the column major axis.)

4.4 Differential deflections

The distance between an external edge beam and the first internal rib will be half the normal rib spacing because the rib beams are displaced to miss the columns as indicated in Figures 1 and 2. This rib (assuming all ribs are of similar section) will not have the same magnitude of deflection as an internal rib. The transition from no deflection adjacent to an external column, to full deflection at the middle of a long span rib beam will be eased thereby.

In some situations it may be necessary to further ease the transition by the introduction of semi-flexible structural members between the external short span edge beams and the internal long span ribs. This increases the length over which the change in deflection occurs, thus decreasing the rate of change of slope.

These semi-flexible members are often in the form of 'goal posts' as shown in Figure 5. They allow use of the space between beams for services while providing lateral and torsional restraint to the beams during erection. They will also provide stability during floor construction when the concrete is poured but not hardened.

4.5 Length of beams

The repetitive use of members of the same size, the need to achieve structural continuity, and the constraints of the column grid determine the length of the rib and spine beams. For most efficient use, the composite members are made continuous over two spans, but occasionally three spans may be necessary when odd numbers of bays occur. In three span rib beams, the centre span is often subject to hogging over its length under pattern loads with the central span unloaded. Structural efficiency of three span beams can be enhanced by making the internal span longer than the outer spans.

In determining the length of beam to be used, consideration must also be given to the transportation, fabrication and erection processes. The beam may require turning on its side, lifting and handling in restricted spaces. There are special restrictions on transportation of beams over 27.4 m long. Plastic sections are anyway a requirement of P.B.A. as currently conceived; these have the advantage of being easier to handle. As a guide, the minimum flange width should be approximately 1/125 of the length of the beam to be handled.

By arrangement with British Steel, some sections can be provided up to 26.5 m long. Normally the basic price of steel applies to lots over 20 tons for lengths of 6 m to 12 m. The premium for maximum length sections is approximately £10/tonne.

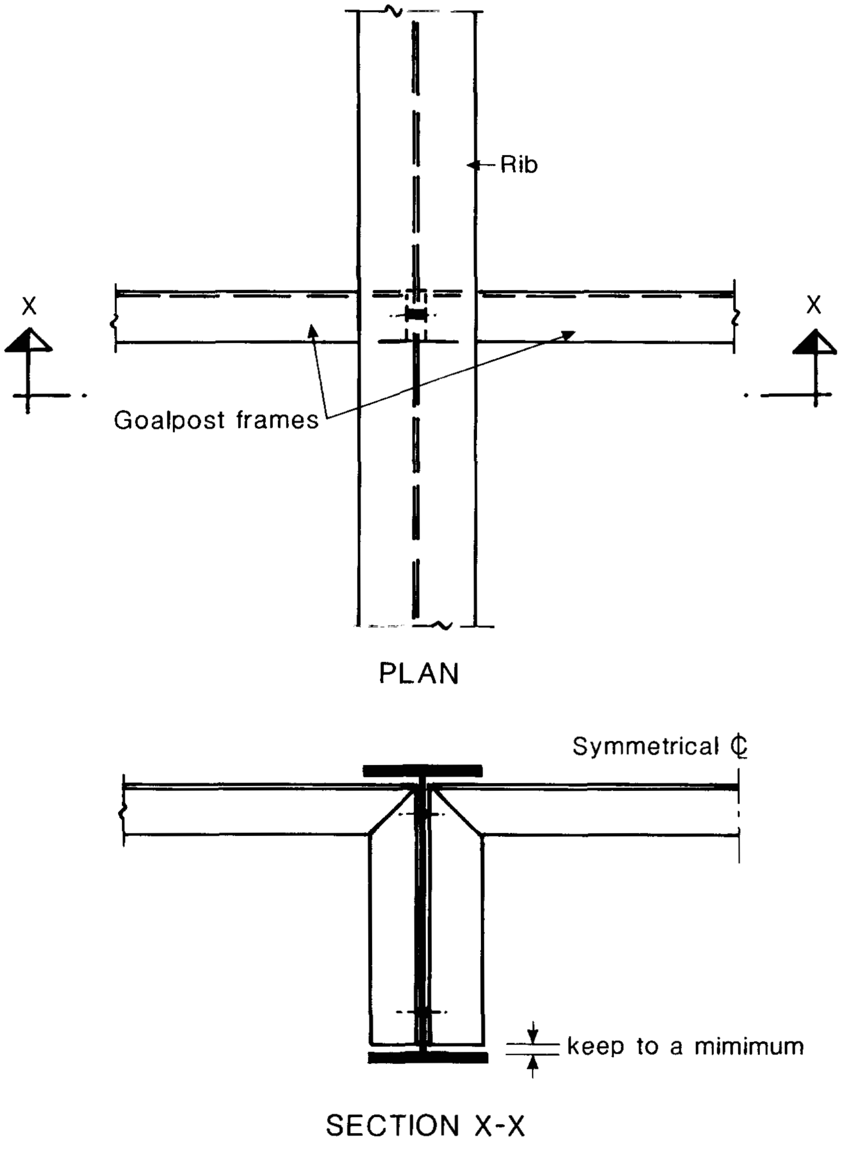


Figure 5 *'Goalpost' restraints*

From experience, the maximum span for a three-span, composite rib is approximately 6 m, i.e. a total length of 18 m, and for a three-span spine beam is approximately 8 m, a total of 24 m. For two span spines and ribs, maxima are about 12 m spans. Longer span beams may need jointing. This is normally achieved by full strength butt welding on site, the weld being located over a spine beam. Site welding is discussed in Section 4.10. As an alternative, bolted splices at positions of contraflexure can be used to achieve continuity in the design of the beam.

4.6 Construction aspects

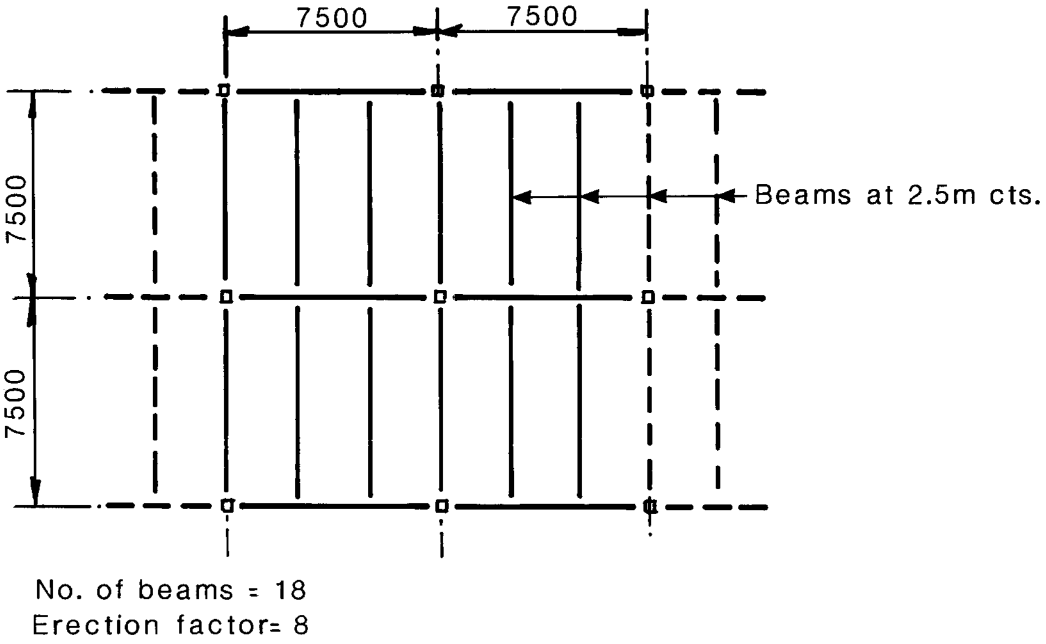
The time taken to hoist a 15 m long beam is normally the same as for a 7.5 m beam. On large sites 'hook time' may constitute the critical path for the construction programme. The parallel beam arrangement can lead to savings in construction time.

As a way of quantifying erection efficiency, an erection factor has been introduced, defined as:

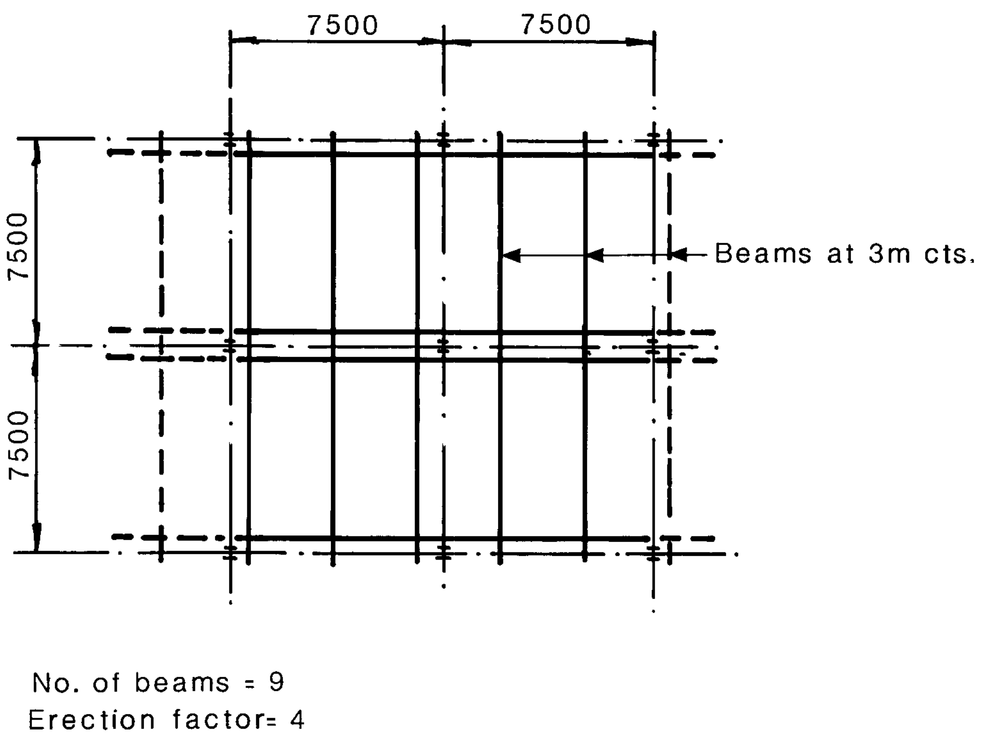
$$\text{erection factor} = \frac{\text{number of pieces of steel} \times 100}{\text{floor area in metres}^2}$$

Figure 6(a) shows the traditional layout of part of a floor to a multi-storey building having a 7.5 m square grid. The positions of the primary and secondary beams are pre-determined by the column grid.

Figure 6(b) shows the alternative parallel beam layout which has ribs at wider spacing but half the number of beams, albeit twice as long. Additionally, accumulative tolerance problems in multiple lengths of beams are reduced.



(a) *Traditional construction*



(b) *Parallel beam approach*

Figure 6 *Examples of beam layouts*

4.7 Fabrication

End plates and beam notches are no longer required; beam lengths now relate to grid centrelines, as opposed to the distance between the faces of column or beams. Beam lengths are therefore the same even though the supporting column or beam section size may have changed. Fabrication is simplified and repetition of members increased. The external and internal spines may often have the same section, thus increasing repetition.

4.8 Erection

With a large reduction in the number of lifts and increased repetition, combined with a system in which beams are 'landed on' rather than suspended between supports, erection is much simpler and faster. Alignment of bolt holes can be more safely achieved.

4.9 Services

In conventional framing a service zone is provided under the steelwork. However, in the parallel beam approach services can be dealt with in a similar manner to the beams by being split into two orthogonal zones. Thus in one direction the services are located parallel to and in the same zone as the rib beams, and in the other, they are parallel to and in the same zone as the spines. By this arrangement the whole of the structural depth is available for service runs and clashes between services and/or structure can be avoided. This is illustrated in Figure 1.

Considering the overall depth of the floor zone, buildings framed in this way generally have the same or less depth overall than conventional framed construction.

4.10 Site welding

Where long span continuous beams are required, it may be necessary to consider the need for site connections. This can be conveniently done by site welding. Provided the welding is properly specified and tested, experience has shown that site welding can be carried out to the same standard as shop welding and frequently the standard is higher. Precise details of the welding procedures and preparation should be agreed by the Engineer with the Main Contractor, Steelwork Fabricator, Welding Sub-contractor and the Testing Authority prior to commencement of fabrication. Test pieces of the actual sections and preparation should be carried out and tested for compliance with the specification. Any necessary revisions to the procedure and preparation may then be incorporated and retested. See Appendix A for typical specification and welding details.

Currently (1990), the cost of full strength butt welding for say 533×210 UB carried out on site is of the order of £150.

The actual cost is obviously dependent on number of welds carried out, access, etc. To date, site welds have been successfully carried out on sections up to 914×305 UB.

4.11 Lateral stability of frame and beams

Lateral stability of the frame in both the horizontal (X and Y) directions is normally provided by the floor beams and slab transmitting the horizontal shear forces to vertical bracing or stiff cores or shear walls. Alternatively, the spine beams and columns may be considered as multi-bay portals to resist the horizontal load in the

Y direction. In this case, vertical bracing is required for the plane parallel to the rib beams, i.e. in the X direction.

Ideally, the structure providing the permanent lateral stability should also provide the temporary stability during construction. In composite construction the insitu floor slab is not able to transmit in plane forces until the concrete has reached the required strength.

In the temporary condition, it is important to ensure the stability of the structure under the weight of wet concrete and other construction loads, and also wind forces. The steel decking should be properly fixed to the beams in order for it to act as a shear diaphragm. Where shear forces in the diaphragm are high, seam fasteners should also be used. (These have the added benefit of preventing differential displacement between sheets during concreting.) Where two edges of a sheet occur over a beam, the studs should be through deck, welded in a staggered pattern to ensure adequate connection of both edges.

Pattern loading of wet concrete should be considered to determine the length of the adjacent beam subject to negative bending. Restraint can be provided to the top and bottom flanges of ribs by the use of goal posts as shown in Figure 5. Temporary vertical bracing may also be required until the concrete slab has sufficient strength to span between the positions of permanent vertical restraint.

4.12 Propping during construction

The composite rib beams should be checked for strength and deflection in the non-composite condition when subject to the wet weight of concrete and the construction loading (taken as $0.5 kN/m^2$). From experience, it has been found necessary to prop composite beams supporting 130 mm thick lightweight concrete slabs for spans in excess of 9 m. There is of course the option of providing heavier and/or deeper sections if it is important to avoid propping during construction.

If propping is inconvenient to the construction process, the framing may be arranged so that the composite rib beams span less than 9 m, with spine beams having larger spans. This arrangement will save propping and rib costs at the expense of additional spine steel. It should be remembered that the deflections of slab, ribs and spine beams are accumulative.

Discuss me ...

5. PRACTICAL EXAMPLES

5.1 Example of P.B.A. framing no. 1

A part of the floor plan of a three storey office block with a 6 m square grid of columns is shown in Figure 7; it also indicates a conventional simply supported steel beam layout, with 93 beams required to frame one wing of the floor plan. If the internal columns are moved to a 9 m×6 m grid, 87 beams are required.

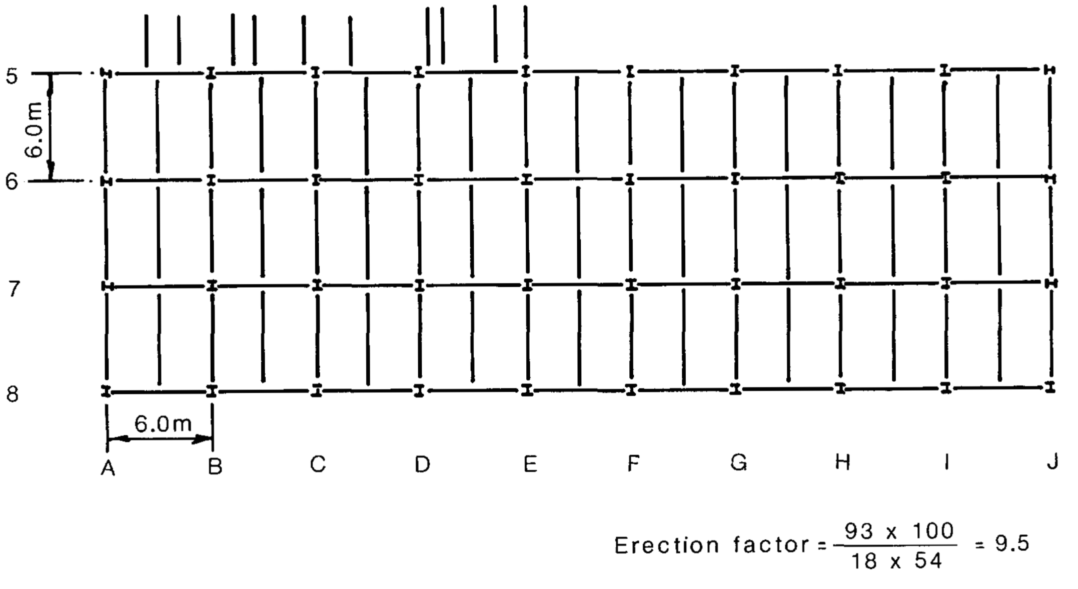


Figure 7 *Traditional construction for 6 metre square grid*

Figure 8 indicates the P.B.A. layout adopted. The 9 m×6 m internal grid has been used and 42 beams are required. Further points of interest relating to the adopted scheme are:

(a) On grids A and J conventional simply supported beams have been used between the columns.

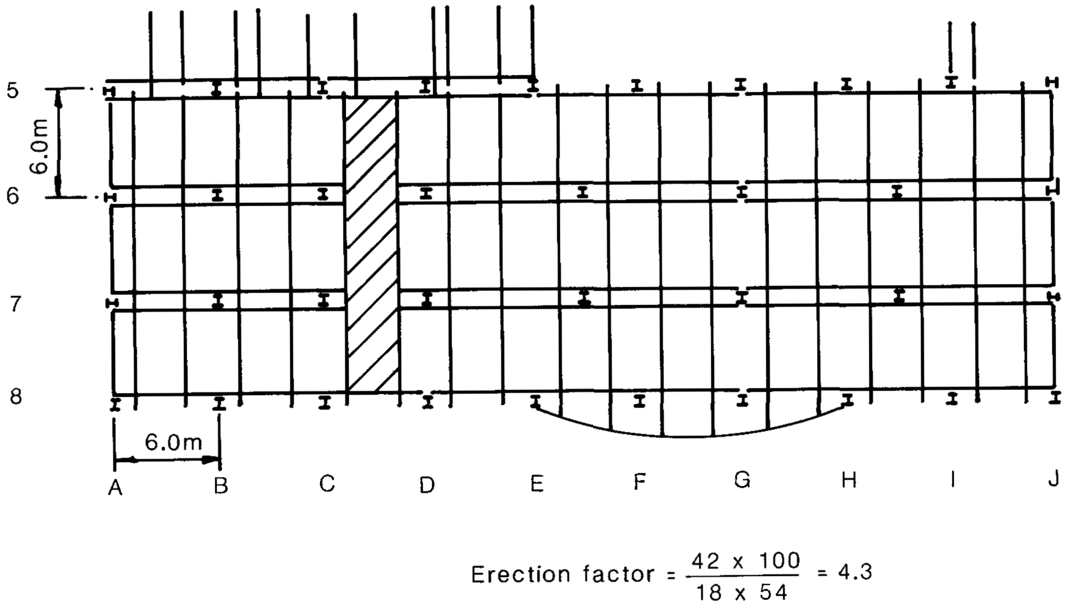


Figure 8 *P.B.A. for a 9m × 6m grid*

(b) The projecting curved bay on grid 8 was added to give interest to the facade. The framing to this feature was simply achieved by continuing the rib beams beyond their supporting spine beam, as cantilevers of varying lengths.

(c) The main plant rooms for the building are located on the roof with service ducts dropping vertically into the hatched area indicated between grids C and D. The main manifold ducts then run in the hatched area parallel to grids C and D. The distributor ducts from the manifold run the length of the building parallel to and in the same zone as the spine beams. Secondary distribution is parallel and in the same zone as in the ribs.

The spine beams span 9 m between columns. To facilitate the maximum size of manifold duct, the spines terminate as cantilevers adjacent to the hatched area. In the hatched area, the whole of the depth from soffit of slab to top of ceiling is thus available for services. Reorientating the beam spans through 90° would result in a slightly cheaper steel structure, but would not facilitate the service installation. The beams in this scheme are approximately 18 m to 21 m long.

5.2 Example of P.B.A. framing no. 2

This example, illustrated in Figure 9 has a 9 m×9 m square grid. The ribs are designed to be unpropped during construction. Typical hand calculations for the framing members of this example are given in the Worked Example (Appendix D). A comparison with a computer analysis of a two bay square part of the floor structure is presented in Section 6.7.

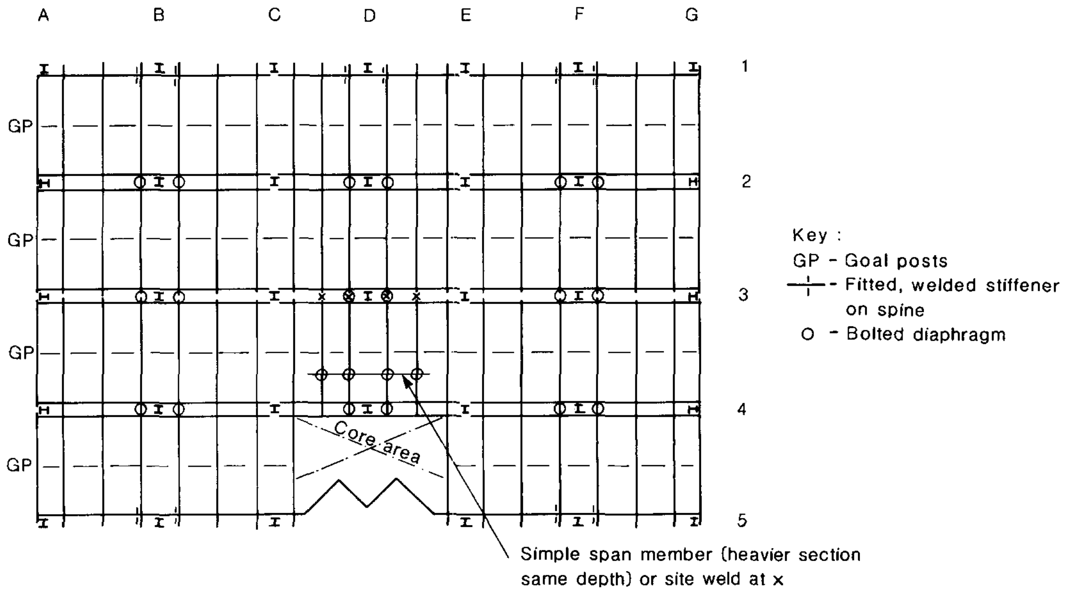


Figure 9 *P.B.A. for a 9m × 9m grid*

5.3 P.B.A. and precast units

As an alternative form of frame construction, parallel spine beams and precast floor slabs may be used to produce very economic flat soffit floors requiring very little fire protection to the steelwork. This is a variety of construction known as a 'slim' floor.

As shown in Figure 10, the columns and spine beams are arranged using universal column sections for the spine beams continuous over two spans to achieve minimum depth construction. There are no rib beams. Tie members, for stability during erection, are connected on the column centre lines in the same plane as the spine beams.

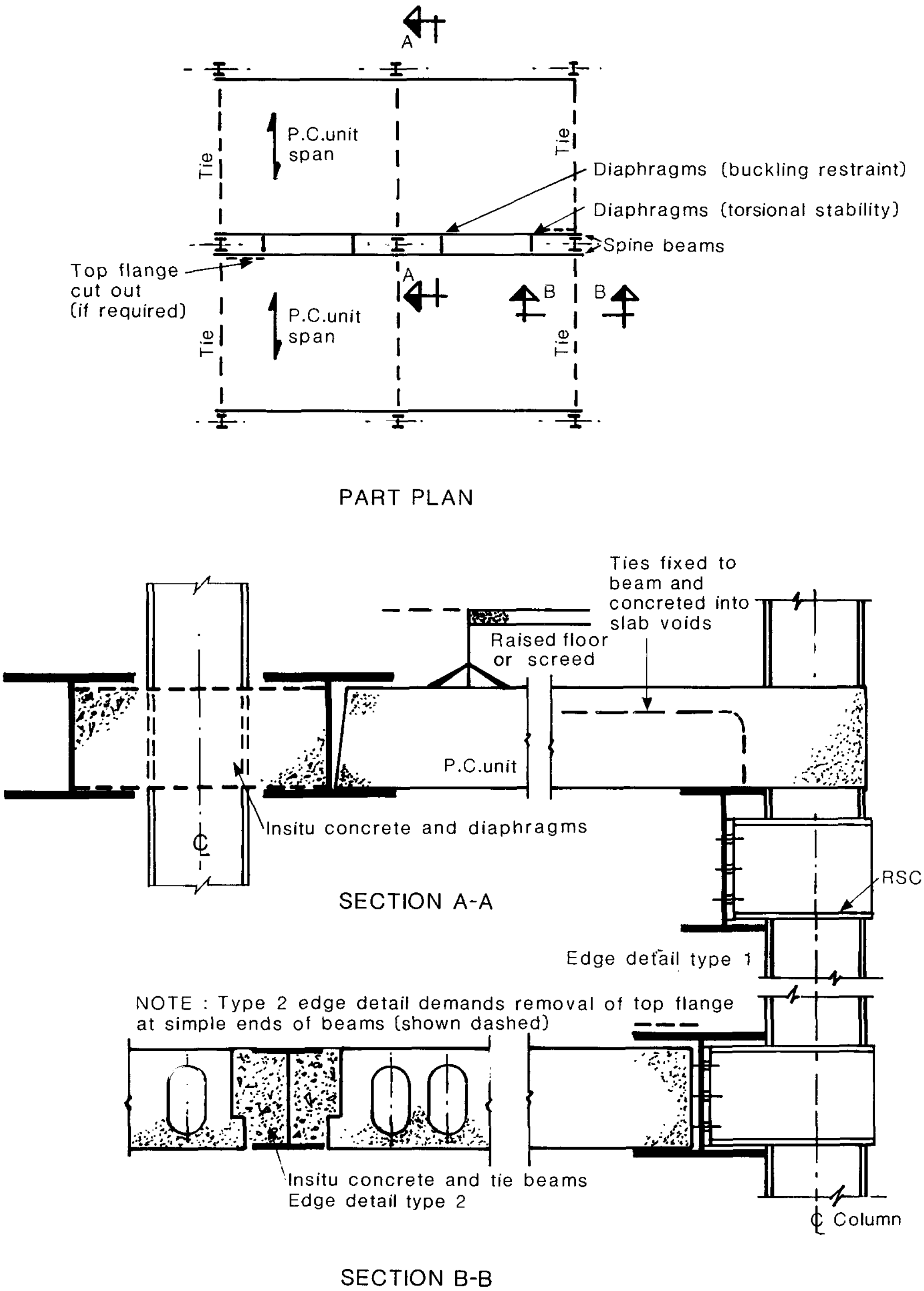


Figure 10 *Use of P.B.A. with precast planks and 'slim' floor construction*

The precast planks are supported on top of the bottom flange of the spine beams. The tie beam is contained within the precast floor zone and is protected against fire. The space around the tie beam and between the spine beams is made up with insitu concrete. The precast planks are grouted and/or mechanically fixed, as is required, in the normal way. Overall stability is provided by stiff cores or braced bays acting in conjunction with the floor plate. Stability during construction may be critical before the insitu concrete has hardened to develop diaphragm action in the floors. Erection stability may be provided by the use of temporary bracing. In addition, attention has to be paid to local beam torsions from eccentric beam reactions because the beams are only loaded on one side. This can be resolved by the use of diaphragms that are bolted between the spine beams.

To enable erection of the precast planks, the width of the spine beam flange is reduced by notching adjacent to the simply supported ends. Alternatively, where the floor is only two spans wide, the external edge beams may be lowered so that the external end of the precast units sit directly on the top flange as shown in Figure 10.

This system is limited by the depth and strength of the available UC sections. Some projection of the spine beam above the precast units may be tolerated where permitted by floor finishes or raised flooring. The spine beams are not normally greater than 305 UC's with 245 UC's being more common. Grade 50 steel can be used, provided checks are made on cumulative deflections.

Since most of the floor beams are contained within the floor concrete, the top and bottom flanges are normally only the areas requiring fire protection. These can be sprayed with fire protection or intumescent paint. In many cases a fire engineering analysis shows that 60 minutes fire resistance can be achieved without additional fire protection.

6. SPECIAL ASPECTS OF STRUCTURAL BEHAVIOUR AND CONSTRUCTION

6.1 Introduction

As with any new development in structural form, the parallel beam approach involves certain aspects of structural behaviour and construction that do not commonly occur in more traditional forms of building construction. It is important for the designer to have a sound appreciation of these in order that he may design with confidence and safety.

6.2 Lateral stability of the continuous composite ribs

6.2.1 Construction condition

(All Code references are to *BS 5950: Part 1*)

Where propped construction is adopted there is generally no requirement to consider the lateral stability of the ribs during construction.

Where propping is not used the ribs are required to remain stable under the wet concrete condition. Their top flanges are stabilised by the decking which is running transverse to the ribs (its stiff direction) and is attached at regular intervals by the through deck welding of the shear connectors. Simply supported composite beams are thus fully stable during construction. In continuous beams the situation is less straight forward. In the negative moment region the compression flange is restrained at the supports where it passes over, and is bolted to, two spine beams. The presence of two supports gives restraint to plan bending that is not available with a single support. The tension flange is laterally restrained by the decking. If the plan bending restraint to the bottom flange is neglected, this becomes a standard case in *BS 5950: Part 1*[1] and may be checked accordingly. In carrying out this check it should be noted that:

1. The rib should remain elastic throughout the construction condition;
2. the critical load case for the stability check is likely to be the situation where only one span is loaded, giving the greatest length of unrestrained compression flange in the neighbouring unloaded span;
3. buckling checks should initially be carried out to Section 4, ignoring any restraint from the deck and assuming an effective length of 1.0 × distance between spine beams (nominal span minus spacing between pairs of spine beams) or from the support to mid span restraint (goal posts). The coefficient 1.0 comes from Table 9 of the Code.
4. Where necessary *BS 5950: Part 1,* Appendix G, Clause G.3.3 may be used to determine the effective slenderness for lateral torsional buckling acknowledging the restraining influence of the deck on the top flange.

From Clause G.3.3.

$$\lambda_{TB} = n_t u \nu_t c \lambda \qquad (1)$$

The expression for ν_t simplifies to the following because the deck restraint acts at the level of the top flange:

$$\nu_t = \frac{1}{\left[1 + \frac{1}{40}\left(\frac{\lambda}{x}\right)^2\right]^{1/2}} \qquad (2)$$

6.2.2 Completed structure

Once the concrete has gained adequate strength the stability of the continuous rib is enhanced. The slab, via the shear connection, provides both lateral and torsional restraint to the top flange and this is transmitted to the bottom flange by the bending stiffness and strength of the web. Thus lateral torsional buckling is suppressed and only the distortional buckling of the type shown in Figure 11 can occur. One method of checking resistance to this form of instability is presented in reference 8. An alternative, which may be more appealing to designers familiar with *BS 5950: Part 1* is to use Appendix G to determine what discrete restraints, if any, are necessary to stabilise the rib.

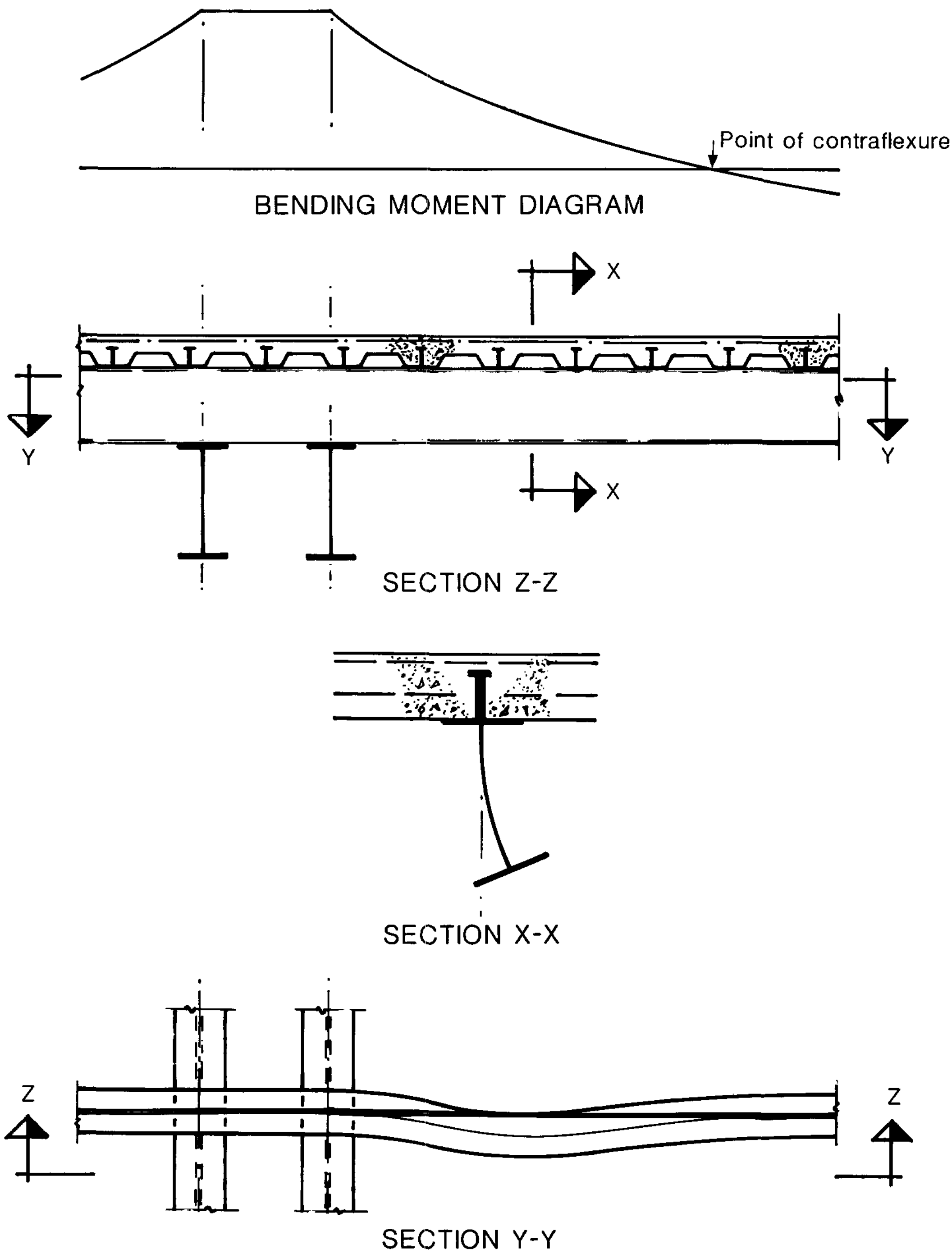


Figure 11 *Distortional buckling of a composite rib in negative moment region*

Appendix G only assumes translational restraint to the top flange. If discrete restraints are required then it will often be possible to demonstrate that reasonable lengths (i.e. small proportions of the overall span and not more than, say, 50% of the maximum unrestrained length) of shear connection and web in bending can provide a torsional restraint with adequate strength. The appropriate strength criterion for this torsional restraint is that it is capable of developing 3% of the maximum flange force. The draft Eurocode[2] gives a method of checking the overall stiffness of a restraint system which is applicable in this instance. It may be simplified to a requirement that, under the same 3% restraint force, the deflection of the restraint system shall not exceed beam span/1700.

6.3 Capacity of ribs at internal supports under combined moment, shear and bending

At internal supports the ribs normally pass over and are connected to the twin spine beams. The connection is made with four bolts through the bottom flange of the rib and the top flange of each spine beam. This connection provides a restraint against torsion and transverse bending of the bottom flange of the rib; this flange is in compression at this point. The top flange of the rib is restrained by the slab. There is therefore overall torsional restraint to the rib at all supports. Any buckling of the web will be as a plate between clamped top and bottom edges, i.e. the web may be checked to Clause 4.5.2.1 of *BS 5950: Part 1* with a slenderness of $\lambda = 2.5$ d/t. The web of the rib at internal supports is required to resist shear, moment and bearing compression. The contribution to moment resistance provided by the web is of course dependent on the magnitude of the shear and the bearing compression.

If the beam is designed plastically and a plastic hinge occurs at the supports, a stiffener will be required within D/2 of the plastic hinge to meet the requirements of *BS 5950*.

If the web remains elastic it can be analysed in the normal way and a stiffener may not be required.

The omission of stiffeners or their use on one side of a section only, clearly simplifies the fabrication of long beams. Unfortunately the development of analysis of unstiffened sections with moment, shear and bearing compression, in the plastic condition is not currently sufficiently advanced to provide reliable design methods, but further work is being carried out. However, in the simplified plastic design method that is used to obtain the appropriate rib section, certain conservative assumptions are made:

1. The analysis of the ribs is usually based on the assumption of a simple support on each column centre line. In practice the double spine beams reduce the actual span of the ribs thereby reducing the moments in the rib beam.

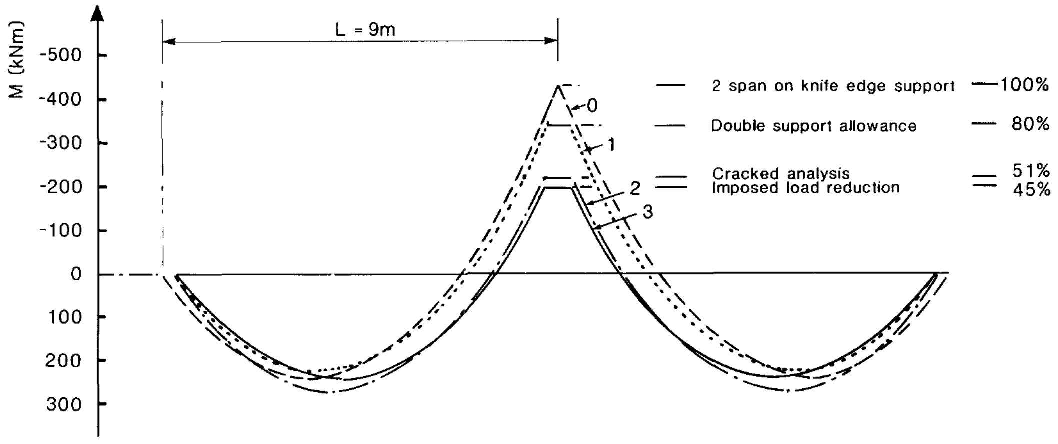
2. A uniform second moment of area is assumed throughout the beam when assessing the moment, whereas cracking will reduce this property in the support region from one half to one third of its uncracked value. The effect of this is to reduce the actual support moment and increase the span moment.

3. In calculating the maximum support moment the superimposed loading has not been reduced, but to achieve the maximum loading it would be necessary to load more than two adjacent slab spans as well as two adjacent spans of the rib.

   Appreciable reductions in superimposed load (for non-storage buildings) may therefore be allowable.

   (Figure 12 shows the influence of the above effects on a typical 2 span continuous beam; a three span beam would show similar results.)

4. The yield strength of the web is greater than the yield strength of the flange. The web yield strength is often greater than 1.1 p_y.



0 : Two 9m continuous spans (uncracked section)
1 : As above but modified to show effect of double spine and double support
2 : Bending envelope further modified for cracked analysis (0.12L each span)
3 : Imposed load reductions applied

Figure 12 *Modifications to elastic analysis to account for twin spine supports, variable inertias and imposed local reductions*

5. The section used normally has a strength slightly above that required by calculation.
6. For long spans (above 9 m say) deflection or vibration will usually dictate the section size.

The effect of all the above points is to reduce the support moment requirement and for the web in most cases to remain elastic.

Table 1, incorporating 1–3 above, has tabulated the reduction factors for the elastic moment at internal supports for ribs with two equal spans with twin spine beams 0.5 m apart and the concrete slab is assumed to be cracked over 0.12 L on either side of the support. The following variables are considered:

- Span (l_s) 9 m, 7.5 m and 6.0 m
- Bays (l_b) 9 m, 7.5 m and 6.0 m
- Ratios of $\dfrac{\text{Span Inertia}}{\text{Support Inertia}}\left(\dfrac{I_g}{I_s}\right)$ of 2, 2.5, 3 and 3.5

Appendix B gives the detailed background to Table 1.

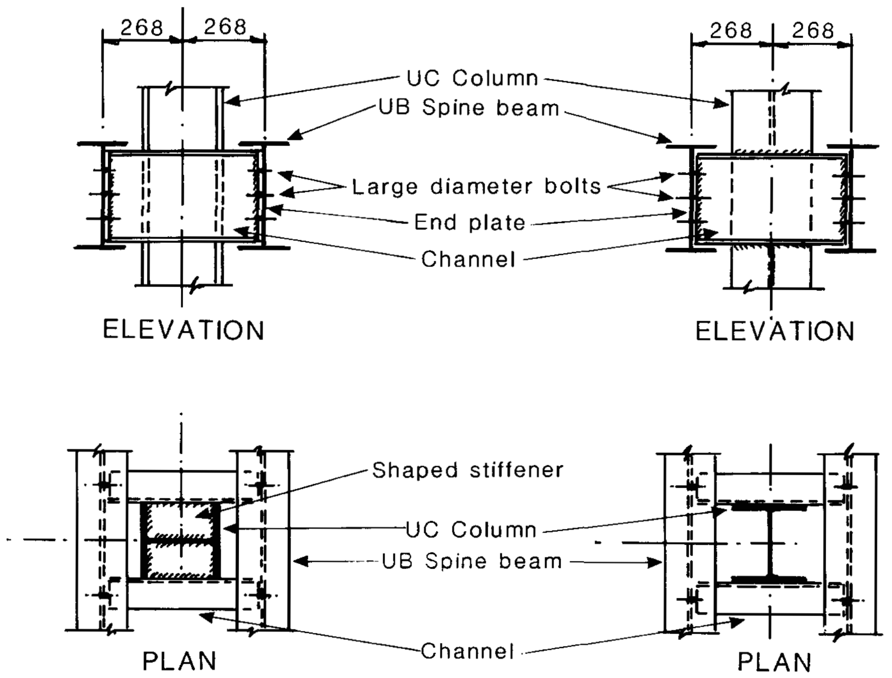
Table 1 *Reduction factors for elastic moments in two span continuous beams*

		Ratio of $\dfrac{\text{Span } I_g}{\text{Support } I_s}$			
Span	Bay width	2.0	2.5	3.0	3.5
$l_s = 9.0$ m	$l_b = 9.0$ m	0.625	0.570	0.526	0.490
	$l_b = 7.5$ m	0.638	0.582	0.537	0.500
	$l_b = 6.0$ m	0.650	0.593	0.547	0.510
$l_s = 7.5$ m	$l_b = 9.0$ m	0.630	0.574	0.511	0.490
	$l_b = 7.5$ m	0.640	0.583	0.519	0.498
	$l_b = 6.0$ m	0.651	0.593	0.527	0.507
$l_s = 6.0$ m	$l_b = 9.0$ m	0.581	0.527	0.484	0.449
	$l_b = 7.5$ m	0.588	0.533	0.489	0.455
	$l_b = 6.0$ m	0.596	0.540	0.496	0.461

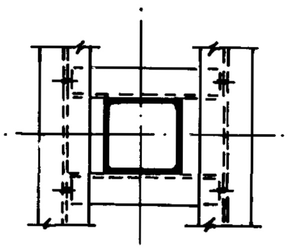
6.4 The Bracket connection

6.4.1 General

The bracket is an important component of the Parallel Beam Approach to framing. It is the mechanical connection between the spine beam and the columns, using site bolted fixings and a shop welded fabrication which is often based on a channel section. Typical bracket configurations shown in Figure 13 illustrate the load path from the spine beam to column. Bracket flexibility directly affects the moments and shears distributed through the frame members. Consideration of the loads carried by the bracket in a form suitable for calculation by hand methods, leads to simplifying assumptions aimed at modelling joint flexibility and ensuring safe design.



(a) *Bracket attached to column toes* (b) *Bracket attached to column flanges*



(c) *Bracket attached to SHS column*

Figure 13 *Typical bracket details*

6.4.2 Influence on global analysis

A conservative estimate of the spine beam moments can be obtained by analysing them as continuous beams with each beam resting on a single knife-edge support on the column grid line. This in effect assumes that the brackets are torsionally flexible.

Practical bracket arrangements have significant stiffness and therefore can cause moments both in themselves and their supporting columns from out-of-balance moments in the spine beams.

A conservative estimate of these bracket and column moments can be obtained by analysing, under appropriate loading, a sub-frame which models the spine beams as being directly attached to the column, i.e. assuming the bracket is infinitely rigid.

6.4.3 Governing load cases for bracket moments

Out of balance moments in the spine beams result either from variations in spans of spines and ribs or from pattern loading effects being transferred via the continuous ribs and spines.

Pattern loading of the ribs leads to torsion in the spine beams with consequent out of balance load effects at the column connection. Generally the spines run close beside column faces and eccentricities are not high. The provision of diaphragms joining the twin spine beams at intervals along their length encourages combined structural action of the two beams and this loading case is not normally the significant design condition.

Pattern loading of the spines leads to torsion in the bracket connection with consequent out of balance load effects at the column connection and moments induced in the column.

6.4.4 Bracket behaviour

The bracket transfers these moments to the column by a combination of the three modes of behaviour indicated in Figure 14:

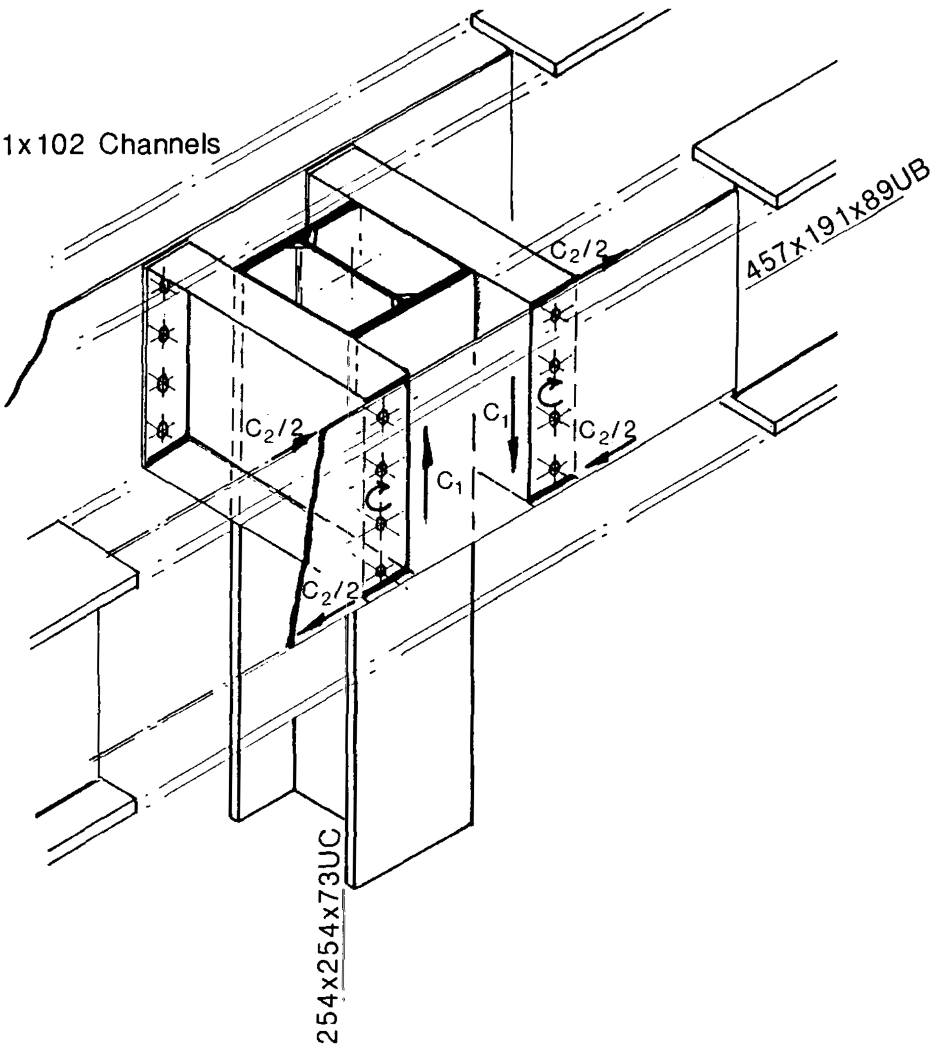


Figure 14 *Transfer of moment from spines to column by brackets*

- A couple with forces of magnitude C_1 acting through the shear centres of the channels.
- A couple with forces of total magnitude C_2 acting through, and causing transverse bending of the all channel flanges.
- Torsional moments within each channel.

The relative contributions from these modes will depend on the proportion and spacing of the channels. As illustrated in Appendix C an elastic analysis can readily be carried out by applying a unit rotation to the spine beam about the column YY axis, determining the relevant displacements of the brackets and hence determining the couple or torsion associated with each mode of behaviour. The ratio of these moments will then give the distribution of the overall moment into the three modes of behaviour. For the case considered, the elastic line of action of the forces was very close to the centreline of the bolt group. The convenient design assumption can therefore be made that the moment is transferred from the beam to the bracket by a couple with a lever arm of the bolt group cross-centres.

This elastic distribution may well be modified by lack of fit, for example if the brackets are set at different heights on the column. Because the brackets are so important to the overall structural safety of the system, it is important to ensure that they will behave in a ductile manner if subject to an overload. The only components that might behave in a non-ductile manner are the bolts in shear and the welds attaching the end plates to the brackets and the brackets to the column.

Concerns over bolt ductility are resolved by using large diameter bolts that have single shear values in excess of their bearing strength. (In the event of bearing overload on the connected plies the bolt holes under greatest load, i.e. the bolts furthest from the centre of rotation, will 'oval'. Under these conditions the inherent capacity of the connection to resist vertical load is not impaired.)

Weld ductility is achieved by designing them for the maximum moments and shears they could receive. The maximum moment is taken as the greater of:

- the factored moment arising from the elastic analysis under pattern loading described above
- an upper bound on the moment arising from lack of fit.

The maximum shear is taken as the greatest of:

- the factored shear arising from pattern loading
- the factored shear from symmetric full loading
- the shear associated with the lack of fit condition.

The worst lack of fit that could occur is for only one bracket to be effective because the bolts in the other bracket are not in bearing. Thus the basic lack of fit case is to apply the more severe of the full load or pattern load shears and moments to one bracket. Because of the conservatism of this assumption, it is suggested that a reduced load factor of 1.25 on dead and imposed load is taken for this case. (It should be noted that this extreme case is only taken for weld design, to ensure ductile behaviour of the connection. It is not necessary to use the same case for other components in the connection because if they are subject to an overload due to lack of fit they can safely yield, thus redistributing the forces in the connection.) The choice of a load factor of 1.25 for this extreme lack of fit condition is primarily a matter of engineering judgement. The best justification that can be offered is that there is a precedent within EC3 of requiring a 20% over capacity where there are concerns over lack of ductility and this should be added to the 1.05 factor that is required for a key element that has to survive an extreme event, in this case an extreme lack of fit.

6.5 Deflections of continuous composite beams and shakedown

There is no design requirement to prevent plasticity at internal supports at the serviceability limit state. In practice, where the rib is designed for strength, plasticity may occur at working loads.

At the initial application of serviceability loading the calculated elastic moments at internal supports may exceed the first yield moment of the section, leading to inelastic rotation at the internal supports. The associated inelastic deflection is normally small. However, on removal of the superimposed element of loading causing these moments, these additional deflections remain, together with a residual sagging moment in the beam. This residual moment has a prestressing effect and ensures that subsequent loading of the same nature and magnitude produces only elastic behaviour. (This action is known as 'shakedown'.)

The deflection of the beam could be calculated by first assuming the beam remains elastic everywhere and calculating the maximum deflection, which for a two equal span beam would be with dead load on both spans and superimposed on one span. This deflection could then be increased by the permanent deflection due to inelastic behaviour at the support. This approach is conservative, because it assumes two applications of the full serviceability load, the first on both spans and the second on one only. To overcome this conservatism, *BS 5950: Part 3.1* proposes that the shakedown load may be taken as the dead load plus 80% of the superimposed load.

It should be noted that supports adjacent to cantilevers are not treated as internal supports. These moments cannot be redistributed and plasticity must not occur at serviceability.

6.6 Effect of reinforcement on the strength of composite section in negative moment regions and on the cross-section classification

At an internal support the concrete tensile strength is neglected and only the tensile reinforcement is considered to complement the steel beam. However, some plasticity may occur at internal supports leading to high local strains in the reinforcement at the ultimate limit states. Cold drawn reinforcement and all bars of less than 10 mm diameter may rupture in the presence of these strains and therefore should be ignored. In sagging regions of the beam, reinforcement in the slab should be ignored when assessing the compressive strength of the section.

The Parallel Beam Approach is designed on the assumption that only plastic sections are used. In areas of negative (hogging) moment where reinforcement is used to enhance the capacity, it affects the section classification. It is necessary, therefore, to check that the steel section in the hogging region complies with:

$$b/T \leq 8.5\varepsilon \text{ and } d/t \leq \frac{64\varepsilon}{1+0.6r} \quad (3)$$

where $r = \dfrac{Y_c - Y_t}{d}$ (see Figure 15)

Where the total reinforcement within the effective flange width of the hogging moment region is less than $dt/5$ any Grade 43 steel section having a $b/T < 8.5$ and $d/t < 59$ or for Grade 50 $b/T < 7.48$ and $d/t < 52.7$ will satisfy the requirements for a Plastic section.

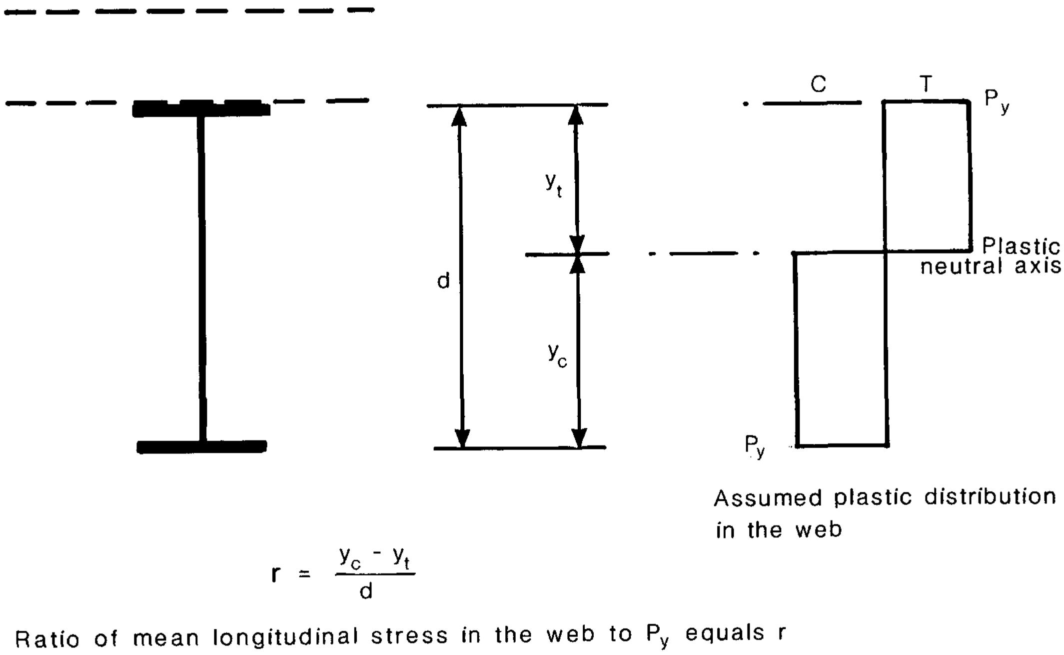


Figure 15 *Plastic stress distribution in web of composite beam under negative moment*

6.7 Comparison between proposed simple design method and elastic finite element analysis

This Section examines a 9 m × 9 m floor system and compares the hand calculated values for bending moment and deflection with an elastic computer analysis. The computer method models the following effects:

- The true disposition and support condition of the spine beams (off grid etc.).
- The variable stiffness of the composite beam due to partial interaction effects and a fully cracked zone 0.12L on either side of the centreline through the twin spine support.

The same imposed loading reductions used in the hand calculations are applied to the computer analysis at the ultimate limit state.

Results are summarised in Figures 16 to 19.

Rib strength

Table 2 *Rib bending moments under full dead plus imposed loading*

Rib Bending Moments (No imposed load reduction)		
Location	Moment from Computer Analysis	Moment Capacity provided
Support (kNm)	218.5	195.4 (Mp steel beam)
Span (kNm)	272.0*	382.0 (Composite section)

**Note that 340 is hand calculated requirement.*

At the support (see Table 2) the plastic capacity of the rib is lower than the required moment. The moment redistribution into the span, occurring after the section has reached full yield is:

$$\frac{218.5 - 195.4}{2} = 11.6 \text{ kNm (i.e. 10.6\% without the benefit of imposed load reduction)}$$

Clause 5.4.1 of *BS 5950* allows the elastic moment diagram for continuous beams to be modified by up to 10% providing the moments and shears remain in equilibrium with the factored loads.

Notwithstanding the Code allowance, the effect of strain hardening and the fact that typically, a web of a rolled section has a yield strength of $1.1p_y$, suggests that the support strength could be in excess of that required by analysis.

In the sagging region of the span, the composite section strength is

$$\frac{382-(272+11.6)}{382}\times 100 = 26\%.$$

in excess of that required by the computer analysis modified by the redistribution of 11.6 kNm into the span. This achieves a considerable reserve against the formation of a mechanism provided the support section will allow the necessary degree of rotation (NB: Plastic sections only are used).

Spine beam strength

Table 3 *Spine Bending Moments allowing 25% imposed load reduction*

Spine Bending Moments		
Location	Moment from Computer Analysis	Moment Capacity provided
Support (kNm)	2 × 555	2 × 534 (Mp)
Span (kNm)	2 × 349†	2 × 429*

**i.e. the buckling resistance M_b between rib connections.*
†Note that the hand calculation equivalent uniform moment is 2 × 415.

At the support the moment redistribution into the span, occuring after the section has reached its plastic moment is:

$$\frac{555-534}{534}\times 100 = 4\%$$

As stated above, strain hardening etc., suggests that a lower percentage will occur in practice. Clause 5.4.1 of *BS 5950* would also apply, permitting 10% redistribution.

In the sagging region of the span the limiting criterion is elastic buckling between the rib connections. If required, additional diaphragms can be used between rib centres. From the above results (Table 3) it can be seen that the span moment reserve is 429 − 349 = 80 kNm per beam, when 4% of the support moment has been redistributed, this spare capacity reduces to 70 kNm per beam (i.e. 40% more than that required).

Deflections

Table 4 *Deflections. (Note: 16% reduction in imposed load assumed in serviceability analysis of spine beams)*

Deflections		
Elements	Deflection from Computer Analysis	Deflection by Hand calculation
Rib (dead) mm	14.1	14.3
Spine (dead) mm	7.5	10.8
Rib (imposed) mm	18.8	18.3
Spine (imposed) mm	14.0	16.5
Maximum (dead) mm	21.6	–
Maximum (imposed) mm	29.4	–
Maximum estimated mm	51.0	59.9

The deflection summary (see Table 4) obtained from the computer corresponds to the worst pattern for rib and spine 'imposed' deflections, e.g. region A–B/1–2 in Figure 9. Although the analysis does not take account of inelastic effects, it does allow for cracking in the hogging bending region and a reduction in stiffness elsewhere, due to partial interaction.

The hand calculated deflections are conservative in that they have been calculated *individually* for rib and spine under their worst pattern imposed loading e.g. region A–B/1–3 in Figure 9 and factored up to take account of partial interaction. The worst loading for rib deflection does not coincide with that for the spine.

The spine remains elastic under pattern loading but inelastic effects in the rib have been calculated in accordance with *BS 5950: Part 3.1*, (shakedown deflection).

Bearing in mind that only partial cracking of the concrete slab under negative moment will occur, the computer analysis is conservative by assuming the slab is fully cracked and therefore allowing for more elastic redistribution than would occur in reality. In summary, the hand calculated deflection estimate is a safe conservative estimate of overall floor deflections.

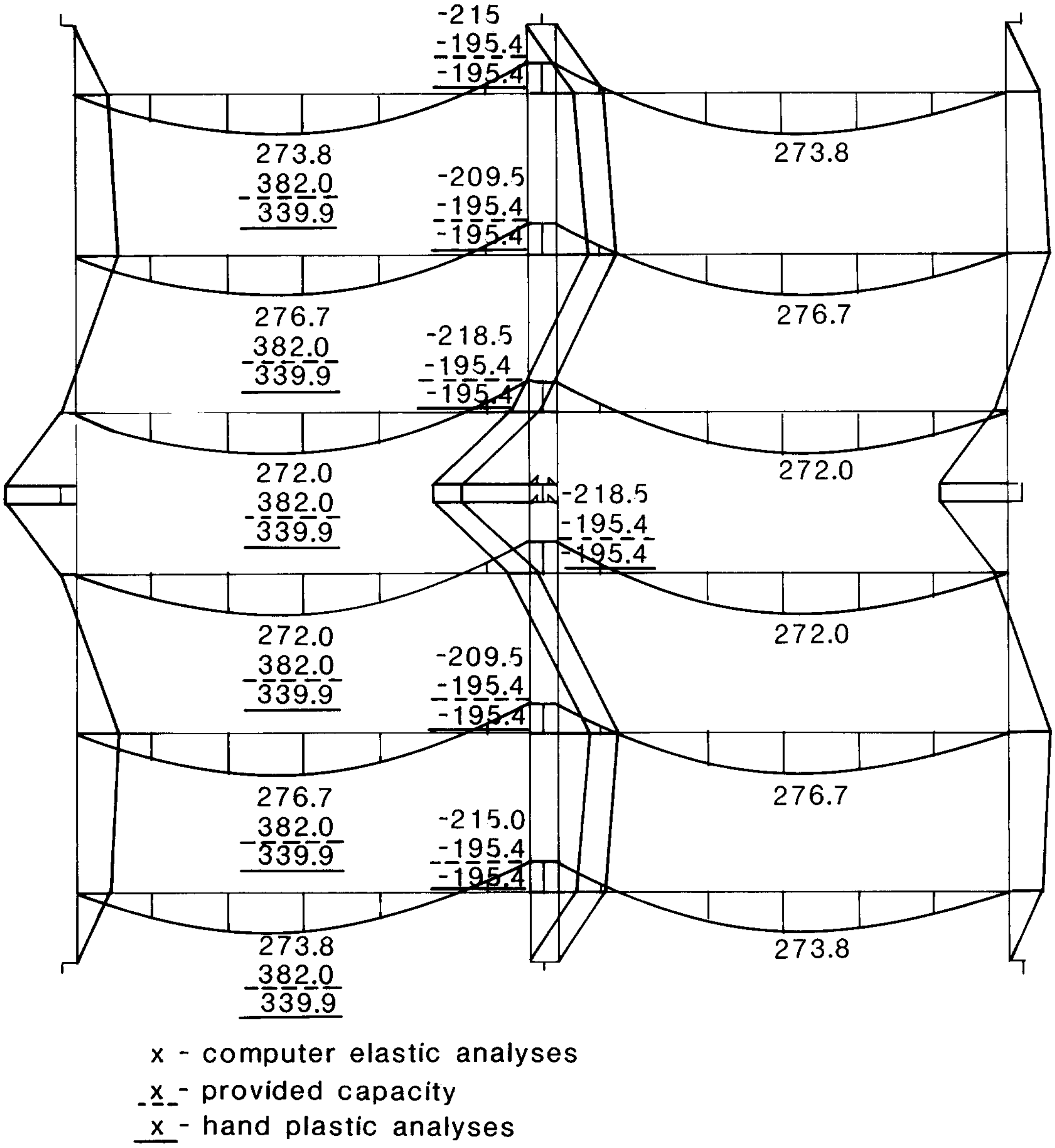
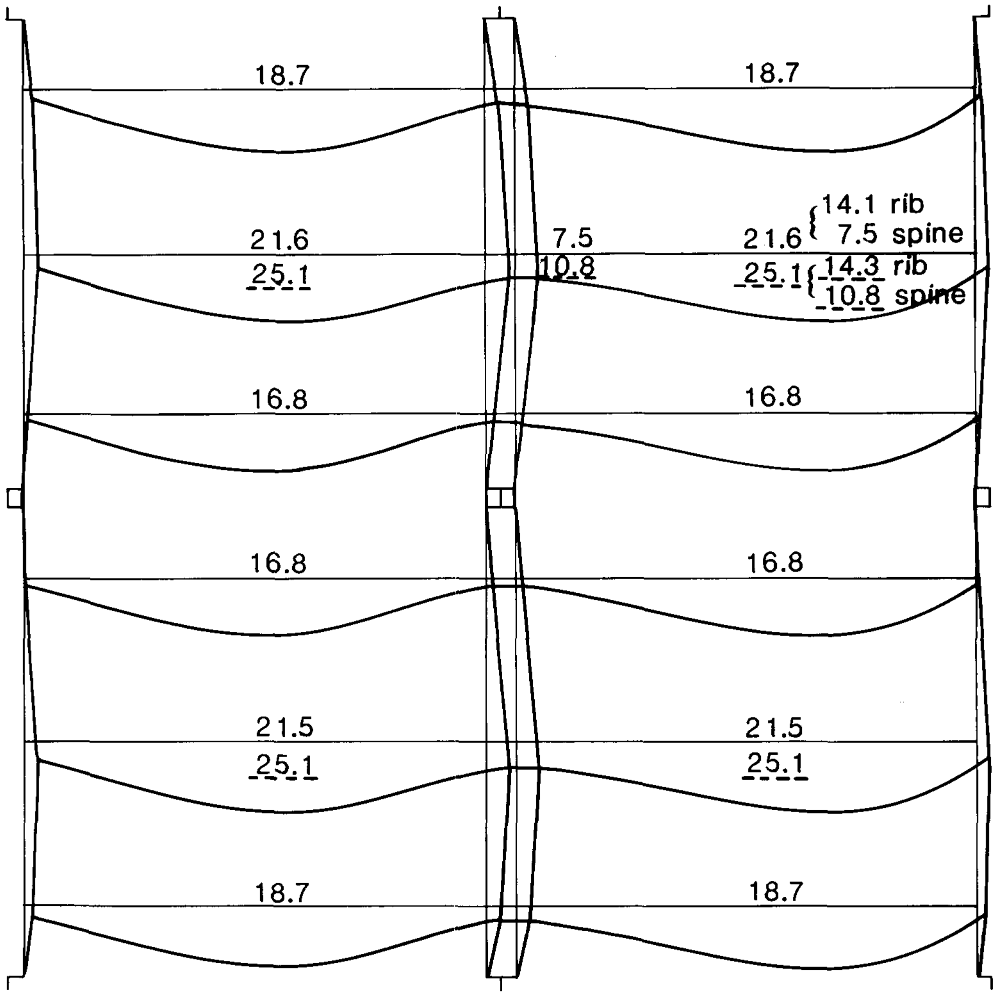


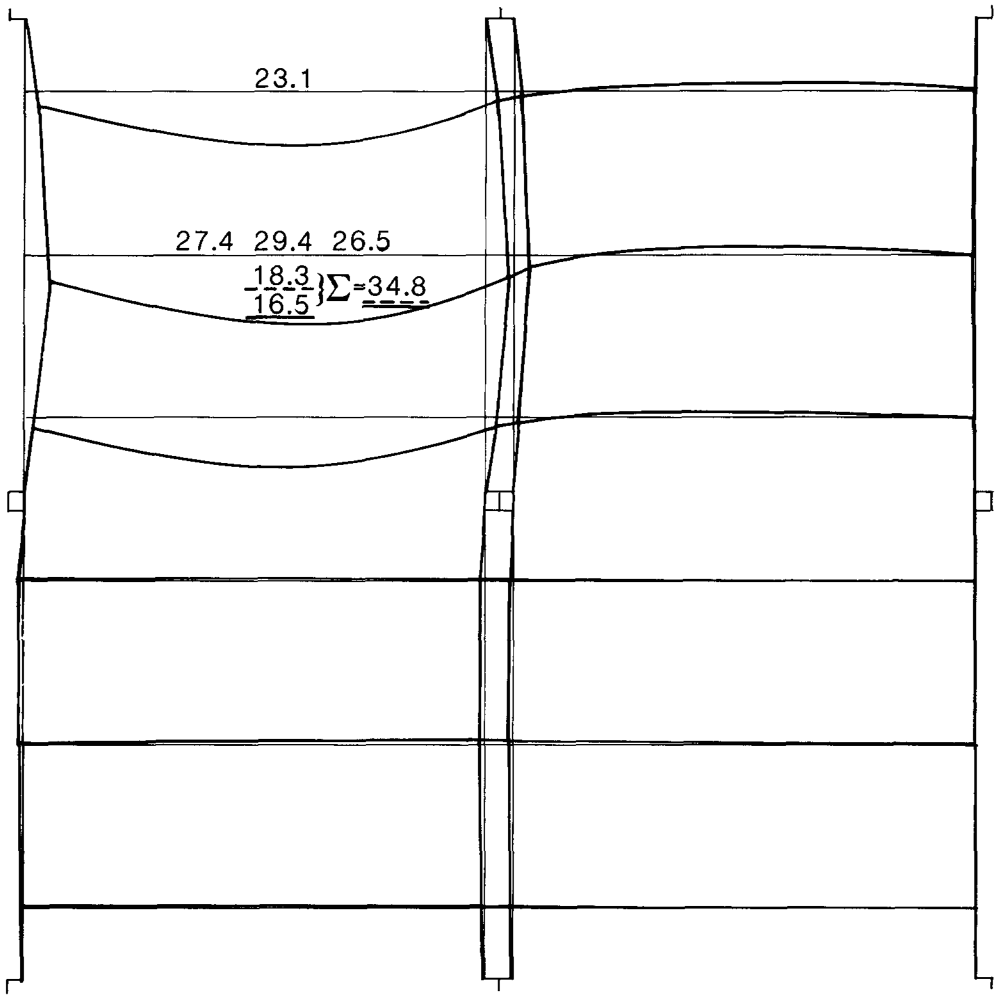
Figure 16 *Comparative analyses, rib bending moment, maximum dead and imposed load on all span*



x - computer analyses

x - hand calculations

Figure 17 *Comparative analyses, rib deflections, full dead loading including spine deflections*



x - computer calculation under full live

x - hand calculation rib under full live

x - hand calculation spine under reduced live, but more onerous load distribution

Figure 18 *Comparative analyses, rib deflections, pattern imposed loading including spine deflections*

Discuss me ...

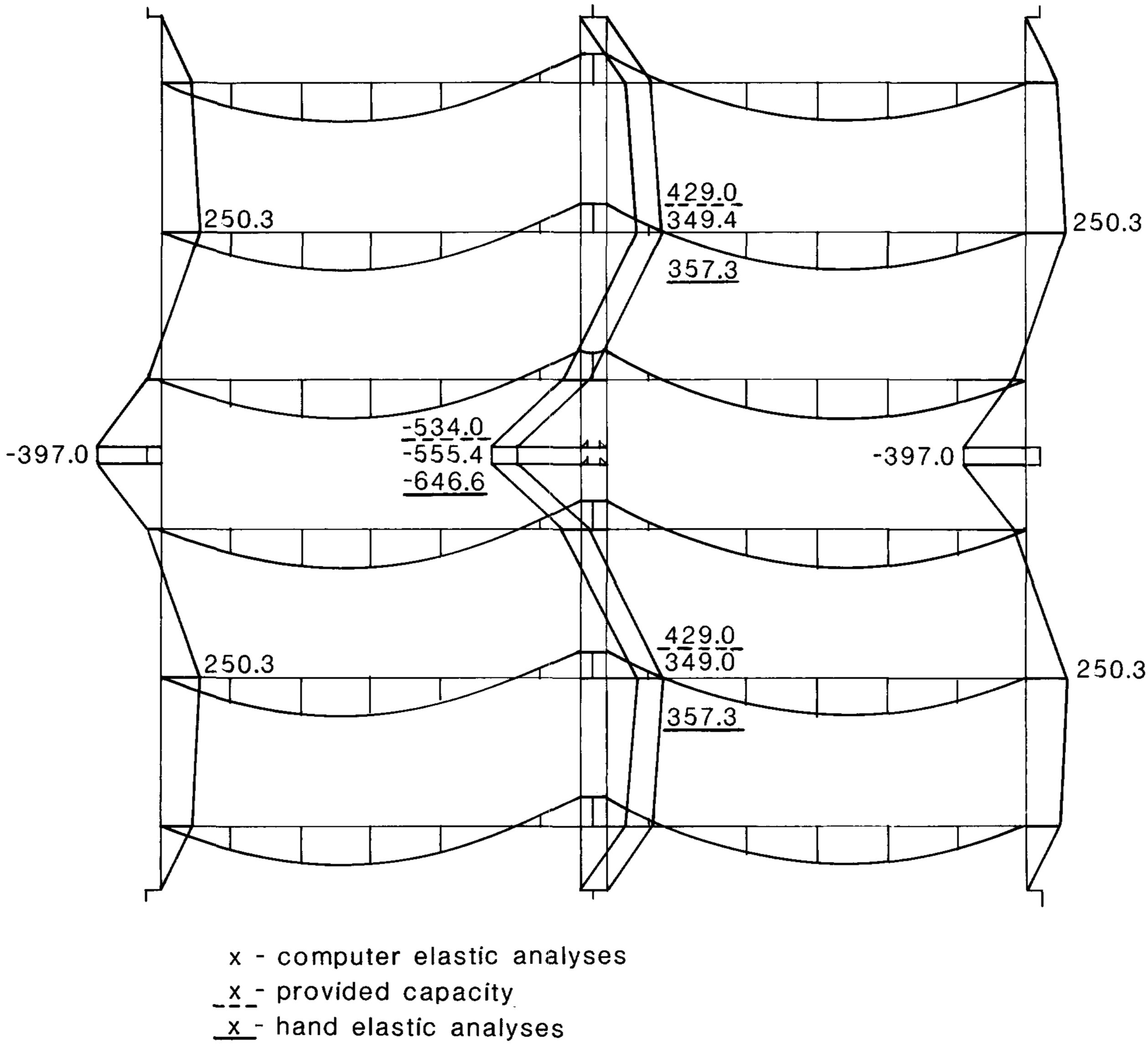


Figure 19 *Comparative analyses, spine bending moments, full dead and imposed load on all spans*

7. DESIGN

7.1 Introduction

The Parallel Beam Approach generally aims to achieve continuous beam design; however, simple design or a mixture of simple and continuous design may be used to achieve the best solution for a given project.

The continuous beams may be of composite or non-composite construction, normally the twin spine beams are non-composite and the ribs are composite.

Global analysis of a frame with the P.B.A. beams assumes the floor slab or a system of bracing, transmits wind loads and horizontal sway forces to vertical braced frames or stiffcores.

7.2 Scheme design

The structure should be laid out to the general principles covered in Section 4, to satisfy the architectural brief.

The slab thickness is determined by the deck profile, fire and sound requirements and, in some instances, the rib spacing. Slab thickness is typically 130 mm which can provide up to 1.5 hour fire resistance and avoid congestion of reinforcement mesh at laps. Decking profiles are generally designed to be unpropped in the wet concrete condition. Lightweight concrete is normally used to reduce shrinkage, give enhanced fire resistance and reduce weight. Its reduced weight improves the spanning capacity of the unpropped deck and reduces loads on foundations. Typical slab spans are 2.4 m to 3.0 m but 3.6 m is possible.

The rib beams are generally designed as composite beams. For most schemes the section will be determined by the end span moments and deflections in the end span. Johnson[(5)] gives the ratio of support moment to span moment of 0.5–0.7 for uniformly loaded end spans.

If a ratio of 0.6 is adopted, this implies a support moment of $0.45 \times WL/8$ (where W is the factored Dead and Imposed Load) or $WL/17.7$.

This is used to estimate the required plastic modulus of the steel beam. Grade 43 is normally used. The serviceability limit state can generally be satisfied if the span/depth ratios shown in Table 5 are followed.

Table 5 *Typical span/depth ratios for composite beams*

Nature of composite span	Span/Steel beam depth
Simple composite beams	23–25
End span continuous composite beams	28–30
Internal span continuous beams	33–35

Only plastic sections are selected. To assist handling, B (the breadth of the steel beam) should be approximately 1/125 times the length to be handled.

As an example of the above, a beam having two 7.5 spans carrying a uniformly distributed load and being delivered to site 15 m long would require the following section properties:

$$D = \frac{7.5 \times 10^3}{30} = 250 \text{ mm}$$

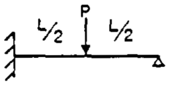
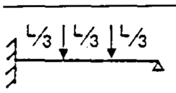
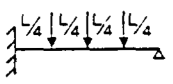
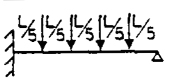
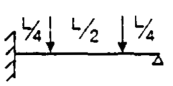
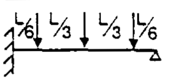
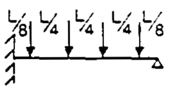
$$B = \frac{15 \times 10^3}{125} = 120 \text{ mm}$$

$$S_x = \frac{0.45 \text{ WL}}{8 \text{ P}_y}$$

where W = factored Dead and Imposed loads

The spine beams may be sized by plastic analysis for bays up to 9 m × 9 m. Grade 50 may be used where deflections are not a problem. Where spans are equal or end spans are the greater, the latter will govern design and Table 6 tabulates required plastic moment capacity. If internal spans are greater they may govern.

Table 6 *Spine Design*

Load Case	Mp	Elastic Support Moment	Maximum Deflection
	$\frac{PL}{6}$	$\frac{3PL}{16}$	$\frac{PL^3}{107 EI}$
	$\frac{PL}{4}$	$\frac{PL}{3}$	$\frac{PL^3}{66 EI}$
	$\frac{PL}{3}$	$\frac{15 PL}{32}$	$\frac{PL^3}{48 EI}$
	$\frac{3 PL}{7}$	$\frac{3 PL}{5}$	$\frac{PL^3}{38 EI}$
	$\frac{PL}{5}$	$\frac{5 PL}{13}$	$\frac{PL^3}{86 EI}$
	$\frac{5 PL}{18}$	$\frac{19 PL}{48}$	$\frac{PL^3}{59 EI}$
	$\frac{4 PL}{14}$	$\frac{33 PL}{64}$	$\frac{2 PL^3}{9 EI}$

Above a grid size of 9 m × 9 m, deflections are likely to govern design. The rib span is significant to spine beam selection because the suggested deflection criteria is to limit total live load deflection (i.e. slabs, ribs and spines) of the bay to diagonal length/360. Spine selection will therefore depend on the remaining deflection allowance after slab and rib deflection have been allowed for. Approximate deflection checks for the rib are assisted by Figure 20 which gives the factors by which steel beam second moment of area (I) should be multiplied to obtain an initial estimate of the equivalent I of the composite ribs.

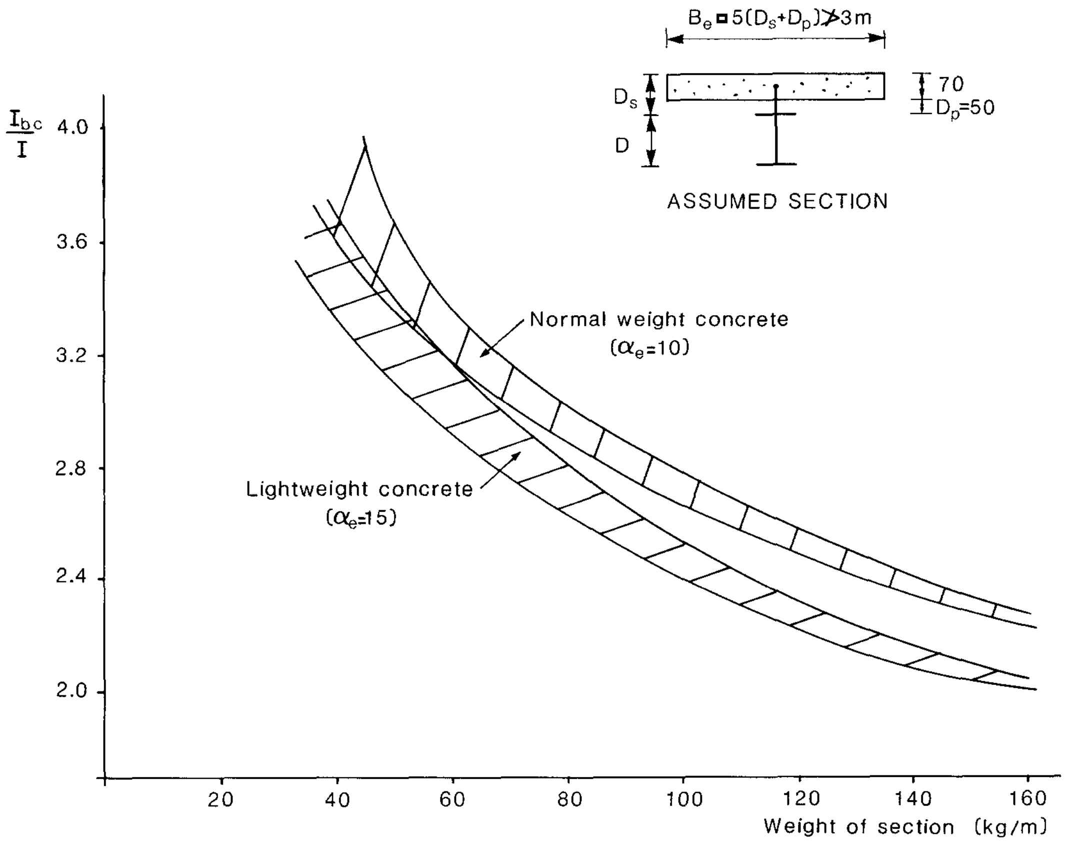


Figure 20 *Ratio of second moment of area of composite section to that of steel section*

7.3 Detailed design

The detailed design of this approach is fully illustrated in the Worked Example. The commentary provides necessary guidance on the practical application of the special considerations presented in Section 6.

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Appendix A: SAMPLE SPECIFICATION FOR SITE WELDING

INTRODUCTION

This appendix presents a sample specification for site welding and weld testing, together with typical site weld preparation drawings (see Figures A1, A2 and A3).

A1. Welding specification

A1.1 Welding consumables

All welding consumables for *BS 4360* steels shall comply with Clause 5 in *BS 5135:1984*. Consumables shall be selected to ensure that the performance of the deposited weld metal is not less than that of the parent metal.

Proper protection shall be provided for all consumables against damage or the effects of weather, preferably in a heated store.

Consumables showing signs of damage or deterioration shall not be used.

A1.2 Welding procedure specifications

Welding procedure specifications are required for all butt welds, unless written instructions to the contrary have been obtained from the Engineer.

The procedures shall include the following information:

(a) Weld preparation.
(b) The classification, type and size of electrode to be used.
(c) Method of providing preheating where required.
(d) The welding sequence.
(e) The arrangement, size and number of weld runs required.
(f) Any other relevant information to ensure compliance with Clause 20 in *BS 5135: 1984.*

Testing of the procedures is required in accordance with *BS 4870: Part 1*. This may also require procedure tests to be carried out on certain joints prior to fabrication in accordance with BS 4870:Part 1.

A1.3 Welding and welders

Welding shall only be performed by welders certified in accordance with *BS 4871: Part 1*. A copy of the Welders Approval Test record (for each welder employed on the construction site) should be made available to the Engineer before fabrication commences.

Under no circumstances shall welders perform a particular type of weld for which they are not certified.

Welding shall be carried out on clean, dry material, free from all millscale and rust, in conditions conductive to achieving good welds. Material over 40 mm thick requiring butt welds shall be preheated in accordance with *BS 5135* to a temperature of 125°C. For thicknesses below this, the material shall be warmed to a minimum temperature of 25°C to dispel moisture.

All welds and welded material shall be cleaned free of all weld slag and weld splatter.

The Contractor shall ensure that distortion during welding is minimal. The Engineer's attention should be drawn to any detail which may, in spite of the Contractor's proposed details, cause excessive distortion. The Engineer shall also be informed of any distortion due to welding and all excessive weld distortion shall be rejected and rectified or replaced at the Contractor's expense.

A1.4 Site welding

Site welding shall be carried out in accordance with Clauses A1.2 and A1.3 of this specification.

The Contractor is to keep a permanent record of which welder has carried out each weld with the date and time each weld was started and finished and the weather conditions.

The Contractor shall provide moveable covers of a type which give proper protection to the welders and welded connections during inclement weather.

Cope holes shall be filled using methods and materials approved in writing by the Engineer.

The Contractor shall provide all necessary jacking and levelling devices to ensure that the member 'fit' is within specified tolerances prior to commencement of welding.

Where specified by the Engineer, the Contractor shall use such jacking and levelling devices necessary to ensure the mid-span points are within specified tolerances of the support positions.

A2. Weld testing

A2.1 All weld testing shall be carried out by an approved independent Test Authority (see 'Conditions of Tender and Contract' for specified authority).

A2.2 The Contractor shall be responsible for liaising with the independent Testing Authority as to programme, with a copy of all correspondence being forwarded to the Engineer.

A2.3 The Contractor shall provide all necessary access and covers to allow testing to proceed unimpeded.

A2.4 The Contractor shall observe all safety precautions, particularly during the use of radiographic tests.

A2.5 Where a series of similar members are used, one in four members shall be tested. Where less than eight similar members are employed, a minimum of two tests shall be required.

In the event of one test showing unacceptable results, the Contractor shall provide satisfactory evidence for all other members at his own expense.

A2.6 Where a weld is found to be unacceptable, the Contractor shall bear the cost of cutting out the weld in a way which leaves the subsequent strength of the member unimpaired and replacing it with a sound weld.

A2.7 Where practicable, all butt welds shall be tested by either radiographic or ultrasonic methods. No other testing method shall be accepted without the written consent of the Engineer.

A2.8 All other welds shall be tested as requested by the Engineer, at the Client's expense.

A2.9 Where site welding is an integral part of the structural concept (e.g. butt welding of continuous beams) test welds shall be carried out before fabrication, at the contractor's works, in order to prove the suitability of welding procedures.

A2.10 The rate of testing site welds shall be agreed between all parties after the initial testing of the first stage welds. The rates shall be reviewed as site work commences and shall be adjusted at the Engineer's discretion.

Prior to testing, welding procedures shall be forwarded to the Engineer for his comments.

A2.11 The acceptance criteria to be applied during non-destructive testing of all welded connections shall be specified by the Engineer prior to commencement of site welding.

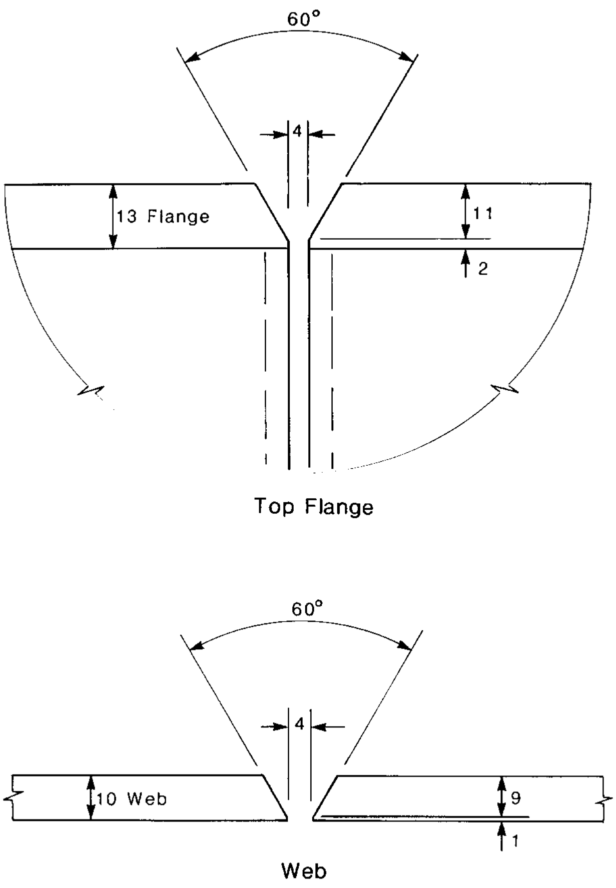


Figure A1 *Site weld preparation for 533 × 210UB82*

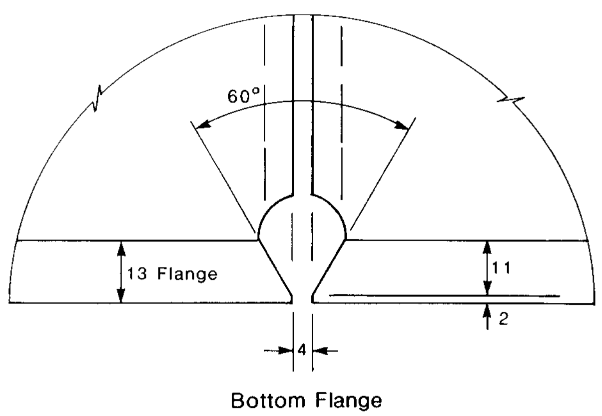
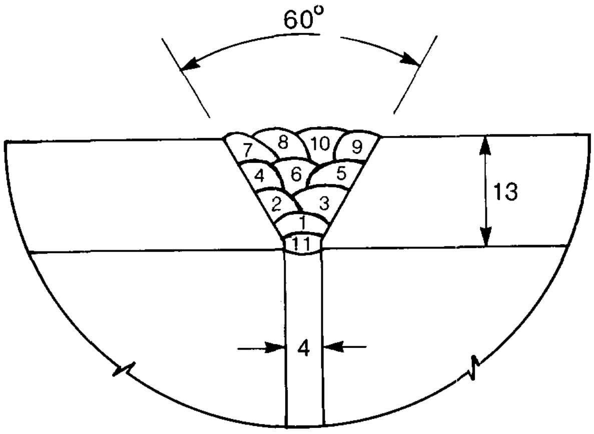
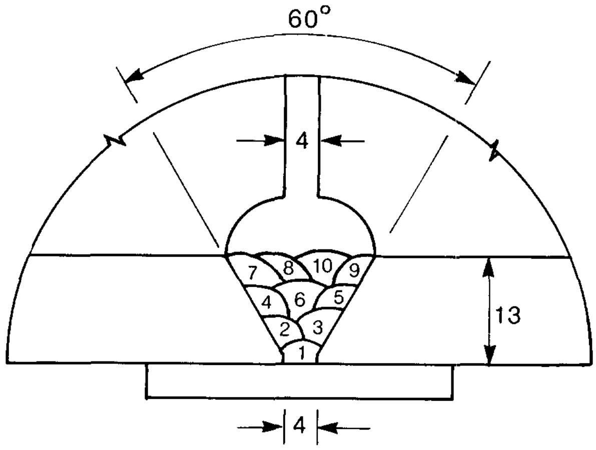


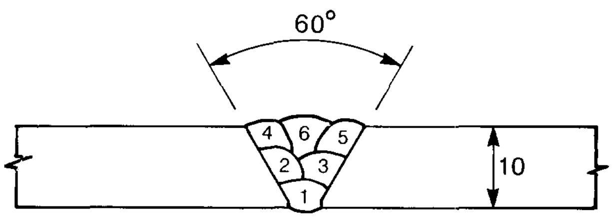
Figure A2 *Site weld preparation for 533 × 210UB82*



(a) *Sample weld procedure for top flange*



(b) *Sample weld procedure for bottom flange*



(c) *Sample weld procedure for web*

Figure A3 *Sample weld procedures*

Appendix B: BACKGROUND TO TABLE 1

If the web of the steel beam remains elastic over the internal support under ultimate loading, then *BS 5950: Part 1* does not automatically require a bearing stiffener. Instead an elastic bearing check can be performed, even when the flanges go fully plastic.

For hot rolled universal beams, the moment of resistance, M_{pf}, which corresponds with full plasticity in the flanges is between the elastic moment of resistance, M_{es}, and the plastic moment of resistance M_{ps} of the section.

Table 1 provides limiting distribution factors for $w_u l^2/8$, which ensure that the web remains elastic. If the web is adequate for bearing and buckling no stiffener is required.

This Table is applicable to ribs with certain practical geometric and loading characteristics, i.e:

- the spans of the ribs are equal in length
- dead load = 3.3 kN/M^2
  imposed load = 6.0 kN/M^2
- the distance between the twin internal supports is 0.5 m
- the steel beams are UB.

To obtain this Table, a number of design assumptions have been made:

- the slab is cracked over a distance of $0.12l$ on either side of the centreline through the internal support
- live load reductions are applied to the rib. The loaded area is defined by the bay width (l_b) × the length of the rib ($2l_s$)
- the ratio M_{ps}/M_{pf} is 1.08.

The use and the background to this Table are given below in a number of design steps.

In order to scheme the rib beam, the nominal elastic design moment $w_u l_s^2/8$ for full factored dead and superimposed load can easily be calculated, where the span l_s, is the nominal span between column centrelines.

Due to cracking, live load reductions and twin internal supports, the actual elastic moment (M_u) at the internal centreline will be less than $w_u l_s^2/8$.

$$\text{Say } M_u = \alpha^1 \frac{w_u l_s^2}{8} \qquad \text{(B1)}$$

Since the live load reductions are coupled to both the bay width and the span and since cracking is characterised by the ratio I_g/I_s, M_u is a function of I_g/I_s, l_s and l_b. For the web to remain elastic M_u should be less than M_{pf}, i.e:

$$M_u(I_g/I_s, l_s, l_b) < M_{pf} \qquad \text{(B2)}$$

Since only values of M_{ps} and M_{es} are readily available, the ratio M_{ps}/M_{pf} has been calculated for all practical universal beam sections. The average ratio is 1.08, i.e:

$$M_{pf} = M_{ps}/1.08 \qquad \text{(B3)}$$

Therefore, to avoid web plasticity,

$$M_{ps} > 1.08\ \alpha^1 \frac{w_u l_s^2}{8} \qquad \text{(B4)}$$

If we define α as $1.08\ \alpha^1$, for web plasticity to be avoided:

$$M_{ps} > \alpha \frac{w_u l_s^2}{8} \qquad \text{(B5)}$$

Table 1 tabulates values of α for different I_g/I_s, l_s and l_b.

The use of this Table is illustrated in the rib design of the worked example.

Appendix C: ELASTIC ANALYSIS OF COLUMN BRACKETS

As shown in Figure 14, moments may be transferred from the twin spine beams to the column through the brackets by means of the following three structual actions:

(1) A couple with forces of magnitude C_1, acting at the shear centres of the channels.

(2) A couple with forces of total magnitude C_2 acting through and causing horizontal bending of all the channel flanges.

(3) Torsional moments within each channel.

An elastic analysis is carried out below, which determines the relative contributions of these three actions to the transfer of the total moment and hence determines the effective line of action of the resulting forces. It relates to the twin 381 × 102 channels passing a 254 × 254 UC 73 that is shown in Figure C1.

Consider the vertical bending shown in Figure C2(a)

(i) Bending displacement of channel continuous over two supports.

$$\delta = \tfrac{1}{3}\frac{PN^3}{EI} + \tfrac{1}{2}\frac{PLN^2}{EI}$$

For unit P and geometry shown

$$\delta = \frac{\tfrac{1}{3} \times 1 \times 120^3 + \tfrac{1}{2} \times 1 \times 240 \times 120^2}{205 \times 14894 \times 10^7} = 7.5 \times 10^{-8} \text{ mm/N}$$

(ii) Cantilever bending

$$\delta = \frac{PN^3}{3EI} = \frac{1.0 \times 120^3}{205 \times 14894 \times 107} = 1.9 \times 10^{-8} \text{ mm/N}$$

Since channel is partially restrained by attachment to column and stiffeners, take bending displacement as average of (i) and (ii).

$$\delta = \frac{(7.5 + 1.9)}{2} \times 10^{-8} = 4.7 \times 10^{-8} \text{ mm/N}$$

(iii) Shear deflection from *Roark's formulas for stress and strain*(11)

$$\delta = \frac{F(PN)}{AG}$$

where G is the shear modulus

Approximate results can be obtained for I and channel beams by using $F = 1.0$ and taking A as area of web.

$$\delta = \frac{1.0 \times 1.0 \times 120}{381 \times 10.4 \times 79 \times 10^3} = 3.83 \times 10^{-7} \text{ mm/N}$$

Total $\quad \delta = (0.47 + 3.83) \times 10^{-7} = 4.3 \times 10^{-7}$ mm/N

Corresponding couple acting on one side of bracket system under unit rotation of double channel system (see Figure C1).

$$= \frac{1 \times 98 \times 2}{4.3 \times 10^{-7}/98} = 4.47 \times 10^{10} \text{ N/mm}$$

Consider the horizontal bending of the channel flanges, acting as cantilevers from the centreline of the UC flanges.

(a) Bending displacement (allowing 10% increase because of lack of total restraint at centreline of UC flange)

$$\delta = 1.1 \frac{PN^3}{3EI} = \frac{1.1 \times 1 \times 120^3}{3 \times 205 \times 2.62 \times 10^9} = 1.18 \times 10^{-6} \text{ mm/N}$$

(b) Shear displacement

$$\delta = \frac{F(PN)}{AG} \text{ when F} = 1.0 \text{ and A} = 0.7 \text{ of flange area}$$

$$\delta = \frac{1.0 \times 1.0 \times 120}{0.7 \times 102 \times 16 \times 79 \times 10^3} = 1.33 \times 10^{-6} \text{ mm/N}$$

Total displacement $= 2.51 \times 10^{-6}$ mm/N

Corresponding couple acting on one side of bracket system under unit rotation of double channel system (see Figure C1)

$$= \frac{182.5 \times 365 \times 2}{2.51 \times 10^{-6}} = 5.31 \times 10^{10} \text{ N/mm}$$

Consider the twisting of the channels, ignoring the warping restraint which has been considered above.

Twist under unit torque $\theta = \dfrac{T.N}{G.J}$

J for channel section under consideration $= 46$ cm^4

$$\theta = \frac{1.120}{79 \times 10^3 \times 46 \times 10^4} = 3.3 \times 10^{-9} \text{ radians/N.mm}$$

Corresponding couple acting on one side of bracket system under unit rotation of double channel system.

$$= \frac{2}{3.3 \times 10^{-9}} = 0.06 \times 10^{10} \text{ N.mm}$$

Thus total reactive torque induced in one side of bracket system under a unit rotation is given by:

(1) Vertical bending of channels	4.47×10^{10} N.mm (45%)
(2) Horizontal bending of channel flanges	5.33×10^{10} N.mm (54%)
(3) Unrestrained torsion of channel section	0.06×10^{10} N.mm (1%)

(As a comparison analysis of restrained torsion (reference 10) gave the following distributions: 1. 45%, 2 and 3 combined 55%. Thus the uncertainties of boundary conditions, which the foregoing analysis seeks to resolve appear to cancel out in this instance.)

The vertical bending action gives a vertical couple acting through the channel shear centres, i.e. 98 mm from the column centreline. The line of action for the total response corresponds to lines of action of:

$$\frac{98}{0.45} = 218 \text{ mm from the column centreline}$$

Discuss me ...

This is 40 mm (218–98–29.2–51) away from the centrelines of the bolt groups, as shown in Figure C3.

When these out of balance vertical forces are combined with the symmetric forces transferring the vertical shear to the column, the effective line of action on the governing side (where the two effects are cumulative) will close on the bolt group centreline, to the point where the eccentricity may be discounted.

In this instance the final eccentricity was 12mm and this only reduces the bolt group capacity by 1%.

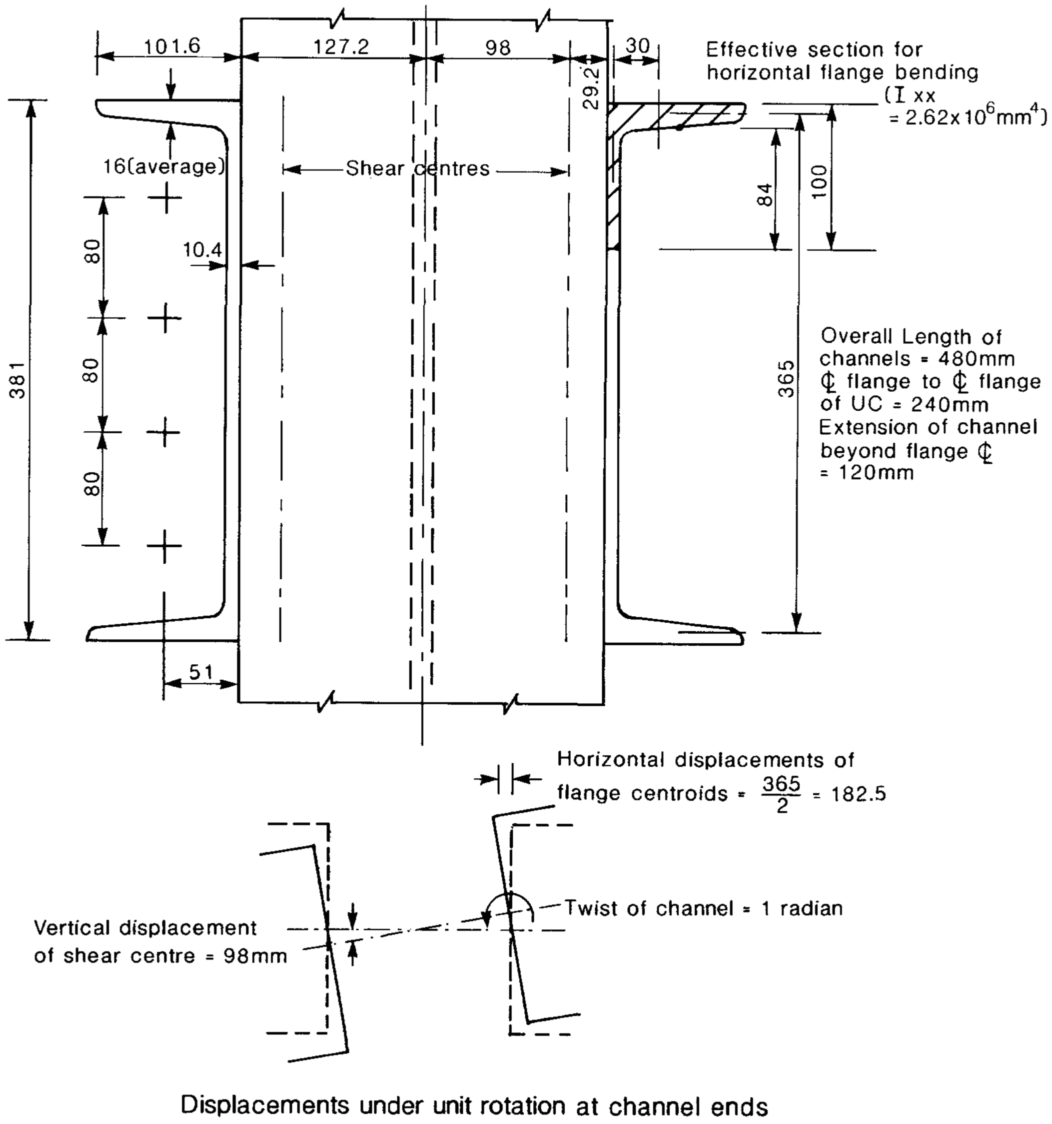
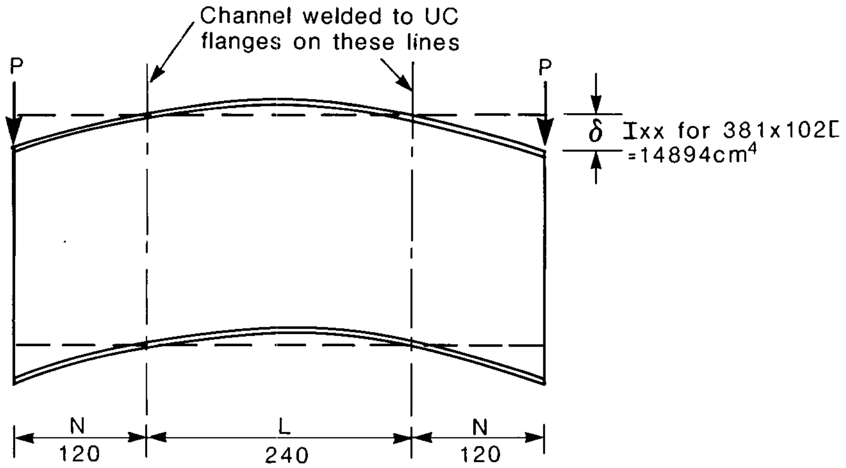
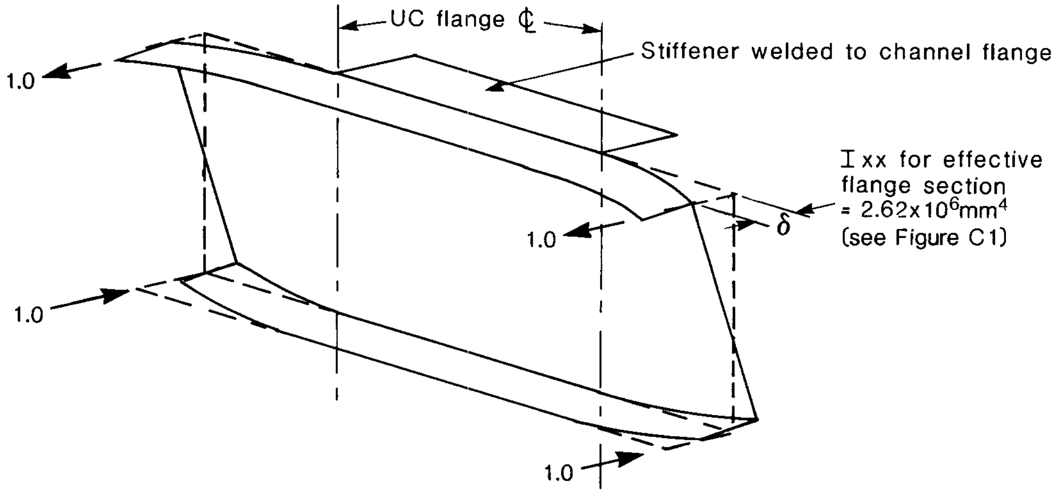


Figure C1 *Geometry of twin 381 × 102 channels passing 254 × 254UC*



(a) *Verical bending of channels*



(b) *Horizontal bending of channel flanges*

Figure C2 *Displacements of channel brackets under local forces arising from out-of-balance moments*

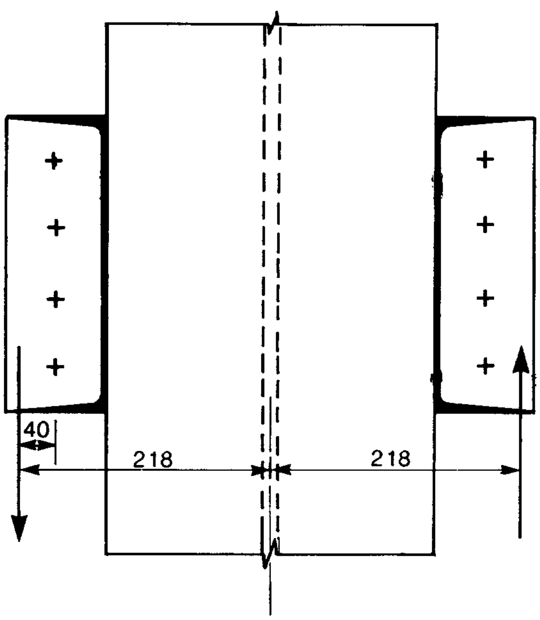


Figure C3 *Lines of action of equivalent couple transferring out-of-balance moment from spine beams into column*

Appendix D: Worked Example

Use of this example

In this section of the worked example, a calculation sheet may be preceded by a commentary sheet. Where this occurs, the commentary sheet should be considered as the left-hand page (facing) and the calculation sheet as the right-hand page of a pair of A4 pages

Commentary to calculation sheet

The worked example is based on the structure described in Sections 5.2 and 6.7, The column grid is 9 m x 9 m; the ribs are at 3 m centers and are continuous over two spans; the spine beams are also continuous over two spans.

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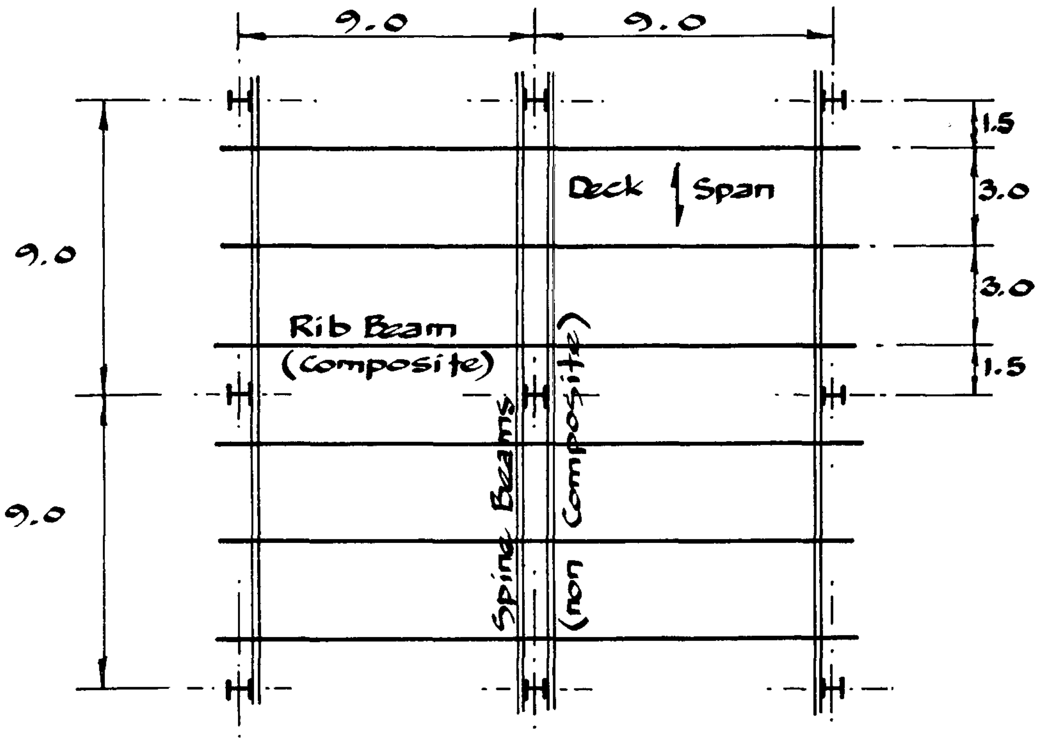
CALCULATION SHEET

Job No.		Sheet 1 of 46	Rev.
Job Title	Design Example		
Subject	Framing & Loads		
Client	SCI	Made by AY	Date Nov 89
		Checked by JAR	Date Dec 89.



TYPICAL LAYOUT

Loading	kN/m²
Construction Stage	
130mm. LW. Grade 30 Concrete Slab (wet)	= 2.35
Steel Self wt.	= 0.40
	2.75 kN/m²
construction load	0.50 "
Composite Stage	
Dead	
Raised floor	= 0.40
130mm. LW. Concrete Slab (Dry)	= 2.10
Ceiling & Services	= 0.40
Steel Self wt.	= 0.40
	3.30 kN/m²
Imposed	
Occupancy	= 5.0
Partitions	= 1.0
	6.0 kN/m²

Commentary to calculation sheet

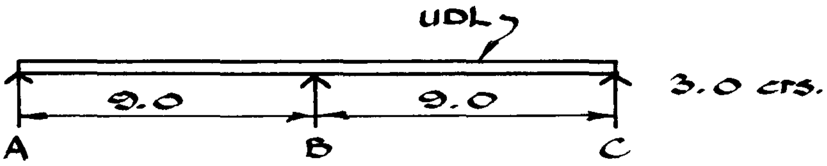
See Section 7.2

Discuss me ...

The Steel Construction Institute Silwood Park Ascot Berks SL5 7QN Telephone: (0990) 23345 Fax: (0990) 22944 Telex: 846843 **CALCULATION SHEET**	Job No.	Sheet 2 of 46	Rev.
	Job Title Design Example		
	Subject Rib Beam		
	Client SCI	Made by DLM. Date Nov 89	Checked by JAR Date Dec 89.

RIB BEAM

Two span continuous composite beam



Factored Loading

Dead = 3.30 × 1.4 = 4.62

Imposed = 6.00 × 1.6 = 9.60

14.22 kN/m²

Span Load = 14.22 × 9 × 3 = 384 kN.

Scheme Design

i) Section Dimensions:

minimum section depth = $\frac{9000}{30}$ = 300 mm.

minimum section width = $\frac{18000}{125}$ = 144 mm.

ii) Modulus requirement

Support Moment = $\frac{0.45 \times 384 \times 9}{8}$ = 194.4 kN.m

for Grade 43 S_x req'd = $\frac{194.4 \times 10^3}{275}$ = 707 cm³

iii) Section Classification

Only plastic sections to be used.

iv) Deflection

Assume span is unpropped during construction.

Assume composite inertia is two and a half times the basic steel beam inertia.

Discuss me ...

Commentary to calculation sheet

The following Table shows Universal sections that do not comply with "plastic section" criteria (see Section 6.6). The criterion for each section is shown in brackets.

Grade 43 d/t ≯ 49 b/T ≯ 8.5	
Beams (UB)	*Columns (UC)*
357 x 171 x 45 (b/T) *203 x 133 x 25 (b/T)*	*356 x 368 x 153 (b/T)* *356 x 368 x 129 (b/T)* *305 x 305 x 97 (b/T)* *254 x 254 x 73 (b/T)* *203 x 203 x 46 (b/T)* *152 x 152 x 23 (b/T)*
Grade 50 d/t ≯ 52.7 b/T ≯ 7.48	
914 x 305 x 201 (d/T & b/T) *838 x 292 x 176 (d/T & b/T)* *762 x 267 x 147 (d/T & b/T)* *686 x 254 x 125 (d/T & b/T)* *610 x 305 x 149 (b/T)* *610 x 229 x 101 (b/T)* *533 x 210 x 82 (b/T)* *457 x 152 x 52 (b/T)* *406 x 178 x 54 (b/T)* *406 x 140 x 39 (d/T & b/T)* *356 x 171 x 45 (b/T)* *305 x 165 x 40 (b/T)* *254 x 146 x 31 (b/T)* *203 x 133 x 25 (b/T)*	*356 x 368 x 177 (b/T)* *356 x 368 x 153 (b/T)* *356 x 368 x 129 (b/T)* *305 x 305 x 118 (b/T)* *305 x 305 x 97 (b/T)* *254 x 254 x 73 (b/T)* *203 x 203 x 52 (b/T)* *203 x 203 x 46 (b/T)* *152 x 152 x 30 (b/T)* *152 x 152 x 23 (b/T)*

Maximum deflection $= \dfrac{WL^3}{185EI} \not> \dfrac{L}{300}$

Minimum $I = \dfrac{300 \times 10^2}{185 \times 205} WL^2 = 0.791\, WL^2$

Where: W *is in kN*
L *is in m*
I *is in* cm^4

Comparison with an equivalent simple span rib
Applying similar loading criteria to a "simple" rib of equivalent span would suggest a rib of size 406 x 140 x 46kg/m UB Grade 43. This beam has the same weight as the continuous rib but is 100mm deeper and its deflection is about 8% higher. If a beam of the same depth is required, heavy weight penalties would occur. In this instance a 305 x 305 x 97 kg/m UC would be necessary.

Effect of slab reinforcement on the classification of steel sections
As derived in Section 6.6 ensure that the cross-section at internal supports is "plastic" by checking that the area of reinforcement in effective breadth of flange ≥ d.t/5

Section is plastic if:

$b/T < 8.50$ *and* $d/t < 59.0$ *For grade 43*

$b/T < 7.48$ *and* $d/t < 52.7$ *For grade 50*

$\dfrac{d.t}{5} = \dfrac{265.6 \times 6.7}{5}$ = *356 which is greater than 1.8 x 142*

$b/t = \dfrac{165.7}{11.8 \times 2}$ = *7.02 which is less than 8.5*

$d/t = \dfrac{265.6}{6.7}$ = *39.6 which is less than 59*

∴ Section is plastic

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CALCULATION SHEET

Job No.	Sheet 3 of 46	Rev.
Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: DLM	Date: Nov 89
	Checked by: JAR	Date: Dec 89

Assume construction load is taken by steel beam only, while remaining load is taken by the composite beam.

Equivalent loading on steel beam.

$$= 2.75 + \frac{(9.3 - 2.75)}{2.5} = 5.37 \text{ kN/m}^2$$

Considering the beam as a propped cantilever and limiting the deflection to span over 300.

I required in cm^4 units

$$= 0.791\, wL^2$$

$$\therefore I = 0.791 (5.37 \times 3 \times 9)\, 9^2$$
$$= 9290\, cm^4$$

Try 305 × 165 × 46 UB Grade 43

$S_{xx} = 722.7\, cm^3 > 707\, cm^3$ ok

$I_{xx} = 9948\, cm^4 > 9290\, cm^4$ ok

$D = 307.1$ mm > 300 mm. ok

$B = 165.7$ mm > 144 mm. ok.

$T = 11.8$ mm.

$p_y = 275\, N/mm^2$ $(11.8 < 16.0\, mm)$.

$t = 6.7$ mm
$d = 265.6$ mm.
$A = 58.9\, cm^2$
$r_y = 3.9$ cm.
$Z_x = 647.9\, cm^3$

Section ∼ Class 1, Plastic.

Commentary to calculation sheet

All Code Preferences for the detailed design for the construction stage, Sheets 4 - 7, are to BS 5950:Part 1.

Detailed design of ribs

The design is usually carried out assuming that the loaded areas and spans are measured between the centrelines of the pairs of spines. No allowance is made for the reduction in span due to the use of double spine beams or twin bracket supports to the spine beams. Hence the moments and shears used in the design are an over estimate of the actual effects on the rib. Further, as the design is plastic, no corrections are made to moments and shears for settlement of supports (see Section 6.7 for comparison of moments when settlement has been allowed).

Ultimate limit state for construction condition

The beam remains elastic and its strength during construction can be checked in the conventional way for continuous steel beams, using elastic analysis. The only particular feature is the restraint provided to the ribs by the attachment of the deck to the top flange. This is discussed in Section 6.2.1.

Where goal posts are provided as in Figure 5, the simplest approach is to use the distance between restraints to determine λ_{LT}. As a refinement, Appendix G of BS 5950:Part 1 may be used to take account of the restraint to the top flange from the deck.

Where goal posts are not provided, the design is only likely to be justified if Appendix G is utilised.

Serviceability Limit State for Construction Condition

Because of its possible impact on deflections, it is considered prudent not to permit any plasticity under unfactored loads during construction. In this case, this is clearly satisfied because the extreme fibre stresses at B under unfactored loads are:

$$\frac{71.2 \times 10^3}{647.9} \times \frac{(2.75 + 0.5)}{4.65} = 77 N/mm^2$$

Discuss me ...

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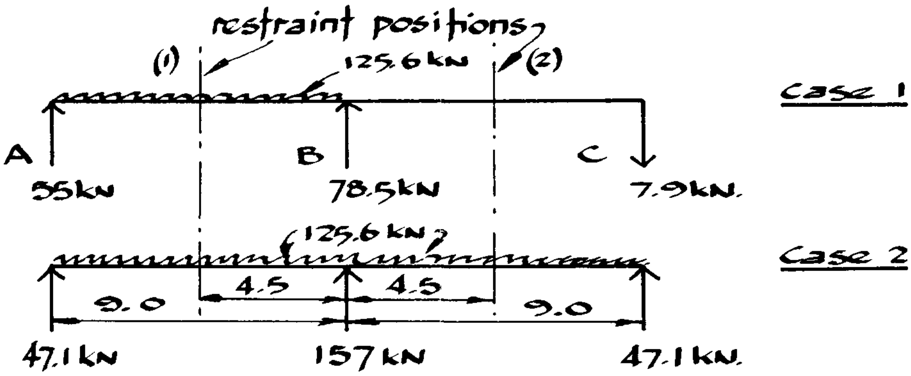
CALCULATION SHEET

Job No.		Sheet 4 of 46	Rev.
Job Title Design Example			
Subject Rib Beam			
Client SCI	Made by DLM	Date Nov 89	
	Checked by JAR	Date Dec 89.	

Construction Stage ~ Final Design

loading cases:



Factored loading

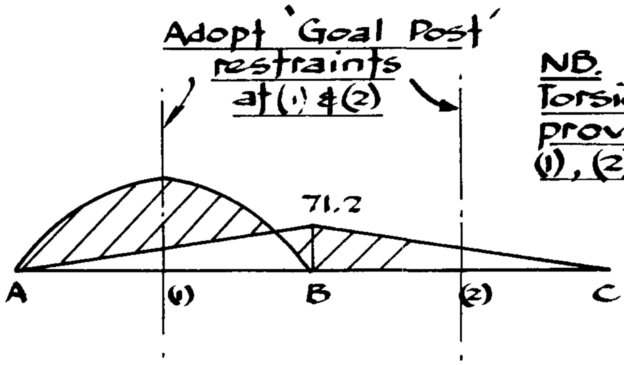
Dead $= 2.75 \times 1.4 = 3.85\ kN/m^2$

Constn $= 0.50 \times 1.6 = \underline{0.80}$

$\underline{4.65\ kN/m^2}$

$W = 4.65 \times 9 \times 3 = 125.6\ kN.$

Case 1 ~ Unloaded between B and C restrained at position 2



NB.
Torsional restraints provided at positions (1), (2) and supports.

BM at Supports B $= 0.063 \times 9 \times 125.6 = 71.2\ kN.m$

Span (Max) $= 0.096 \times 9 \times 125.6 = 108.5\ kN.m$

Beam unloaded between B & C

$\therefore$ use the 'm' approach

hence, $M_b = S_x \cdot p_b$.

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CALCULATION SHEET

Job No.	Sheet 5 of 46	Rev.
Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: RM	Date: Nov 89
	Checked by: JAR	Date: Dec 89.

To find p_b

$\lambda_{LT} = n.u.v.\lambda$ where: $n = 1.0$, $u = 0.89$

$\lambda = L_E/r_y = \frac{4.5 \times 10^3}{39} = 115.4$

$\lambda/x = \frac{115.4}{27.2} = 4.2$

∴ from Table 14 $v = 0.85$

∴ $\lambda_{LT} = 1.0 \times 0.89 \times 0.85 \times 115.4$

$= 87.3$

from Table 11 & using $p_y = 275 N/mm^2$

$p_b = 149 N/mm^2$

∴ $M_b = \frac{722.7 \times 149}{10^3} = 107.7 \text{ kN.m}$

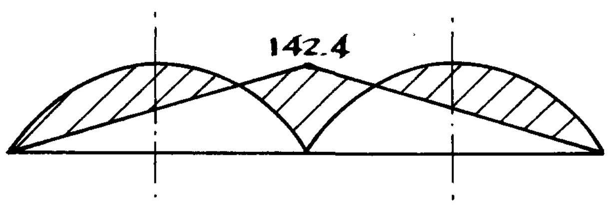
To find m

$\beta = 0.5$ from Table 18, $m = 0.76$

∴ $\bar{M} = 71.2 \times 0.76$

$= \underline{54.1 \text{ kN.m}} < \underline{107.7 \text{ kN.m}}$ OK.

Case 2 ~ Unrestrained between support B and restraints 1 and 2.



BM at Support B = $71.2 \times 2 = 142.4$ kN.m.

Span (Max.) = $0.07 \times 9 \times 125.6 = 79.1$ kN.m.

Span AB & BC loaded

∴ Use the 'n' approach

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CALCULATION SHEET

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Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: TDM.	Date: Nov 89
	Checked by: JAR	Date: Dec 89

To find n

firstly, determine M_o

$W = 125.6/2 = 62.8\,kN.$
$L = 4.5m.$
$M_o = 62.8 \times 4.5/8 = 35.3\,kN.m$

$\gamma = 142.4/35.3 = 4.0$

$\beta = -0.5$

$\therefore n = 0.7$ from Table 16

$\lambda_{LT} = 0.7 \times 0.89 \times 0.85 \times 115.4$
$= 61.1$

from Table 11 & using $p_y = 275\,N/mm^2$

$p_b = 211\,N/mm^2$

$\therefore M_b = \dfrac{211 \times 722.7}{10^3} = 152.5\,kN.m > 142.4\,kN.m$ OK

As these two bending moments are close, check for the high shear load condition cl. 4.2.6

$F_v = 157/2 = 78.5\,kN$ (see sheet 4)

$P_v = 0.6 \times 307.1 \times 6.7 \times 275/10^3 = 339.5\,kN.$

$0.6P_v = 0.6 \times 339.5 = 203.7\,kN > 78.5\,kN$

$\therefore$ Low Shear

$\therefore$ Continuous beam satisfactory for Lateral Torsional Buckling & Vertical shear in the Construction Stage.

Note: By inspection it can be seen that the support moment against the span moment is the design criterion.

Commentary to calculation sheet

Dead load deflection may need to be limited for aesthetic reasons, if the soffit is exposed, if thicknesses of levelling screed becomes excessive, or if a large range of adjustment becomes necessary on the raised flooring.

If the dead load deflection has to be reduced then either prop the ribs during concreting or choose a heavier / deeper rib.

For cumulative affects of deflection see Sheet 12.

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CALCULATION SHEET

Job No.	Sheet 7 of 46	Rev.
Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: DLM	Date: Nov 89
	Checked by: AR	Date: Dec 89

Construction stage Deflection

Loading kN/m²

130 L.W. Concrete slab = 2.35

Steel self wt = 0.40

2.75 kN/m².

$W = 2.75 \times 3 \times 9 = 74.3\text{kN}.$

$$\delta = \frac{74.3 \times 9000^3}{185 \times 205 \times 9948 \times 10^4}$$

$$= 14.4\text{mm}. \quad (1/625)$$

Commentary to calculation sheet

Ultimate limit state for completed structure

Since it has already been demonstrated that the section can provide, by rotation, the appropriate redistribution, plastic hinge analysis may be used at the ultimate limit state for the composite ribs. For uniform loading on an end span, the collapse mechanism consists of a hinge at the support and close to mid-span. The design should be so arranged that the hinge at the support forms first. Section properties at the support may be taken as for the plain section. The contributions of the mesh or bars of 10 diameter or under are ignored in determining the strength of the section.

Code requirements and methods of analysis

Plastic analysis will be adopted for the continuous beam design. The two span rib beam is symmetrical about its central support and loaded uniformly. The design procedure will be in accordance with Clause 5.2.4 plastic analysis. Clause 5.2.4 lays down conditions which have to be satisfied, they are as follows:

a. *The beam section is to be non-reinforced Class 1 plastic section; this is defined in Clause 5.2.1.2 and above.*

b. *Conditions (1) to (4) given in Clause 5.2.2 (simplified method) will also have to be satisfied.*

(1) *The steel beam should be of uniform section with equal flanges and without any haunches.*

(2) *The steel beam should be of the same section in each span.*

(3) *The loading should be uniformly distributed.*

(4) *The unfactored imposed load should not exceed 2.5 x the unfactored dead load.*

Alternatively the general plastic method given in Appendix D may be used.

As the first hinge will form at the central support, limit the support moment to the moment capacity of the steel beam. Check for high or low shear condition of central support. This is a check on the influence of shear on moment capacity. Once this fixing moment has been established the rest of the moment and shear diagram can be constructed. Determine span moment and compare with moment capacity. Finally check the effects of shakedown (Clause 6.1.3.3).

Discuss me ...

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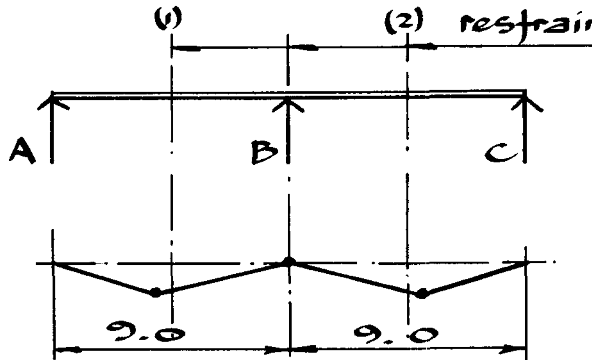
CALCULATION SHEET

Job No.		Sheet 8 of 46	Rev.
Job Title Design Example			
Subject Rib Beam			
Client SCI	Made by DRM	Date Nov 89	
	Checked by JAR	Date Dec 89	

Composite Rib ~ Detailed Design



Plastic Hinge Diagram

Loading

Slab	2.10 kN/m²
Steel	0.40 "
	2.50 kN/m²
Raised floor	0.4
C&S	0.4
Dead (Imposed)	0.8 kN/m²
Imposed	6.0 kN/m²

At support B, consider steel beam only
In the span, composite section.

∴ firstly, calculate the plastic moment capacity of the steel beam, M_s

$M_s = 722.7 \times 10^3 \times 275 / 10^6 = 198.7$ kN.m.

Factored loading/span

$= [(2.5+0.8)1.4 + (6 \times 1.6)] 9 \times 3 = 384$ kN.

Shear force at support B, F_v

$F_v = \frac{384}{2} + \frac{198.7}{9} = 214.1$ kN.

Shear capacity, P_v

$P_v = 339.5$ kN (see sheet 6).

$0.5P_v = 0.5 \times 339.5 = 169.8$ kN < 214.1 kN over

∴ High shear load hence, moment capacity has to be reduced

As the steel beam alone is being considered use BS 5950: Part 1 to determine the reduction in moment capacity.

Commentary to calculation sheet

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CALCULATION SHEET

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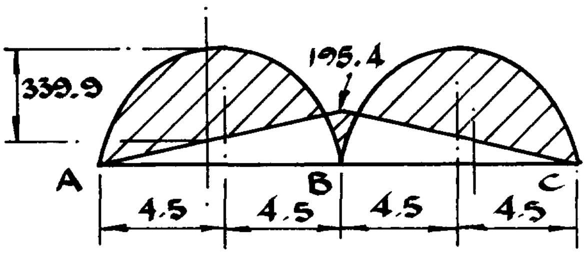
Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: DM	Date: Nov 89
	Checked by: [illegible]	Date: Dec 89.

$$M_S = p_y\,(S_x - S_v\rho_1)$$

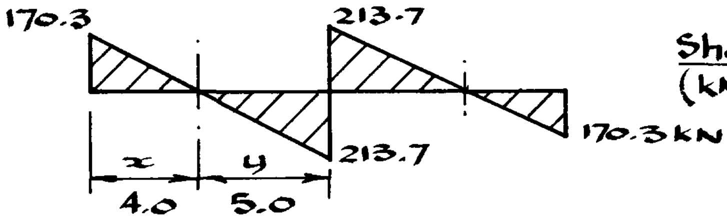
$$\rho_1 = \frac{2.5F_v}{P_v} - 1.5 = \frac{2.5\times 214.1}{339.5} - 1.5 = 0.0766$$

$$S_v = \frac{td^2}{4} = 0.67\times 30.71^2/4 = 158\ \text{cm}^3$$

$$M_S = \frac{275}{10^3}\left[722.7 - 158\times 0.0766\right] = 195.4\ \text{kN.m.}$$



Moment Diagram (kN.m).



Shear Diagram (kN).

Free moment = $384\times 9/8$ = 432 kN.m.

Shear at support B = $\frac{384}{2} + \frac{195.4}{9}$ = 213.7 kN.

Position of zero shear

$x/170.3 = y/213.7$ but $y = 9 - x$

$\therefore\ x/170.3 = \frac{9-x}{213.7}$

$213.7x = (9-x)\,170.3$ $\quad x = 4.0$m & $y = 5.0$m.

∴ Max. Span moment

$= 170.3\times 4 - 384\times 4/9\times 2$

$= \underline{339.9\ \text{kN.m.}}$

Commentary to calculation sheet

Determination of cross-section classification and properties

Effective width of concrete flange

In any one span of a continuous composite beam it is necessary to determine the effective concrete flange breadth. As shown in Figure 21, this is taken as 0.25 times the effective span, but not exceeding the beam spacing. The effective span is defined in Part 3.1 of BS 5950 and these provisions are summarised in the Table below:

Position	*Effective Span*
End Span	*0.8 x Span*
Internal Support	*0.25 x (Sum of adjacent Spans)*
Internal Span	*0.7 x Span*
Span adjacent to Cantilever	*0.8 x Span - 0.3 x Cantilever (but ≥ 0.7 x Span)*
Support adjacent to Cantilever	*1.5 x Cantilever (but $\not>$ 0.5 x Span)*
Cantilever	*1.5 x Cantilever (but $\not>$ 0.5 x Span)*

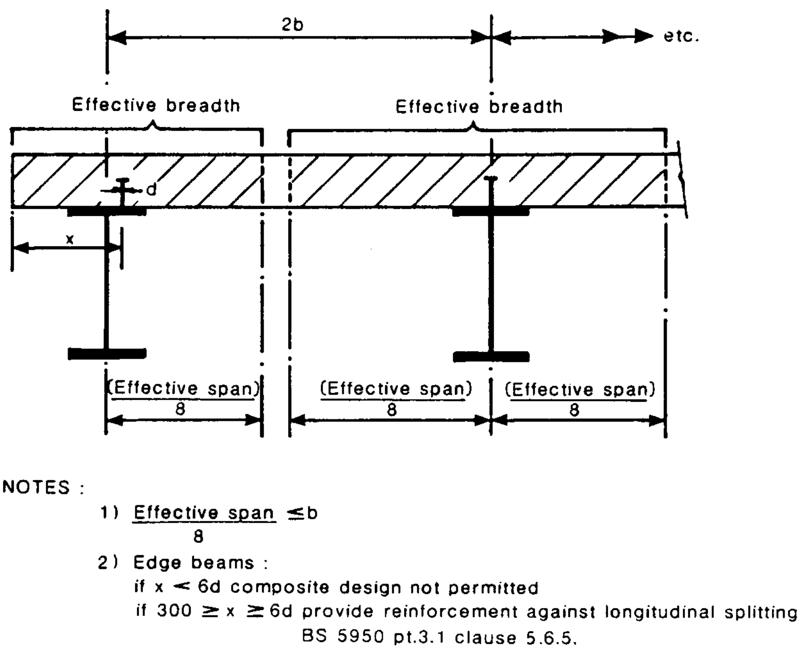


Figure 21 *Effective breadth of concrete flange*

Refer to Clause 6.1.3.3 of BS 5950:Part 3.1

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CALCULATION SHEET

Job No.	Sheet 10 of 46	Rev.
Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: DLM.	Date: Nov 89.
	Checked by: JAR	Date: Dec 89.

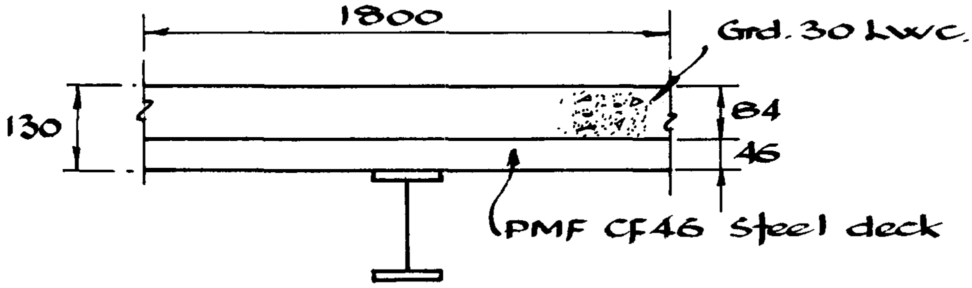
Moment Capacity (positive) based on full shear connection using BS.5950: Part 3.1

Effective width of compression flange, B_e

$L_z = 0.8L = 0.8 \times 9 = 7.2\text{m}$.

$\therefore B_e = 7.2 \times 2/8 = 1.8\text{m} < 3.0\text{m}$ crs.

Effective width = 1800 mm.



$F_c = 0.45 \times 30 \times 1800 \times 84/10^3 = R_c = 2041.2\text{ kN}$.

$F_s = 58.9 \times 275/10 \qquad = R_s = 1619.8\text{ kN}$.

Case 2(b) ~ Appendix B, BS.5950: Part 3.1

$R_s \leq R_c$

$$M_c = R_s\left[\frac{D}{2} + D_s - \frac{R_s}{R_c}\left(\frac{D_s - D_p}{2}\right)\right]$$

$$= \frac{1619.8}{10^3}\left[\frac{307.1}{2} + 130 - \frac{1619.8}{2041.2} \times \frac{84}{2}\right]$$

$= 405.3\text{ kN.m} > 339.9\text{ kN.m}$ ok

$\therefore$ Positive span moment capacity based on full shear connection satisfactory.

The effects of shakedown on deflection and pattern loading will have to be allowed for (see sheet 12).

Discuss me ...

Commentary to calculation sheet

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CALCULATION SHEET

Job No.	Sheet 11 of 46	Rev.
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Subject: Rib Beam		
Client: SCI	Made by: DLM.	Date: Nov 89
	Checked by: JAP	Date: Dec 89.

Composite Section Properties

$$A = 58.9 \times 10^2 \text{ mm}^2 > \frac{84^2 \times 1800}{(307.1 + 2 \times 46)15} = 2122 \text{ mm}^2.$$

$\therefore$ Elastic neutral axis in steel member

$$y_g = \frac{58.9 \times 10^2 \times 15(307.1 + 2 \times 130) + 1800 \times 84^2}{2[58.9 \times 10^2 \times 15 + 1800 \times 84]}$$

$$= 131 \text{ mm}.$$

Uncracked Inertia of the composite section, I_g

$$I_g = 9948 + \frac{180 \times 8.4^3}{12 \times 15} + \frac{58.9 \times 180 \times 8.4(30.71 + 13 + 4.6)^2}{4[58.9 \times 15 + 180 \times 8.4]}$$

$$= \underline{32232 \text{ cm}^4.}$$

$$Z_{conc.} = \frac{32232 \times 15}{13.1} = \underline{36907 \text{ cm}^3.}$$

$$Z_{steel} = \frac{32232}{(30.71 + 13 - 13.1)} = \underline{1053 \text{ cm}^3.}$$

Note: modular ratio for LWC = 15
(see cl 4.1 part 3.1).

Commentary to calculation sheet

Note there is a need for engineering judgement in the total deflection permitted. In this case the imposed rib deflection is limited to not more than 20mm and a total of L/360 (35mm for a 9 x 9 grid) on the diagonal when the spine deflection has to be taken into account.

Partition loading is included, where deflections will occur as they are erected. In cases where brittle masonry or similar deflection sensitive finishes are supported, smaller deflection limits should be considered.

The total rib deflection can be summarised as:

(i) Construction stage deflections (sheet 7) 14.4mm.

(ii) Composite stage deflection (including dead load after construction) (sheet 12) 17.4mm.

(iii) Extra composite stage deflection for partial interaction effect (sheet 18) 0.9mm.

These deflections take into account pattern loading, inelastic behaviour and the effects of partial interaction. For cumulative floor deflections the spine deflections must also be considered.

Consideration of shakedown
From Sheet 9, the elastic midspan moment based on the uncracked elastic analysis is 432Kn.m.

The % redistribution of support moments $= \dfrac{432 - 195.4}{432} \times 100 = 55\%$

Clause 6.1.3.3 generally requires treatment of shakedown where plastic analysis is used for the ULS. However, if it could be demonstrated that the redistribution was less than or equal to 40%, (even if the structure was originally analysed plastically) as for the elastic uncracked section, then it would not be necessary to consider shakedown.

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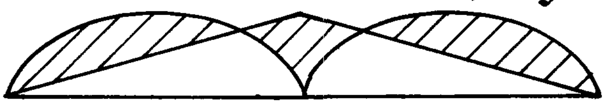
CALCULATION SHEET

Job No.	Sheet 12 of 46	Rev.
Job Title: Design Example		
Subject: Rib Beam		
Client: SCI	Made by: DM	Date: Nov 89
	Checked by: JAR	Date: Dec 89.

Serviceability

Deflection ~ Unpropped Construction

246 kN.m > M_s = 195.4 kN.m



Shakedown Effect

see commentary note

$W_{sh} = W_d + 0.8W_i$

where: $W_d = 3.3 \times 9 \times 3 = 89.1$ kN

$W_i = 6.0 \times 9 \times 3 = 162.0$ kN

$W_{sh} = 89.1 + 0.8 \times 162 = 218.7$ kN.

$M_{sh} = 218.7 \times 9/8 = 246$ kN.m

$M_s = 195.4$ kN.m

$M_{excess} = M_{sh} - M_s$
$= 246 - 195.4$
$= 50.6$ kN.m

Allowance for Pattern Loading

$M_i = 162 \times 9/8 = 182.3$ kN.m

M_i (pattern) $= 182.3 \times 0.7 = 127.6$ kN.m

$M_{support} = 127.6 - 50.6 = 77.0$ kN.m $= M_1$

$M_2 = 0$

$M_o = M_i = 182.3$ kN.m

$$\delta_o = \frac{5 \times 162 \times 9000^3}{384 \times 205 \times 32232 \times 10^4} = 23.3 \text{ mm}$$

$$\delta_c = 23.3\left[1 - 0.6(77 + 0)/182.3\right]$$

$= \underline{17.4 \text{ mm}}$ $(1/517 < 1/360)$ ok.

Discuss me ...

Commentary to calculation sheet

Serviceability considerations

One of the principle advantages of BS 5950: Part 3.1 for composite construction is that it permits some yielding at supports at the serviceability limit state. However it is still necessary to check that there is a reserve against yield in the span.

Dead load stresses are directly available from the elastic analysis of the construction condition. Imposed load stresses are determined by analysing the structure under imposed loading with net support moments of section plastic moment, minus dead load moment.

Elastic deflections of continuous composite beams can be calculated in the conventional manner using a modular ratio that reflects the relative proportions of long and short term live loading (see Clause 4.1 of BS 5950:Part 3.1). Where partial shear connection is adopted its incremental effect on deflection should be included (see Clause 6.1.4 of BS 5950:Part 3.1). Pattern loading effects should be considered as in Clause 6.1.3.2 of BS 5950:Part 3.1.

Section 6.5 discusses the additional deflections that may occur if there is plasticity in support regions on first application of full working live load. As noted there, an appropriate procedure is outlined in Clause 6.1.3.3 of BS 5950:Part 3.1.

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CALCULATION SHEET

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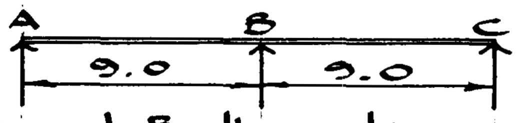
Job Title: Design Example			
Subject: Rib Beam			
Client: SCI	Made by DLM	Date Nov 89	
	Checked by JAR	Date Dec 89.	

Serviceability Stresses



At support B the extreme tensile fibre stress will be based-on the plastic section modulus, S_x. The reason for this is that redistribution of moment is permitted in the serviceability state provided p_y is not exceeded in the span.

ie.

first hinge

These hinges not allowed to form. Limit steel stress to p_y and use elastic section properties for the span.

Construction Load

Slab = 2.1
Self wt: = 0.4
2.5 kN/m²

$W = 2.5 \times 9 \times 3 = 67.5$ kN

Support moment @ B = $67.5 \times 9/8$ = 76 kN.m

Max. permitted moment at support B
= 195.4 kN.m (see sheet 9).

∴ Composite Stage moment at Support B

= 195.4 − 76 = 119.4 kN.m.

Discuss me ...

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CALCULATION SHEET

Job No.

Sheet 14 of 46

Rev.

Job Title: Design Example

Subject: Rib Beam

Client: SCI

Made by: DRM

Date: Nov 89

Checked by: JAR

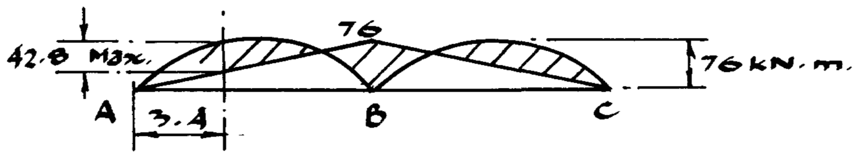
Date: Dec 89.

Imposed Load $= 6.0\ kN/m^2$

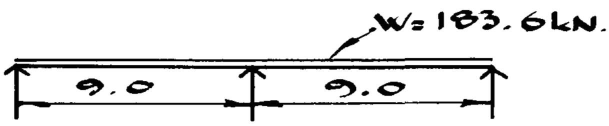
Dead " $= 3.3 - 2.5 = 0.8\ kN/m^2$

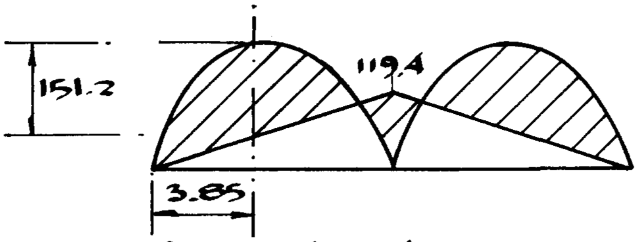
$\therefore W = (6.0 + 0.8) 9 \times 3 = 183.6\ kN.$

free moment $= 183.6 \times 9/8 = 206.6\ kN.m.$



Construction Stage Moments





Composite Stage Moments.

Span Stress (Construction)

$= \frac{42.8 \times 10^3}{647.9} = 66\ N/mm^2.$

Span Stress (composite)

$= \frac{151.2 \times 10^3}{1053} = 143.6\ N/mm^2.$

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CALCULATION SHEET

Job No.		Sheet 15 of 46	Rev.
Job Title: Design Example			
Subject: Rib Beam			
Client: SCI	Made by: DM	Date: Nov 89	
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Combined extreme tensile fibre stress

NB. The positions of maximum moment do not coincide but for simplicity add the stresses and check against py, this will be conservative.

$66 + 143.6 = \underline{209.6 \text{ N/mm}^2} < py = \underline{275 \text{ N/mm}^2}$ ok.

$\therefore$ Span Moment remains Elastic.

Concrete Stress

$= \dfrac{151.2 \times 10^3}{36907} = \underline{4.1 \text{ N/mm}^2} < 0.5 f_{cu} = \underline{15 \text{ N/mm}^2}$ ok.

$\therefore$ Serviceability stresses satisfactory.

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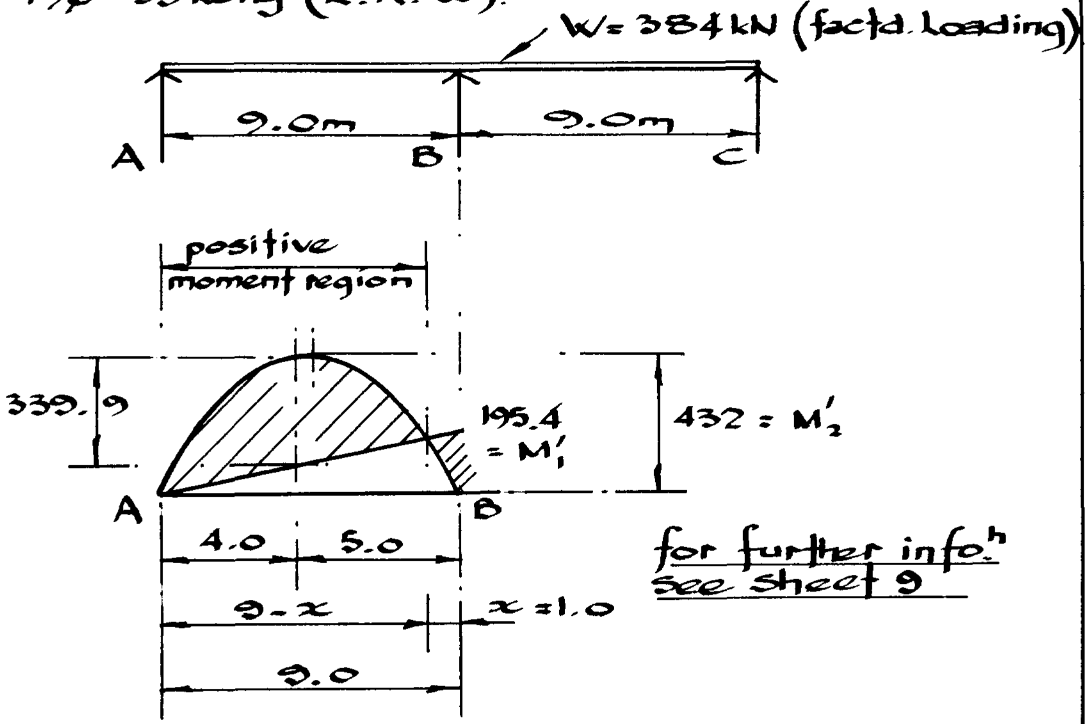
Subject: Rib Beam		
Client: SCI	Made by: DM	Date: Nov 89
	Checked by: AR	Date: Dec 89

Shear Stud Connectors

19ø × 95 long (L.A.W).



for further info.ⁿ see sheet 9

To find x

$$y = \frac{4 . M_2' . x}{L^2}(L - x) \quad \ldots (1)$$

$$\frac{y}{L - x} = \frac{M_1'}{L} \quad \therefore y = \frac{M_1'}{L}(L - x) \quad \ldots (2)$$

put (1) = (2) $\quad x = \frac{L}{4} \cdot \frac{M_1'}{M_2'}$

$$= \frac{9 \times 195.4}{4 \times 432} = 1.0m.$$

Stud Strengths ~ positive moment region
Grd. 30 LWC.
from Table 5, $Q_k = 100kN$.

Q_k for LWC = 100 × 0.9 = 90 kN

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$Q_b = 0.8 \times 90 = 72.0$ kN.

Deck Geometry
Ribs perpendicular to the beam

Assume $k = 1.0$ for wide trough profile

∴ No reduction in stud value
ie. $Q_p = 72.0$ kN.

from sheet 10

$F_c = R_c = 2041.2$ kN.

$F_s = R_s = 1619.8$ kN. governs

Required studs for full shear connection:

$N_p = \dfrac{1619.8}{72} = 22.5$ Say 23

Check stud (available) positions over $8/2 = 4$ m.

trough deck crs = 225 mm.

Available positions $= 4000/225 = 17.8$ Say 18

but 23 req'd for full shear connection.

∴ Consider partial shear connection.

$\dfrac{N_a}{N_p} = \dfrac{18}{23} = 0.78 > 0.4$ ok

∴ As partial shear connection will be adopted the imposed deflection will require revision.

Discuss me ...

Commentary to calculation sheet

Because of the obvious difficulties in identifying the negative moment regions of the rib beam after decking installation, it is normal practice to specify the same stud spacing across the whole span. The few additional studs are of minor cost.

Discuss me ...

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CALCULATION SHEET

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Subject: Rib Beam		
Client: SCI	Made by DM	Date Nov 89
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δ_s: deflection of steel beam alone

$$= \frac{WL^3}{185EI} = \frac{162 \times 9000^3}{185 \times 205 \times 9948 \times 10^4} = 31.3\text{ mm.}$$

$\delta_c = 17.4\text{mm}$ (see sheet 12).

$$\delta = 17.4 + 0.3(1 - 0.78)(31.3 - 17.4)$$

$$= 18.3\text{mm} \left(1/492\right) < \left(1/360\right) \text{ ok.}$$

Check revised moment capacity for partial shear connection

from Appendix B.

$R_q = NQ = 72 \times 18 = 1296\text{kN}.$

$R_w = 544.4\text{ kN} (1619.8 - 2 \times 537.7)$

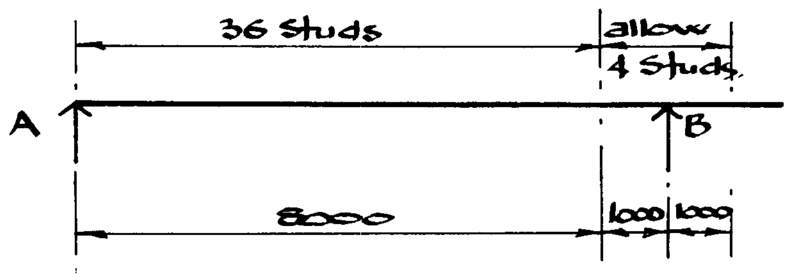
$R_q > R_w$ p.n.a. in flange

$$M_c = \frac{1619.8}{10^3} \times \frac{307.1}{2} + \frac{1296}{10^3}\left[130 - \frac{1296}{2041.2} \times \frac{84}{2}\right] - \frac{(1619.8 - 1296)^2}{537.7 \times 10^3} \times \frac{11.8}{4}$$

$$= \underline{382\text{ kN.m}} > \underline{339.9\text{ kN.m}} \text{ ok.}$$

$\therefore$ Positive span moment capacity for partial shear connection satisfactory.

Stud layout per span



Commentary to calculation sheet

Refer to Commentary to Calculation Sheet 4 for a discussion on stability of the continuous composite rib. The critical case occurs when one span only is loaded, giving the greatest length of bottom flange in compression in the neighbouring span.

The stability of the ribs in the completed structure may be demonstrated as follows:

1) *At the supports, torsional restraint is provided by the combined action of the composite slab, the bolted connection of the bottom flange to the spine beams and the interlinking of the spines to the slab via the column and support brackets. A stiffener may be required as discussed in Commentary to Calculation Sheet 23.*

2) *As discussed in Section 6.2.2, it is possible to take account of the torsional restraint from the slab. The relevant maximum unbraced length (L_t) is obtained from Appendix G2 of BS 5950:Part 1. L_t will generally be less than the full span. The Commentary to Calculation Sheet 21 demonstrates the means by which the designer may check that the slab, acting in conjunction with the shear connection and web influence, may provide an equivalent restraint within the span. The method is conservative because the encastré effect of twin spines at the continuous rib support provides an in-plan stiffness for the rib bottom flange which is not modelled in the connection (see Figure 11).*

3) *Alternatively, if goal posts have been provided for the construction condition or to limit differential deflections, they may be used to satisfy the L_t criteria of 2 above.*

In this example the rib is first checked ignoring the continuous torsional restraint from the slab and the goal posts.

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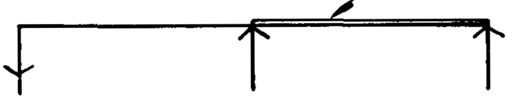
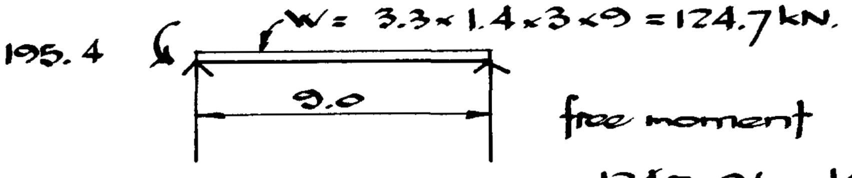
Lateral Stability ~ Composite Stage
(for further information see Reference 12)

1.4× Dead only

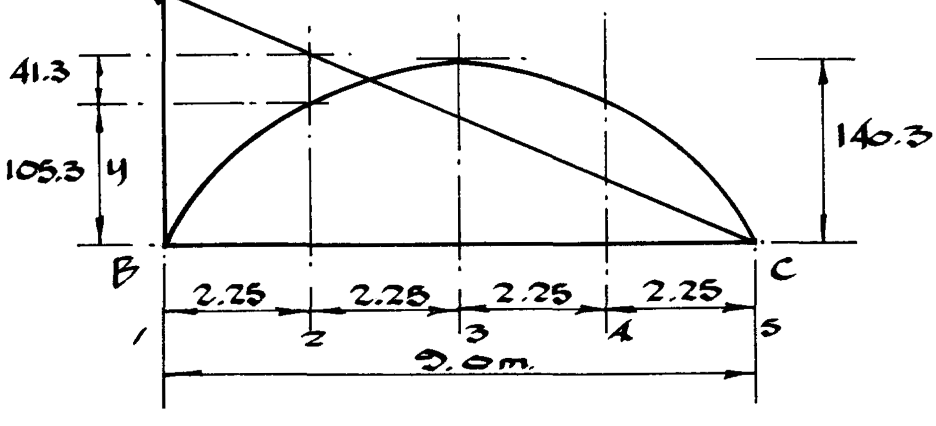




free moment

$= 124.7 \times 9/8 = 140.3$

195.4 (see sheet 9) moments in kN.m.



$$y = 4 \times \frac{140.4 \times 2.25}{81}(9-2.25) = 105.3 \text{ kN.m.}$$

$$\text{mt. at position 2} = \frac{195.4 \times 6.75}{9} - 105.3 = 41.3 \text{ kN.m}$$

	1	2	3	4	5
N	195.4	41.3	Negative		
M	195.4	195.4	N/A		

$$n_t = \left[\frac{1}{12}\left(\frac{195.4}{195.4} + \frac{3 \times 41.3}{195.4}\right)\right]^{1/2} = 0.369$$

Commentary to calculation sheet

In this instance the maximum permitted unbraced length (L_t) is very close to the net span of the rib and the design is therefore seen as satisfactory without any torsional restraint from the slab or recognition of the goal posts.

However, for purposes of illustration of Section 6.2.2, the means by which the slab may be mobilised as an equivalent discrete torsional restraint is demonstrated in the following section of the worked example.

Use of slab to provide torsional restraint within the span

Codified values for restraint forces are in a state of flux. Until more definitive guidance is available it seems appropriate to use a conservative value of 3% of restrained force as a basis for checking the strength of the restraint system.

The designer should determine the minimum length of web necessary to develop this restraint force, acting as a cantilever from the top flange and remaining elastic. He should also determine the minimum length of shear connection that is necessary to transmit this moment into the slab. The critical factor is the pull out strength of the shear connection, assuming a lever arm of half the width of the flange. This is illustrated in Figure 22 (Commentary to Calculation Sheet 22).

Providing both these lengths are significantly less than the maximum unbraced length (say less than a half) it is clear that an equivalent discrete restraint can be developed from the torsional restraint provided by the slab to the top flange of the beam.

Since both the length of web (2287mm) and the length of shear connection (1125mm) required to develop the necessary torsional restraint are small in relation to the beam span and less than 50% of the maximum unbraced length it is clear that the system has sufficient strength to develop the necessary restraining torsion.

Discuss me ...

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$$L_k = \frac{\left(5.4 - 600 \times {}^{275}/_{205000}\right) 39 \times 27.2}{\left(5.4 \times {}^{275}/_{205000} \times 27.2^2 - 1\right)^{1/2}}$$

$$= 3152\text{ mm.}$$

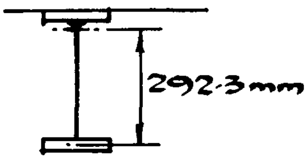
$$L_t \leq \frac{L_k}{C_{nt}} = \frac{3152}{1 \times 0.369} = 8543\text{ mm.}$$

The column centres minus the eccentricities at the supports are similar to the value of $\left({}^{L_k}/_{C.\eta_t}\right)$ which in practice would probably be accepted.

Max. flange force $= 165.7 \times 11.8 \times {}^{275}/_{10^3} = 538$ kN.

Required restraint force $= 3\% \times {}^{538}/_{10^2} = 16.1$ kN.

by inspection, design criteria:



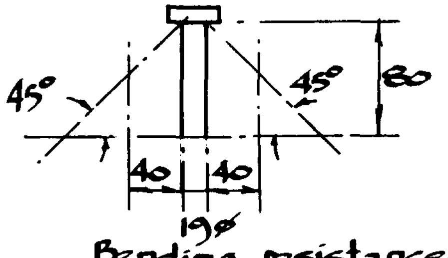
= Top of web in bending

Min. length of web to develop restraint force

$307.1 - (307.1 - 265.6 - 11.8)\,{}^{1}/_{2} = 292.3$

$$= \frac{16.1 \times 292.3 \times 6}{{}^{275}/_{10^3} \times 6.7^2} = 2287\text{mm. OK compared to } {}^{L_k}/_{C\,\eta_t}$$

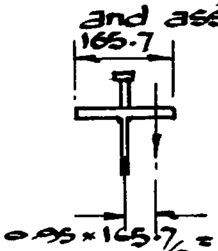
Resistance to uplift for the shear connection is given by:-



$80 \times 9.9 \times \pi \times 0.46/10^3 = 11.4$kN

basic shear strength of concrete $= 0.46$ N/mm^2

Bending resistance of one shear connector and associated beam flange is given by:-



$11.4 \times 78.7 = 897$ kN.mm.

Min. number of shear connectors to develop resistance force:

$$= \frac{17.4 \times 292}{897} = 6$$

$\therefore$ beam length $= (6-1) \times 225$
$= 1125$mm OK

$0.95 \times 165.7/_2 = 78.7$mm.

Discuss me ...

Commentary to calculation sheet

The stiffness of this restraint system should also be checked. BS 5400:Part 3 Clause 9.6.6.2 gives a method for determining the stiffness of such a restraint system.

$$\delta = P\left[\frac{d_1{}^3}{3EI_1} + \frac{u.Bd_2{}^2}{2EI_2}\right]$$

Where

d_1 = *the distance from the centroid of the compression flange to the root line on the web.*
u = *0.33 for multi-beam situations.*
B = *the rib spacing.*
d_2 = *the distance from the centroid of the compression flange to the mid-depth of the floor slab.*
I_1 = *the second moment of area of the web per unit length.*
I_2 = *the second moment of area of the floor slab per unit length.*
P = *the required restraint force per unit length*
δ = *the deflection from the restraint force.*

As discussed in Section 6.6.2, draft Eurocode 3 gives an appropriate criterion for stiffness. This states that the deflection under 3% of the force to be restrained should not exceed span/1700 without account being taken of the restraint flexibility.

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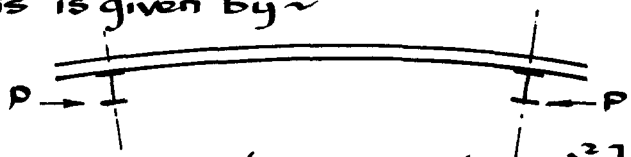
Lateral deflection of bottom flange under a load of P/unit length, this is given by ~

$$\delta = P\left[\frac{d_1^3}{3EI} + \frac{u.B d_2^2}{2EI_2}\right]$$



$$\delta = P\left[\frac{292^3}{3\times205000\times\frac{6.7^3}{12}} + \frac{0.33\times3000(307.1-11.8/2+65)^2}{\frac{205000}{21}\left(\frac{130^3+84^3}{12\times2}\right)2}\right]$$

$$\delta = P[1.62 + 0.06]$$

$$\delta = 1.68P$$

Assume restraint force is distributed over entire span

$$\delta = \frac{1.68\times16.1\times10^3}{9000} = 3.00\text{ mm. } (\text{Span}/3000)$$

from Eurocode 3 ~ Design of steel structures (Draft).

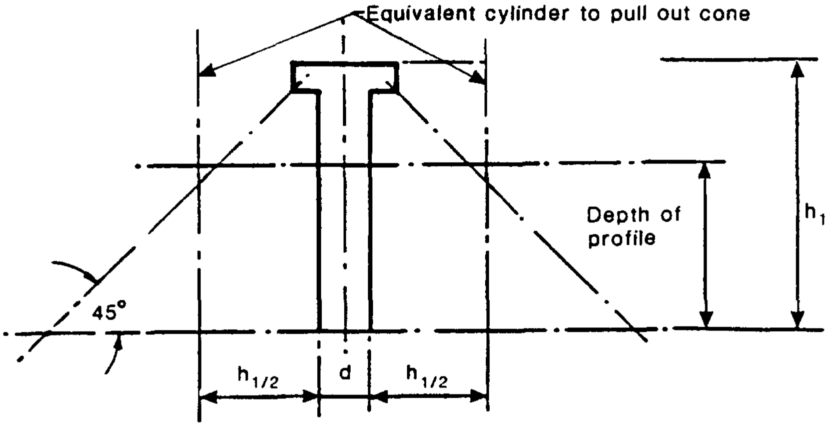
Part 1 ~ General Rules and Rules for buildings.

Fig. 5.2.5 - Single restrained member

this clause proposes a limit of L/1700

∴ Stiffness Satisfactory

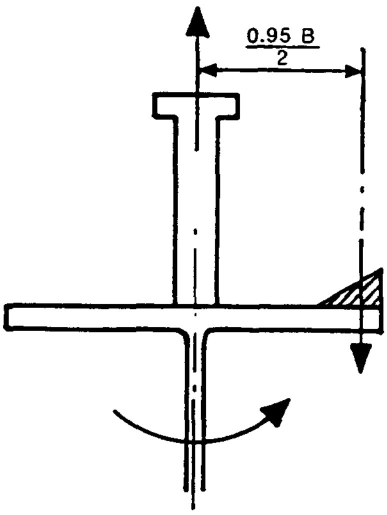
Commentary to calculation sheet



Pull out resistance of shear connector =
Net area of equivalent cylinder x design shear resistance of concrete

Net area of equivalent cylinder =
$\pi\, h_1(h_1+d)$ - loss of area from deck ribs

(a) Pull out resistance of shear connections



(b) Lever arm for transmission of moment into slab

Figure 22 *Details of transfer of restraint moment into slab*

BS 5950:Part 3.1, Clause 5.6.4, permits some account to be taken of the decking as transverse reinforcement, providing it is either continuous across the top flange of the steel beam or it is welded to the steel beam by the shear connectors.

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Transverse Reinforcement

Shear to be resisted, v

$$v = \frac{NQ}{s} = \frac{1.0 \times 72}{0.225} = 320 \text{ kN/m}$$

Resistance of concrete flange, v_r

$$v_r = 0.7 A_{sv} \cdot f_y + 0.03 \eta \cdot A_{cv} \cdot f_{cu} + v_p$$

but $(0.7 A_{sv} \cdot f_y + 0.03\eta \cdot A_{cv} \cdot f_{cu}) \leq 0.8 \eta \cdot A_{cv} (f_{cu})^{1/2}$

where:

f_{cu} = 30 N/mm² (LWC).
η = 0.8 for LWC.
A_{cv} = 84×10^3 mm²/m for one shear plane
A_{sv} = 142 mm²/m Mesh -do-
v_p = steel deck contribution

Contribution of profiled steel sheeting, v_p.

Deck perpendicular to the beam : $n = 5$

$v_p = N/s \,(n d t_p \cdot p_{yp})$ but $\leq t_p \cdot p_{yb}$

$= 1/225 \times 5 \times 19 \times 1 \times 280 = 118.2$ kN/m.

$t_p \cdot p_{yb} = 1 \times 280 = 280$ kN/m.

$\therefore v_p = 118.2$ kN/m.

$0.7 A_{sv} \cdot f_y = 0.7 \times 142 \times 460 / 10^3 = 45.7$ kN/m.
$0.03\eta A_{cv} \cdot f_{cu} = 0.03 \times 0.8 \times 84 \times 10^3 \times 30 / 10^3 = 60.5$ "

for one shear plane = 106.2 kN/m
" two " " = 212.4 "

$2 \times 0.8 \eta A_{cv} (f_{cu})^{1/2} = 0.8 \times 0.8 \times 84 \times 10^3 \times (30)^{1/2} \times 2 = 589$ kN/m.

$\therefore 212.4 < 589$ kN/m ok

$\therefore$ Total shear contribution

$= 2 \times 118.2 + 212.4$

$= \underline{448.8 \text{ kN/m}} > v = \underline{320 \text{ kN/m}}$ ok.

A142 Mesh satisfactory for transverse reinforcement

No additional reinforcement required.

Discuss me ...

Commentary to calculation sheet

Stiffening at internal supports

Section 6.3 gives a detailed discussion of the criteria for stiffening at internal supports. For economy, stiffeners should be avoided if possible.

The designer should seek to demonstrate either by the use of Table 1 or by other detailed elastic analysis of the floor system (as for example illustrated in Section 6.7), that this region does not become a plastic hinge at the ultimate limit state. For the reasons discussed in Section 6.3 this is quite likely to be the case even though the ribs were initially sized by plastic analysis.

If the web remains elastic its adequacy without stiffening may be checked by conventional elastic criteria in accordance with Clauses 4.5.2 - 4.5.5 of Part 1 of BS 5950.

If a plastic hinge forms at the support a stiffener will be required to comply with Clause 5.3.6 of Part 1 of BS 5950.

Stiffeners are only provided on one side of the rib to save fabrication costs.

In the particular example shown, $\frac{I_g}{I_s} = \frac{32232}{9948} = 3.24$

Using Table 1 for a bay 9 x 9m indicates a limiting redistribution factor of 0.508. The actual redistribution exceeds this value hence because the web is not elastic, a web stiffner is required.

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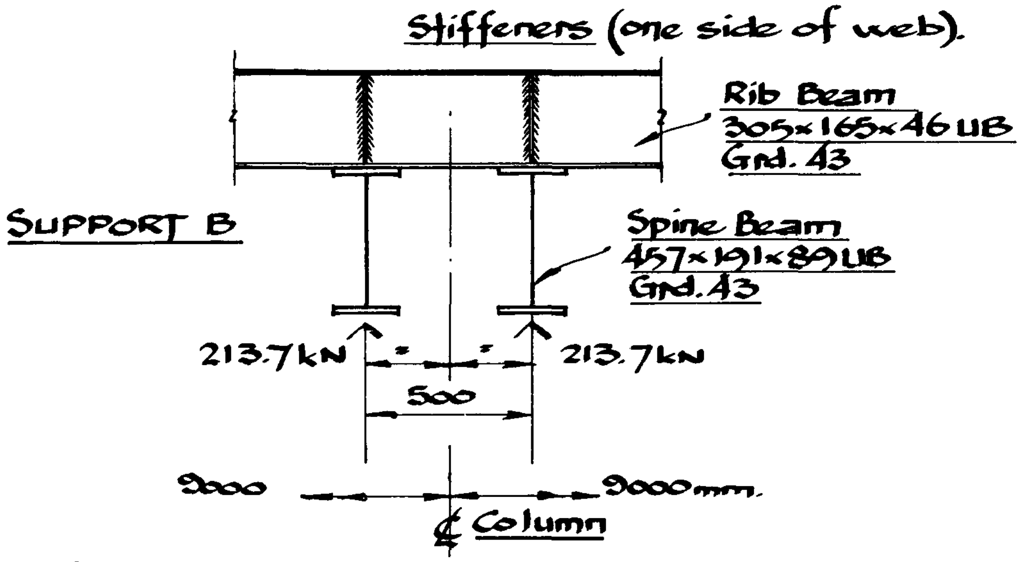
The procedures given in 6.3 (Table 1) of the text show that the rib beam will require stiffening. Therefore, design the stiffeners in accordance to BS 5950: Part 1 (cl. 5.3.6).

R_v = 339.5 kN (sheet 8)

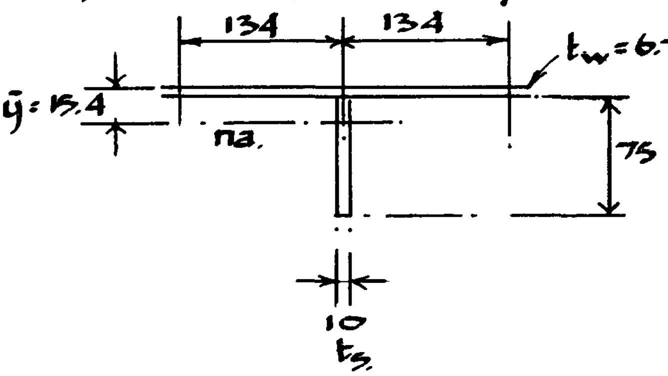
$0.1R_v$ = 34 kN < 213.7 kN. ∴ provide stiffeners.



for loading data see sheet 9

Stiffener sizing

Stiffeners are required either side of the hinge at a distance no greater than D/2, for practical purposes place over spine web.



$b_s/t_s \not> 9$

∴ Try t_s = 10

Commentary to calculation sheet

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to find cg:

$$(134\times 2)\frac{6.7^2}{2} + 75\times 10\times 44.2 = \left[(75\times 10)+(6.7\times 2\times 134)\right]\bar{y}$$

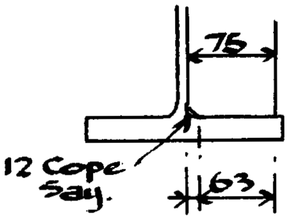
$$\bar{y} = 39165/2546 = 15.4\text{mm.}$$

$$I_{xx} = \frac{10\times 66.3^3 + 2\times 134\times 15.4^3 - 258\,(15.4-6.7)^3}{3}$$

$$= 1.24\times 10^6\text{mm}^4$$

$$r_y = \left[\frac{1.24\times 10^6}{2546}\right]^{1/2} = 22.1\text{ mm.}$$

cl. 4.5.4.2 - Bearing Check.



Area in contact with flange, A

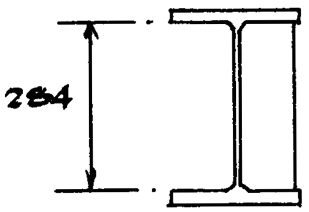
$A = 63\times 10 = 630\text{mm}^2.$

$A > 0.8F_x/p_{ys}$

$$= \frac{0.8\times 213.7\times 10^3}{275} = 622\text{mm}^2.$$

∴ Bearing check satisfied

cl. 4.5.1.5 ~ Buckling Resistance of stiffeners



75×10 th'k stiffener × 284 lg. Grd. 43

$L_E = 1.0\times 284 = 284\text{mm.}$

$$\lambda = \frac{284}{22.1} = 13$$

∴ from table 27c, $p_c = 275\text{N/mm}^2 = p_y$

$$P_q = \frac{750\times 275}{10^3} + \frac{134\times 2\times 6.7\times 275}{10^3}$$

$= 700\text{kN.} > 213.7\text{kN}$ ok.

∴ Buckling check satisfied.

Discuss me ...

Commentary to calculation sheet

The spine beams are non-composite and all Code references are therefore to BS 5950 :Part 1.

Ultimate limit state

Spine beams are usually selected with plastic cross-sections and can therefore be analysed plastically. Table 6 presents standard cases for end spans. Spine beams are torsionally restrained at their supports by their bracket connections. Additional diaphragm supports are provided near points of dead load contraflexure (ie. at rib positions) to ensure compliance with the requirements of Clauses 5.3.5 or 5.5.3.5 of Part 1 of BS 5950. As shown in Figure 3, these diaphragms are cut from UB sections, the spacing between the spine beams being adjusted to accommodate them. The critical case for stability of the support regions may be under pattern loading since this gives the greatest length of unrestrained bottom flange.

Since the plastic hinges in the span are normally the last to form they do not require torsional restraint. However a check on the buckling moment resistance is required to ensure stability of the top flange between ribs. Once again pattern loading effects should be considered; the critical case for top flange buckling will occur when only that span is loaded.

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CALCULATION SHEET

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Job Title: Design Example		
Subject: Spine Beam		
Client: SCI	Made by: DM	Date: Nov 89
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SPINE BEAM

Central pair of beams.

For the trial beam size consider:

a). Ultimate Limit State
(D+I) on both spans

b). Serviceability Limit State
Deflection.

Imposed Load.

Due to continuity of the spine beams any imposed load within one of the (9×9m) grids will have an effect on the central spine beams.

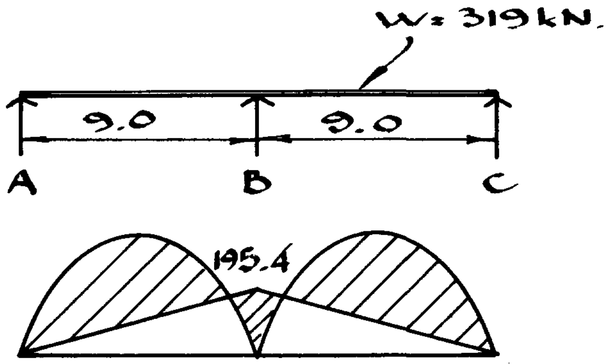
∴ Supported Area = 18×18 = 324 m² > 250 m²

∴ Max. Imposed load reduction = 25%.

Point Loads from Rib beam

Dead = 3.3 kN/m²

Imposed = 6.0 "



$W = [(1.4 \times 3.3) + (1.6 \times 6.0 \times 0.75)]\ 9 \times 3$

$= 319$ kN.

Reaction at B

$= 319 + \frac{195.4}{9} \times 2$

$= \underline{363 \text{ kN}}$ (point load for two beams).

Commentary to calculation sheet

See Table 6 for other standard cases of spine beam moments.

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CALCULATION SHEET

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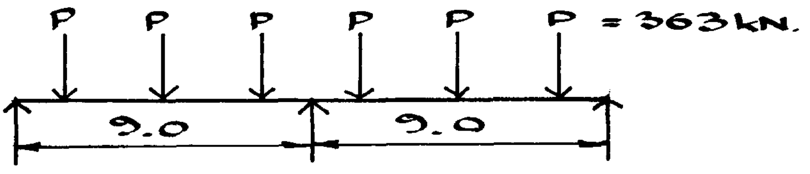
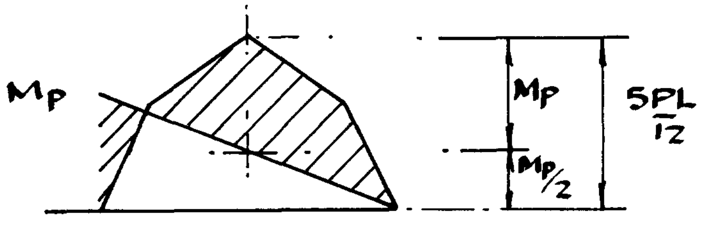
Job Title: Design Example			
Subject: Spine Beam			
Client: SCI	Made by: DM	Date: Nov 89	
	Checked by: JAR	Date: Dec 89	



$1.5 M_p = \frac{5PL}{12}$

$\therefore M_p = \frac{5PL}{18}$



$$\text{Req'd } M_p = \frac{5 \times 363 \times 9}{18}$$
$$= 907.5 \text{ kN.m.}$$

$$\text{Min. } S_x = \frac{907.5 \times 10^3}{265} \quad (\text{assume } T > 16\text{mm})$$
$$= \underline{2 \times 1712 cm^3} (= 3424 cm^3)$$

Deflection

Imposed

The deflections of the Rib and Spine beam for imposed load purposes will be assumed to be accumulative. Consider L/360 on the diagonal line of the 9×9m grid.

∴ Allowable total deflection

$$= \frac{9\sqrt{2} \times 10^3}{360} = 35.4\text{mm.}$$

from sheet 18 δ (Rib) = 18.3 mm.

∴ Allowable δ (Spine)

$= 35.4 - 18.3 \qquad = \underline{17.1 \text{mm}}$

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CALCULATION SHEET

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Client: SCI	Made by: DM	Date: Nov 89	
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Imposed point loads (Serviceability) from Rib beam.
$= 9 \times 3 \times 1.25 \times 6$
$= 202.5$ kN

$$\delta \text{(both sides loaded)} = \frac{0.0169 PL^3}{EI}$$

$$\therefore \text{Req'd } I = \frac{0.0169 \times 202.5 \times 9000^3}{205 \times 17.1 \times 10^4}$$

$$= 71169 \text{ cm}^4 \quad (2/35584 \text{ cm}^4)$$

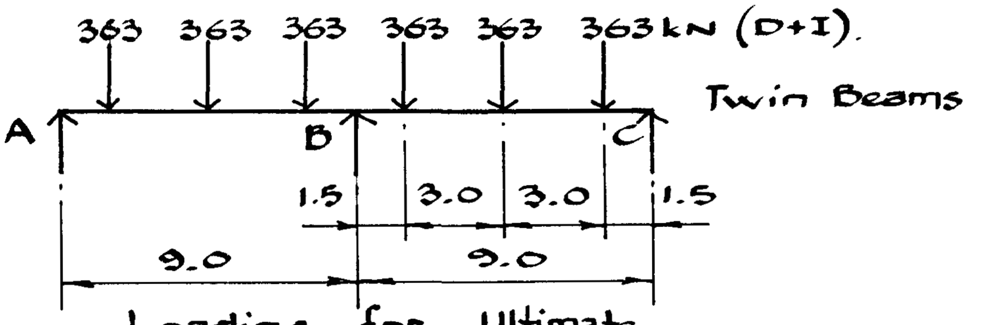
$\therefore$ Try 2/457×191×89 UB. Grd. 43.

$S_x = 2014 \text{ cm}^3 > 1712 \text{ cm}^3$ ok.
$I_x = 41021 \text{ cm}^4 > 35584 \text{ cm}^4$ ok.

$D = 463.6$ mm, $t = 10.6$ mm
$B = 192.0$ mm, $T = 17.7$ mm > 16.0 mm $\therefore p_y = 265$ N/mm^2.
$A = 113.9$ cm^2, $r_y = 4.28$ cm.
$d = 407.9$ mm, $x = 28.3$.

Section Classification
from Vol. 1 - Section Plastic

Detail Design Checks



Loading for Ultimate Limit State.

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CALCULATION SHEET

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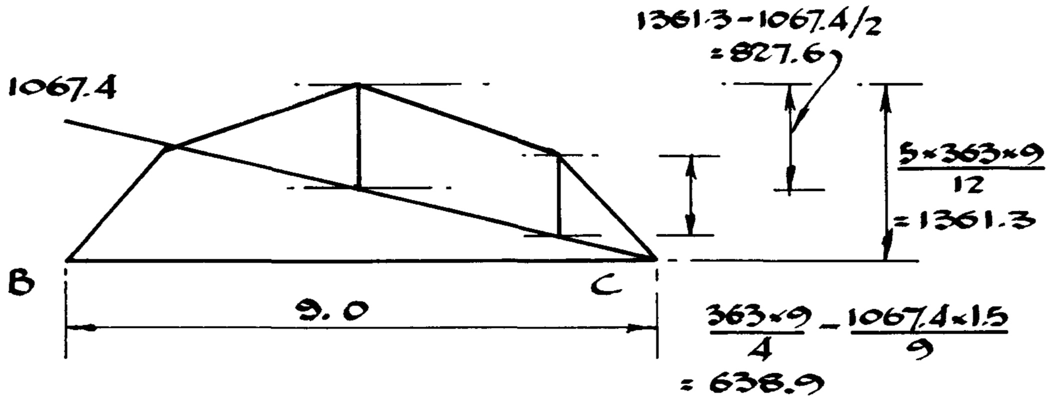
Subject: Spine Beam			
Client: SCI	Made by: DKM	Date: Nov 89	
	Checked by: VAR	Date: Dec 89.	



Mp Required = 907.5 kN.m

Mp Provided = 2014×265×2/10^3 = 1067.4 kN.m

<u>Shear at B</u>

$F_v = \frac{363 \times 3}{2} + \frac{1067.4}{9}$ = 663.1 kN.

$P_v = 0.6 p_y . A_v$

$= 0.6 \times 265 \times 463.6 \times 10.6 \times 2 / 10^3$

= 1562.7 kN

$0.6 P_v = 0.6 \times 1562.7 = 937.6$ kN.

$\therefore F_v = 663.1 < 0.6 P_v = 937.6$ kN

$\therefore$ <u>Low Shear</u>

<u>No moment reduction due to shear at B</u>

Commentary to calculation sheet

The bottom flange restraint at the support is provided by the bolted bracket to web connection. The bracket is detailed such that the lowest bolts in the group are as close as practical to the lower flange of the spine beams. The span restraint is provided by the diaphragm bolted between spines which is normally located at the first rib within the span. Alternatively the clear span can be evaluated in accordance with Appendix G of BS 5950:Part 1 which acknowledges the contribution provided by the rib connection.

Single spine beams occurring at the perimeter of the building are provided with fitted stiffeners in lieu of the bolted diaphragms.

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CALCULATION SHEET

Job No.	Sheet 29 of 46	Rev.
Job Title: Design Example		
Subject: Spine Beam		
Client: SCI	Made by: DM	Date: Nov 89
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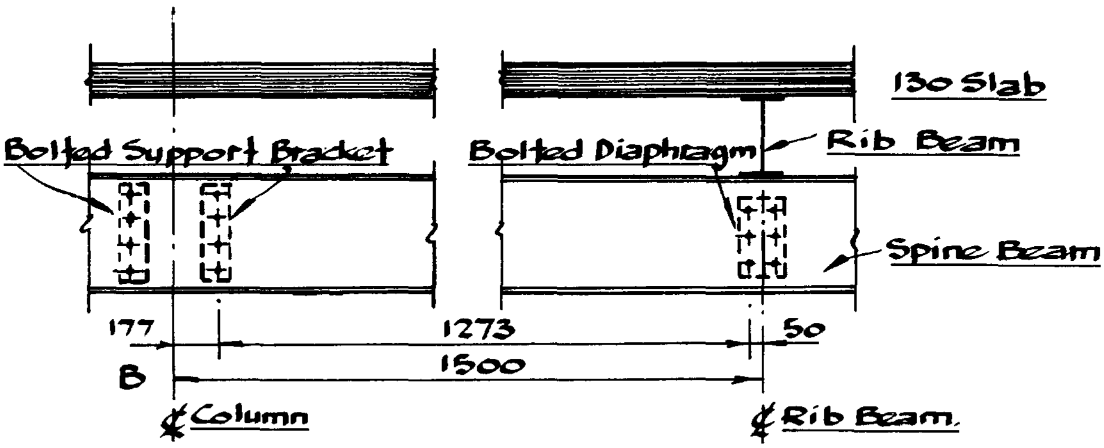
Lateral Restraint



Plastic hinge location at B

Lateral restraint to the compression flange to be provided at a distance not exceeding L_m from the plastic hinge restraint (B).

where:

$$L_m \leq \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{p_y}{275}\right)^2 \left(\frac{x}{36}\right)^2\right]^{1/2}}$$

$$= \frac{38 \times 42.8}{\left[0 + \left(\frac{265}{275}\right)^2 \left(\frac{28.3}{36}\right)^2\right]^{1/2}}$$

$= \underline{2147 \text{ mm}} > \underline{1500 \text{ mm}}$

(1273 mm. Actual distance between restraints)

∴ Restraint at Position of Rib will be satisfactory

Commentary to calculation sheet

It should be noted that this continuous beam analysis, by ignoring the partial rotational restraint from the columns at B, gives conservative, upper bound, values of the negative moments in the unloaded span AB. A subframe analysis, of the type shown in Calculation Sheet 40 is not used because the brackets are only partially effective in mobilising the restraining action of the column on the beam at B; it would therefore underestimate the negative moments in span AB.

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CALCULATION SHEET

Job No.		Sheet 30 of 46	Rev.
Job Title: Design Example			
Subject: Spine Beam			
Client: SCI	Made by DLM	Date Nov 89	
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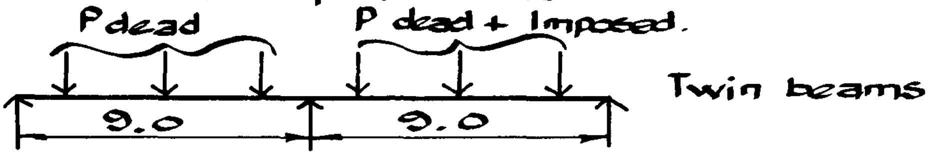
Pattern Loading

Consider loaded area ($18 \times 9 = 162\ m^2$) with (D+I) on one span and dead only on the other.
Imposed load reduction = 16.2%



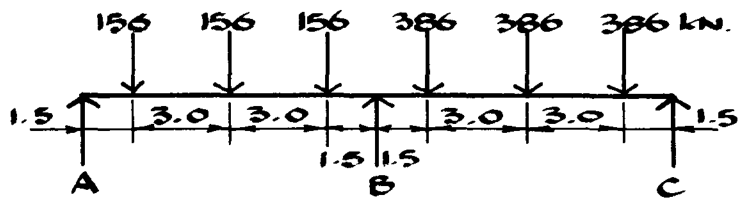
Dead Load (Elastic distribution), P_d.

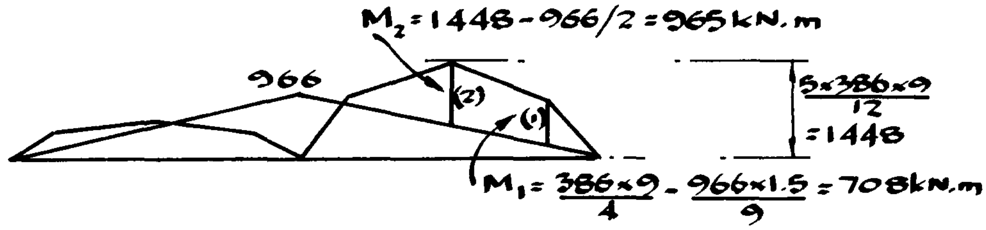
$P_d = 1.25 \times 1.4 \times 3.3 \times 3 \times 9 = 156\ kN$

Dead + Imposed (Plastic distribution), P_{d+i}

$P_{d+i} = [(1.4 \times 3.3) + (1.6 \times 6 \times 0.84)]\, 9 \times 3 + 195.4 \times 2/9$

$= \underline{386\ kN}$





$M_{support} = \dfrac{19PL}{48}$ (propped cantilever).

Dead $= 19 \times 156 \times 9/48 = 556\ kN.m$

D + I $= 556 \times 386/156 = 1375\ kN.m$

$\therefore M_{support} = \dfrac{556 + 1375}{2}$

$= \underline{966\ kN.m}$

Discuss me ...

Commentary to calculation sheet

The value of M_2, 482.5 kN.m, exceeds py.Zx which is 469.0 kN.m. However, since this is the last hinge to form, plastic restraint is not required.

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CALCULATION SHEET

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Client: SCI	Made by: JDM	Date: Nov 89
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Top flange stability between positions (1) and (2).

$M_1 = 708/2 = 354$ kN.m } per beam
$M_2 = 965/2 = 482.5$ "

$\beta = 354/482.5 = 0.73$

∴ From Table 18, $m = 0.86$

$\bar{M} = 0.86 \times 482.5 = 415$ kN.m

$\lambda_{LT} = nuv\lambda$ where: $n = 1.0$, $u = 0.879$

$\lambda = 3000/42.8 = 70.1$ from Table 14
$v = 0.93$

$\lambda/x = 70.1/28.3 = 2.48$

∴ $\lambda_{LT} = 1.0 \times 0.879 \times 0.93 \times 70.1$

$= 57.3$

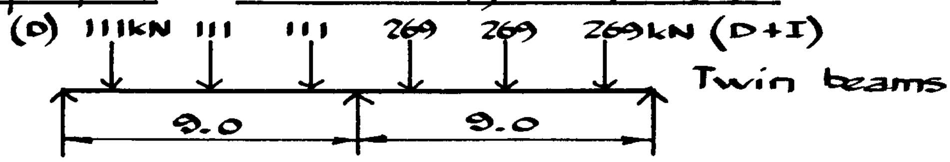
∴ p_b from Table 11 $= 213$ N/mm²

$M_b = 213 \times 2014/10^3$

$= 429$ kN.m > 415 kN.m OK

∴ No restraints required between positions (1) and (2).

Deflection ~ Serviceability Limit State.



$P_{dead} = 3.3 \times 3 \times 9 \times 1.25 = 111$ kN.

$P(D+I) = (0.84 \times 6 + 3.3)3 \times 9 + 195.4 \times 2/6 = 269$ kN.

F.E.M's (Propped Cantilever).

Dead $= \frac{19 \times 111 \times 9}{48} = 395$ kN.m.

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	Client: SCI	Made by: DM	Date: Nov 89
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$$D + I = \frac{19 \times 269 \times 9}{48} = 958 \text{ kN.m}$$

$$M_{support} = \frac{395 + 958}{2} = 676.5 \text{ kN.m}$$

$$\delta = \frac{53 \times 269 \times 9000^3}{1296 \times 205 \times 41021 \times 10^4 \times 2} - \frac{676.5 \times 9000^2 \times 10^3}{16 \times 205 \times 41021 \times 10^4 \times 2}$$

$$= 47.7 - 20.4$$

$$= \underline{27.3 \text{ mm}}$$

Estimated Imposed deflection

$= 5.04/8.34 \times 27.3 = 16.5$mm < 17.1mm ok (see sheet 26).

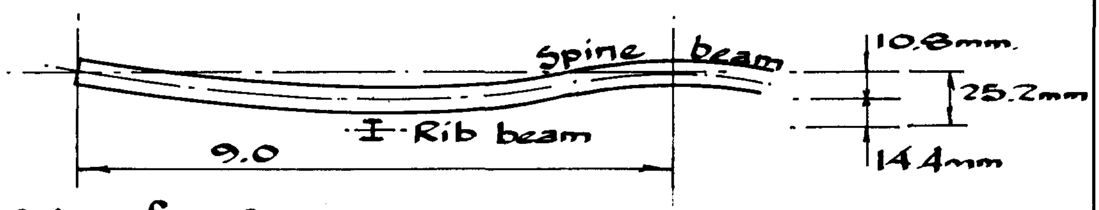
Total Imposed Deflection

Spine Beam = 16.5
Rib " = 18.3
$\underline{34.8 \text{ mm}}$

Allowable deflection (see sheet 26).

$$= \frac{9\sqrt{2} \times 10^3}{360} = \underline{35.4 > 34.8 \text{mm}} \quad \text{ok.}$$

Total Dead Deflection



Spine δ = 27.3 − 16.5 = 10.8
from sheet (7) = Const^n Stage = $\underline{14.4}$
$\underline{25.2 \text{ mm}}$

$\therefore$ Total (D+I) deflection = 34.8 + 25.2 = 60.0 mm

$$\frac{9\sqrt{2} \times 10^3}{60} = 212 \quad \text{Say ok.}$$

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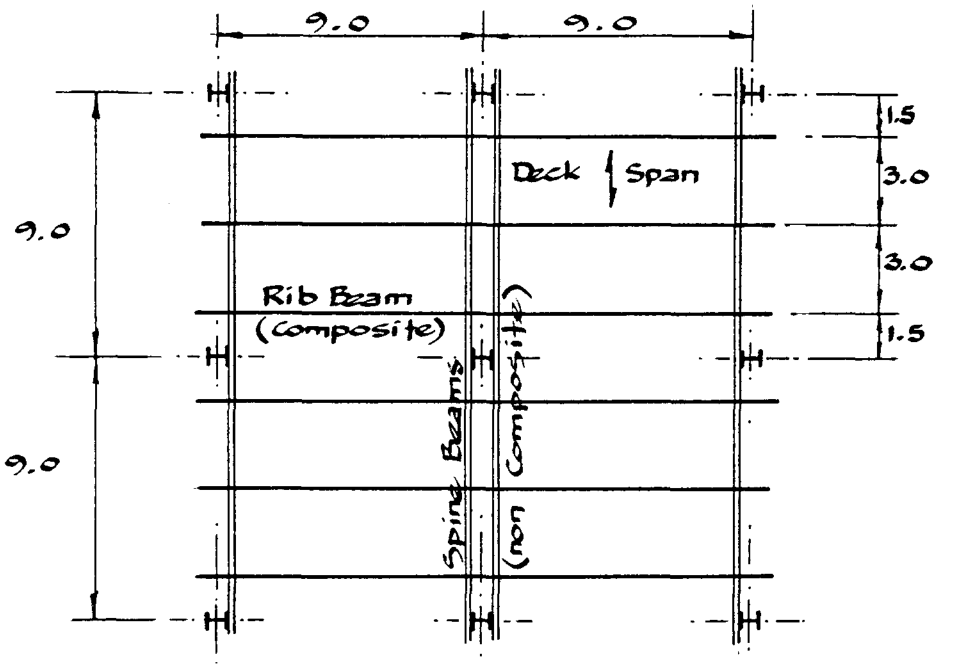
CALCULATION SHEET

Job No.		Sheet 33 of 46	Rev.
Job Title: Design Example			
Subject: Vibration			
Client: SCI	Made by: DLM	Date: Nov 89	
	Checked by: JAR	Date: Dec 89	

Vibration



PLAN.

The vibration check is in accordance with the SCI publication "Design Guide on the Vibration of Floors, 1989".

Loading	kN/m²
Imposed	6.0
Dead	
Slab (LWC)	2.35
C&S	0.40
Raised floor	0.40
Self wt. Beams	0.40
	3.55
10% Imposed	0.60
$w =$	4.15 kN/m²

$$m = \frac{4.15 \times 10^3}{9.81}$$

$$= 423 \text{ kg/m}^2$$

Discuss me ...

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CALCULATION SHEET

Job No.	Sheet 34 of 46	Rev.
Job Title: Design Example		
Subject: Vibration		
Client: SCI	Made by DM	Date Nov 89
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Slab and Beam Inertias

Rib Beam (Composite) ~ increase by 10% to allow for reduced modular ratio for dynamic E_c.

$= 32232 \text{ cm}^4 \times 1.1 = 35455 \text{ cm}^4$.

Spine Beam (non ~ composite)

$= 41021 \times 2 = 82042 \text{ cm}^4$.

Slab (per m. width) say 1000 cm^4.

Mode A (Nodal lines at spine beams)

Natural Frequency

(i) Slab (fixed ended)

$w = 4.15 \times 3 = 12.45 \text{ kN/m}$.

$$\delta_s = \frac{WL^3}{384EI} = \frac{12.45 \times 3000^3}{384 \times 205 \times 1000 \times 10^4} = 0.43 \text{ mm}.$$

(ii) Floor beam (simply supported).

$W = 4.15 \times 3 \times 9 = 112 \text{ kN}$.

$$\delta_f = \frac{5WL^3}{384EI} = \frac{5 \times 112 \times 9000^3}{384 \times 205 \times 35455 \times 10^4} = 14.63 \text{ mm}.$$

(iii) Main beam deflection is zero

$\therefore$ Total Deflection, y_o

$y_o = 0.43 + 14.63 + 0$

$= \underline{15.06 \text{ mm}}$.

$$\therefore NF = \frac{18}{\sqrt{15.06}}$$

$= \underline{4.6 \text{ Hz}}$

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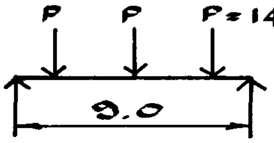
<u>Mode B</u> (Spine beams deflection)

(i) Slab δ_s = 0.43 mm.

(ii) Floor beam (fixed). $\delta_f = 14.63/5$ = 2.93 mm.

(iii) Main beam (simply supported).



$P = 4.15 \times 9 \times 3 \times 1.25$
$= 140$ kN.

$$\delta_m = \frac{53PL^3}{1296EI} = \frac{53 \times 140 \times 9000^3}{1296 \times 205 \times 82042 \times 10^4} = 24.8\text{mm.}$$

Total Deflection, $y_o = 0.43 + 2.93 + 24.8$
$= 28.16$ mm.

$$NF = \frac{18}{\sqrt{28.16}} = 3.4 H_z$$

$\therefore$ <u>Mode B Governs</u>, $f_o = \underline{3.4 H_z}$.

<u>Floor Response</u>

As $f_o = 3.4 H_z < 7.0 H_z$

then $R = \dfrac{68000 . C_f}{m . S . L_{eff} . \zeta}$

where:
$C_f = 0.4$ (since $f_o < 4 H_z$)
$\zeta = 0.03$
$S = 2 \times 9 = 18$m. (measured in the direction of the spine beams).

$RF = 24.8/28.16 = 0.88 > 0.6$
L_{eff} is the smaller of L^* or W.

$$\therefore L^* = 3.8 \left[\frac{EI_b}{m . b . f_o^2}\right]^{1/4}$$

$$= 3.8 \left[\frac{205 \times 35455 \times 10}{423 \times 3 \times 3.4^2}\right]^{1/4}$$

$= \underline{31.9\text{m}}$

Commentary to calculation sheet

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CALCULATION SHEET

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Job Title: Design Example		
Subject: Vibration		
Client: SCI	Made by: DLM	Date: Nov 89
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For the vibration check the width of building will be limited to 18m (2×9m)

$\therefore L_{eff} = W = 18m.$

$$\therefore R = \frac{68000 \times 0.4}{423 \times 18 \times 18 \times 0.03}$$

$= \underline{6.6}$

from Table 7.2 the response factor for a "General Office" = 8

$\therefore$ 6.6 < 8 <u>the floor is acceptable when used for general office purposes.</u>

Commentary to calculation sheet

Detailed design of columns

The normal methods of design used in BS 5950:Part 1 are applicable except that the characteristics of continuous construction and the PBA framing geometry, necessitates some modification of conventional simple design assumptions. Particular aspects are listed as follows:

Design moments

These are obtained from the subframe analysis that was carried out for bracket design.

Influence of beam continuity on column loads

Beam continuity in two directions and the resulting elastic shears can concentrate load into columns. Simple load area scheme calculations result in underestimating column sizes. The designer is recommended, for the purposes of scheme calculations for low rise buildings, to assess axial load from 'load area' with no live load reduction and with a load increase of 25% to allow for elastic effects and moments.

For buildings higher than 2 to 3 storeys, moment effects due to the rigid/semi-rigid spine beam connections will be low in relation to axial load effects. It is therefore possible to take some imposed load reduction into account even at scheme design stage. For final design it is still necessary to take account of the effects of beam continuity on load distribution.

Effective lengths

Where UC sections are used, the recommended orientation in relation to spine beams is for the major (xx) axis to be parallel with the longitudinal axis of the spine beams.

With this column orientation, minor axis buckling governs column axial capacity. The appropriate effective length is shown in Figure 23 (Commentary to Calculation Sheet 38). Clearly this is a conservative approach because of the push-pull action developed between the bracket and the slab. If the column is orientated otherwise, engineering judgement is necessary to evaluate the available restraint and an appropriate effective length, recognising the restraint from the slab and steelwork.

Where SHS columns are used, similar effective lengths apply.

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CALCULATION SHEET

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Job Title: Design Example		
Subject: Column		
Client: SCI	Made by: DLM	Date: Nov 89
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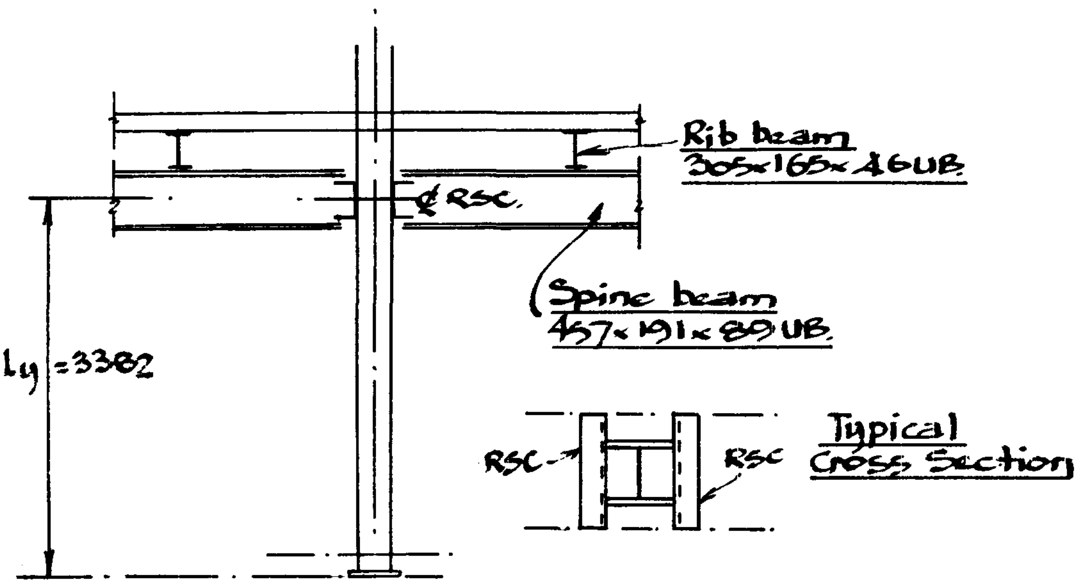
Column Design

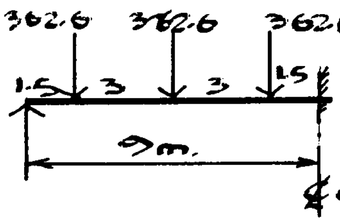


¢ Column 254×254×73UC.

Design cases:

(i) Dead + Imposed on both sides
Supported Area $= 18 \times 18 = 324 m^2$.
∴ Use full imposed load reduction ie 25%
Dead Load $= 3.3 \times 1.4 = 4.62$ kN/m².
Imposed Load $= (5+1) 0.75 \times 1.6 = \underline{7.20}$ "
$\underline{11.82 \text{ kN/m}^2}$

Rib point loads
$= 11.82 \times 3 \times 9 + \frac{2 \times 195.4}{9} = 362.6$ kN.



$R = \frac{91P}{48} = \frac{91 \times 362.6 \times 2}{48} = 1375$ kN.

Say 1380kN allowing for column own wt.
NB.
Two reductions in imposed load exist according to BS.6399:
i) when considering floor beam design
ii) when considering column design.

For the purposes of this design example the reductions in imposed load to the floor beam design have been assumed through-out

Commentary to calculation sheet

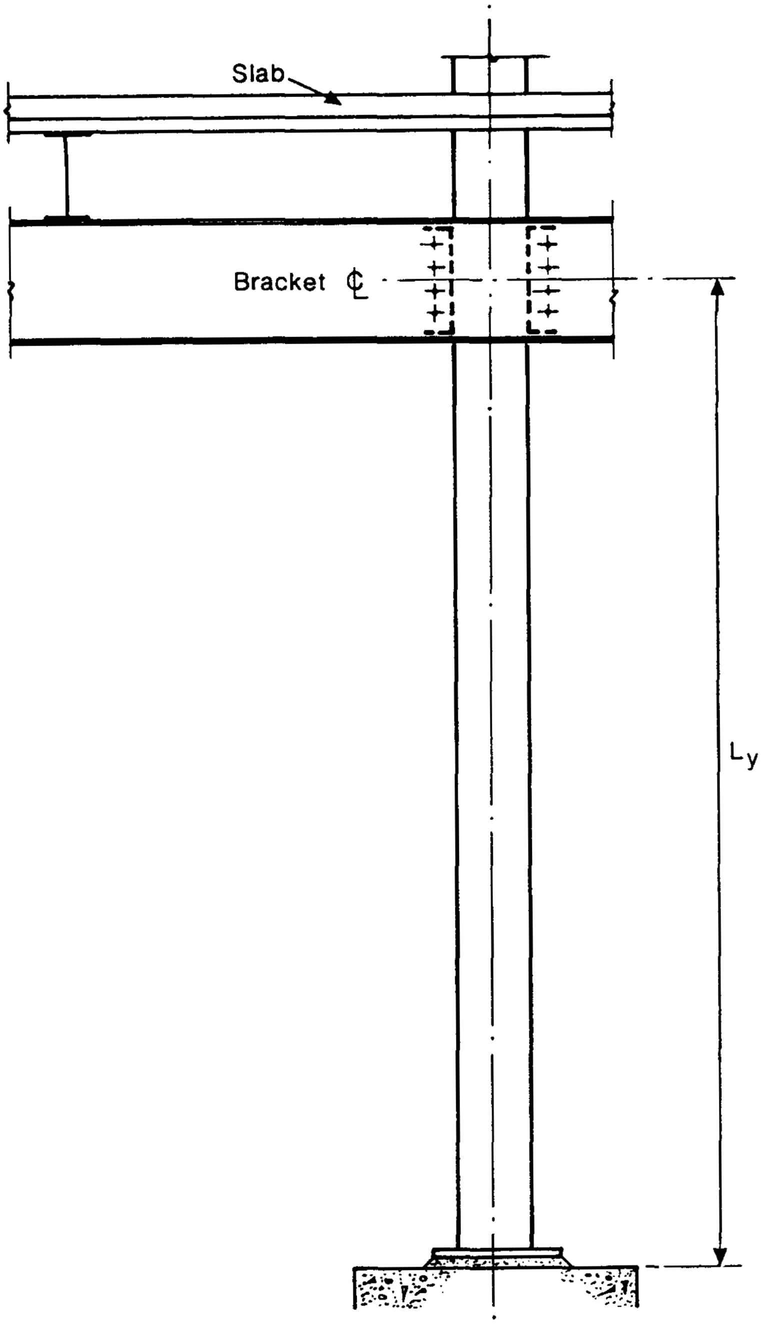


***Figure 23** Effective lengths of columns*

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CALCULATION SHEET

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(ii) Axial Compression = 1028+5 = 1033kN. (Sheet 41)
BM = 98.2 kN.m (Sheet 40).
NB. In practice the above axial column loads would be higher due to the number of stories under consideration.

Effective lengths.

$L_y = 3382$ mm.

Column Section ~ Try 254×254×73 UC. Grd. 43
case (i)
Axial comp. = 1380kN , BM = 0.
from the O/A buckling check cl. 4.8.3.3
check $F/A_g.p_c$ $M_x = M_y = 0$

calculate p_c
$\lambda_y = 3382/64.6 = 52.4$

∴ from Table 27 (c) $p_c = 216 N/mm^2$

$F/A_g.p_c = \frac{1380×10^3}{92.9×10^2×216} = 0.688 \leq 1.0$ ok.

Case 1 Satisfactory for axial compression

Case 2
Axial comp. = 1033kN
B.M.yy = 98.2/2 = 49.1 kN.m.

Local capacity check.
$F/A_g.p_y + M_y/M_{cy} \leq 1.0$ where: $M_y = S_y.p_y = 462.4×275/10^3 = 127.2$ kN.m.

$= \frac{1033×10^3}{92.9×275×10^2} + \frac{49.1}{127.2} = 0.79 \leq 1.0$ ok.

Overall buckling check.
$F/A_g.p_c + m M_y/p_y Z_y \leq 1.0$ the value for m = 0.57 obtained from Table 18.

$= \frac{1033×10^3}{92.9×10^2×216} + \frac{0.57×49.1×10^6}{275×305×10^3} = 0.849 < 1.0$ ok.
(0.515)   (0.334)

Case 2 Satisfactory for axial comp & bending

∴ Use 254×254×73 UC Grd. 43

Commentary to calculation sheet

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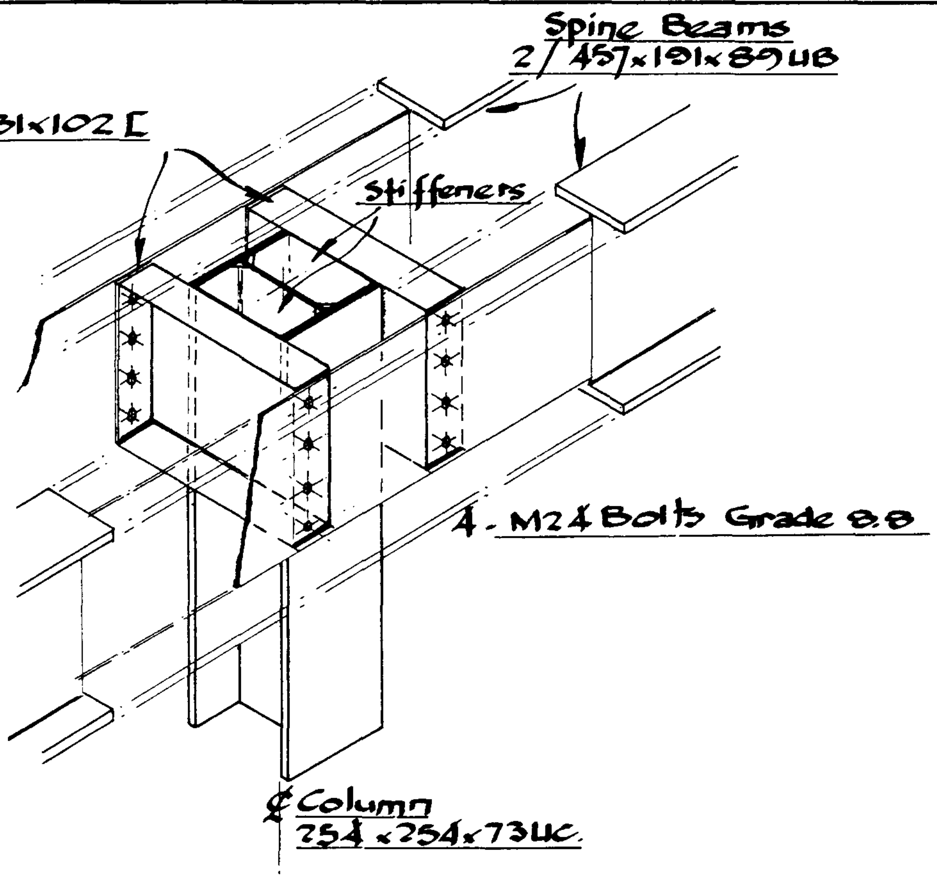

ISOMETRIC PROJECTION.

Commentary to calculation sheet

A subframe analysis is carried out assumming that the spine beams are rigidly attached to the columns by the brackets. This gives a conservative, upperbound, value of the moments which the brackets will have to resist when the spine beams are subject to pattern loading.

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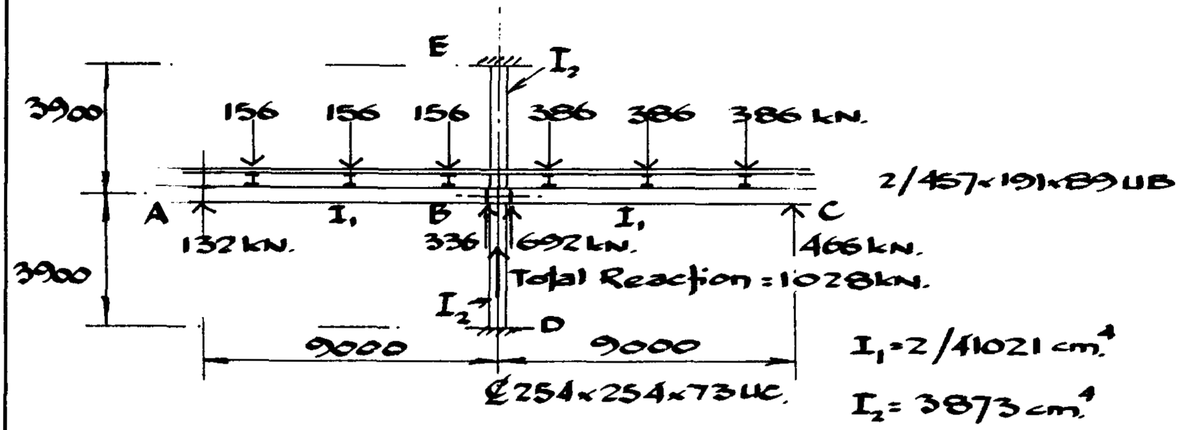
Subframe to determine out of balance moment

for loading details see sheet 30



Proposed Subframe at an intermediate position.

Joint B	k	k/Σk
Span		
AB	2 × 3/4 × 41021/900 = 68.4	0.44
BC	~do~ = 68.4	0.44
BD	3873/390 = 9.9	0.06
BE	3873/390 = 9.9	0.06
	Σk = 156.6	

F.E.M's

Span AB 19 × 156 × 9/72 = ± 371 kN.m

" BC 19 × 386 × 9/72 = ± 917 kN.m.

A: AB	B: BA	BC	BD	BE	C: CB
	0.44	0.44	·06	·06	1
-371	+371	-917			+917
+371	+185.5	-458.5			-917
0	+360.4	+360.4	+49.1	+49.1	0
	+916.9	-1015.1	+49.1	+49.1	

∴ Out of balance moment

= 49.1 × 2 : 98.2 kN.m

Commentary to calculation sheet

As discussed in Section 6.4.4 and analysed in Appendix C, for the case under consideration the moment may be considered as a vertical couple acting through the centre lines of the two bolt groups.

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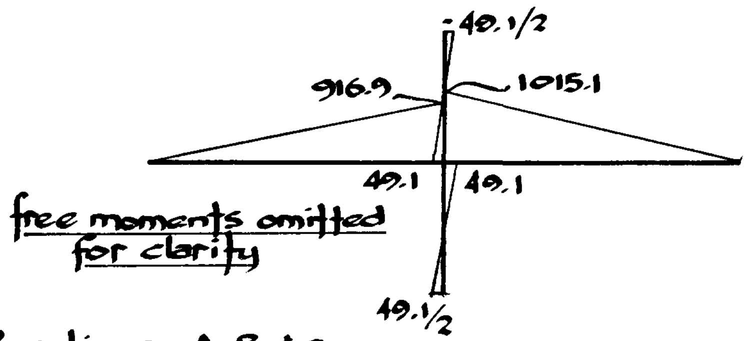
Spine Beam to Column Connection



Reactions A, B & C

$R_A = \frac{3 \times 156}{2} - \frac{916.9}{9}$

$= 132\ \text{kN}.$

Shear BA $= \frac{3 \times 156}{2} + \frac{916.9}{9}$

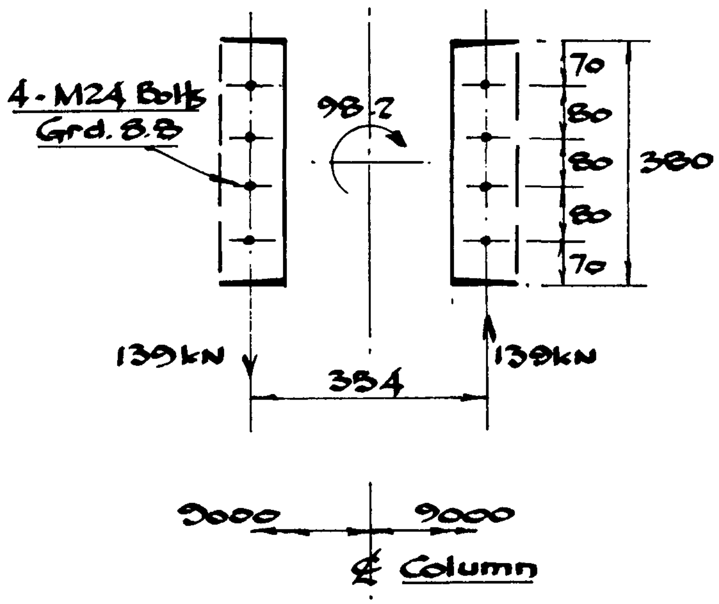
$= 336\ \text{kN}.$

$R_C = \frac{3 \times 386}{2} - \frac{1015.1}{9}$

Shear BC $= \frac{3 \times 386}{2} + \frac{1015.1}{9}$

$= 692\ \text{kN}.$

$\therefore$ Reaction at B $= 336 + 692 = \underline{1028\ \text{kN}.}$



Commentary to calculation sheet

It is a requirement of this design approach that the bolts, passing through the end plate and spine beam web, shall be critical in plate bearing (See Section 6.4.4).

Discuss me ...

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Bolt Forces –

for out of balance loading.

$= \frac{1028}{4} + \frac{98.2}{2 \times 0.354} = \underline{396\,kN}$

for balance loading.

$FEM = \frac{19PL}{48} = \frac{19 \times 386 \times 9}{48} = 1375.1\ kN.m$

Reaction at B

$= \left[\frac{386 \times 3}{2} + \frac{1375.1}{9}\right]2 = 1464\,kN.$

$\therefore$ Bolt force $= \frac{1464}{4} = 366\,kN.$

$\therefore$ Out of Balance loading condition governs.

Bolt Design

Check 4 ~ M24 Bolts Grd. 8.8

Shear Check, $P_s = A_s . p_s$

Force / Bolt $= \frac{396}{4} = 99\,kN.$

Tensile stress area $= 358\ mm^2$
Shear strength $p_s = 375\ N/mm^2$

Shear capacity $P_s = 375 \times \frac{358}{10^3} = \underline{134.3\,kN} > \underline{99\,kN}.$ ok

Bearing Check,
$t_{web} = 10.6\,mm$ & $t_{end\ plt} = 15\,mm.$

Bearing capacity of connected ply, t_{web}
$P_{bs} = d.t.\ p_{bs}$ $\quad p_{bs} = 460\ N/mm^2$

$= 24 \times 10.6 \times 460 / 10^3 = \underline{117\,kN > 99\,kN}$ ok.

Bearing Critical ie. $\underline{134.3\,kN} > \underline{117\,kN}.$ ok

$\therefore$ Use 4 ~ M24 Bolts Grd. 8.8

Commentary to calculation sheet

See Section 6.4.4 for a discussion of this lack-of-fit design case.

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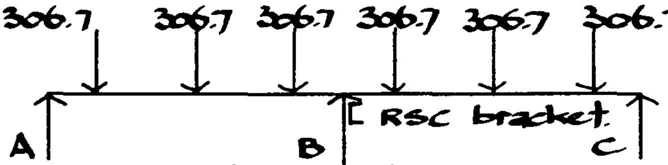
Lack-of-fit condition
weld design only

RSC bracket to UC weld.



NB. one bracket only.

Balanced Loading condition

Loading. (from Rib beam).

P(dead) = 1.25×1.25×3.3×3×9 = 139 kN.

P(D+I) = [(1.25×3.3)+(1.25×6×0.75)]9×3 + 195.4×2/9

= 306.7 kN. Supported area > 250 m² ∴ Impd. Reduction = 25%.

Reaction at B
= 91×306.7×2/48 = 1163 kN. (using the 1.25 factor).

Check against bearing capacity
= 117×4×2 = 936 kN (see sheet 42).

Check max. Reaction with respect to spine beam moment capacity.

moment capacity of spine beams = 1067.4 kN.m (sheet 28)
corresponding point load
$= \frac{1067.4 \times 48}{19 \times 9} = 300$ kN.

∴ Reaction at B = 91×300×2/48 = 1137.5 kN.
(936 < 1137.5 < 1163)

∴ Bearing capacity governs

Reaction at B = 936 kN.

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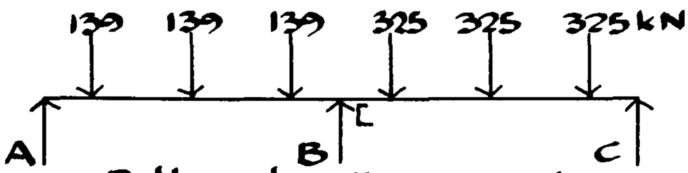
Lack-of-fit cont'd.



Pattern Loading condition

$P(D+I) = \left[(3.3\times1.25)+(1.25\times6\times0.84)\right]9\times3 + 195.4\times2/9$

$= 325\,kN.$ Supported area $= 9\times18 = 162\,m^2$
∴ Impd. Reduction = 16%

Reaction at B

$= \frac{91}{48}(139+325) = \underline{880\,kN.}$

AB	BA	BC	BD	BE	CB
	0.44	0.44	0.06	0.06	
-331	+331	-772			+772
+331	+166	-386			-772
0	+291	+291	+39.5	+39.5	0
	+788	-867	+39.5	+39.5	

FEM's
AB = ± 331 kN.m.
BC = ± 772 kN.m.

(for further information of subframe analysis see sheet 40)

∴ Out of balance moment = 39.5×2 = $\underline{79\,kN.m}$ (for both sides).

~ Loading Summary ~
Design vertical welds for:-
1) Balanced loading
Reaction at B = 936 kN (moment $= 936\times51/10^3 = 47.7$ kN.m)
Design horizontal welds for:-
2) Pattern loading
Reaction at B = 880 kN.

moment $= 880\times51/10^3 + 79.0 = 124\,kN.m. > 47.7\,kN.m$

note, 51mm represents the eccentricity (see fig. C1).

Discuss me ...

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Lack-of-fit cont'd.
Weld design

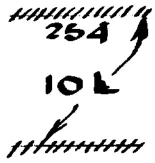
Vertical welds

$$\frac{936}{381 \times 2} = 1.23 \text{ kN/mm.}$$

8mm capacity $= 8 \times 0.7 \times 215 / 10^3 = 1.2$ kN/mm.

∴ Use 10mm fillet weld using E43 electrodes

Horizontal welds



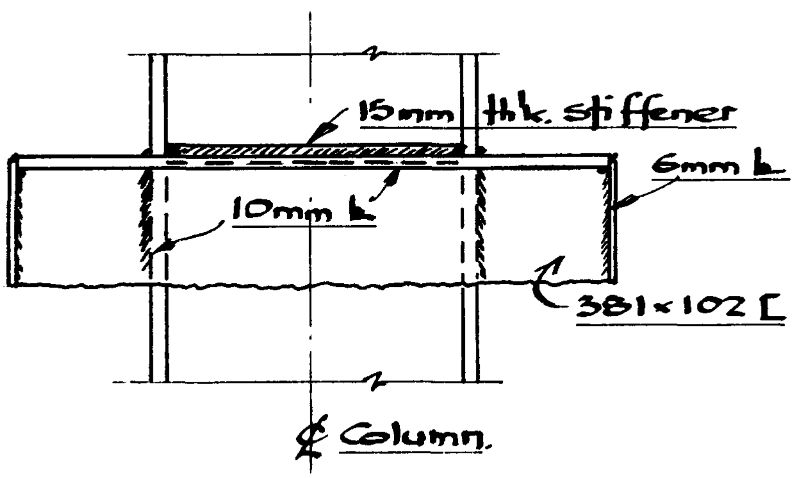
weld group modulus
$= 254 \times 190^2 \times 2 / 229$
$= 80082$

weld stress

$$= \frac{124 \times 10^6}{80082} = 1548 \text{ N/mm}^2.$$

Req'd Leg length $= \dfrac{1548 \times 1/0.7}{215}$
$= 10.3$ mm Say 10mm

∴ Use 10mm fillet weld using E43 electrodes



CHANNEL BRACKET TO COLUMN WELDS

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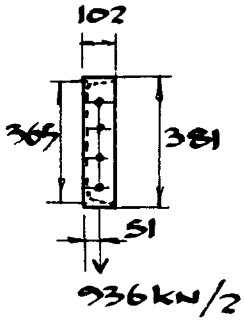
RSC ~ End Plate weld design



6mm ⊾ capacity using E43 electrodes

$= 6 \times 0.7 \times 215 / 10^3 = 0.9$ kN/mm.

moment $= 936/2 \times 51/10^3 = 23.9$ kN.m

(moment small compared to pattern loading).

Vertical welds

$936/2 \times 1/381 \times 0.5$ (2 welds) $= 0.61$ kN/mm < 0.9 ok.

∴ Use 6mm fillet weld.

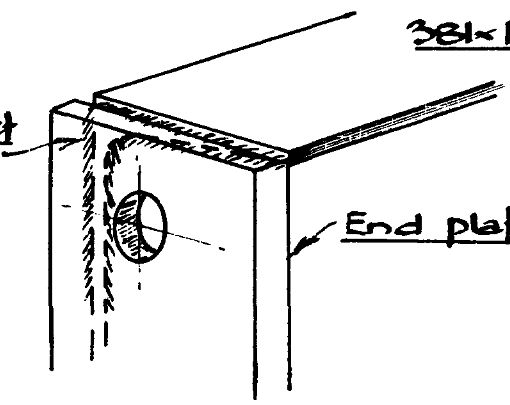
Horizontal welds

Flange force

$= 124/2 \times 1/0.365 = 170$ kN.

$170/102 \times 1/2 = 0.83$ kN/mm < 0.9 ok

∴ Use 6mm fillet weld.



END PLATE WELDED TO RSC BRACKET