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3760 14th Avenue, Suite 200, Markham, ON L3R 3T7
t. 905.604.3231 f. 905.604.3239 e. info@cisc-icca.ca w. cisc-icca.ca

Design and Construction of
COMPOSITE FLOOR SYSTEMS

ERRATUM 1

January 7, 1985

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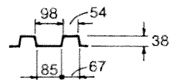
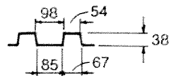
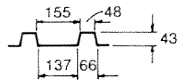
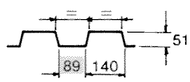
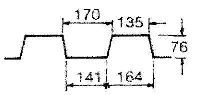
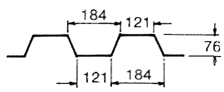
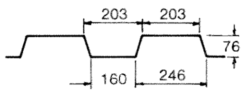
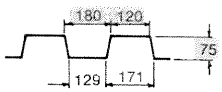
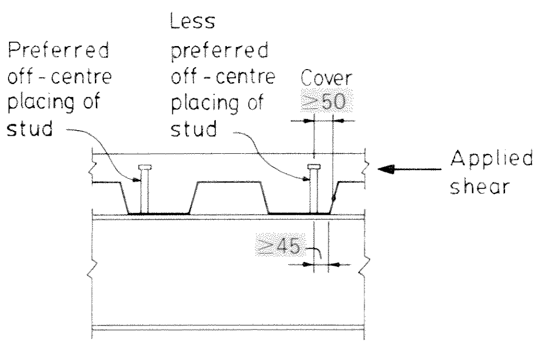
Westeel	T-15 INV	x		457 610	0.76 to 1.52	1.46	
Lorlea	D152CI		x	610	0.76 to 1.52	1.46	
Westeel	T-168 INV	x		1016	0.76 to 1.52	1.34	
Robertson	QL Span- rib	x		457* 914	0.76 to 1.52	1.34	
Canam	P2432	x		610	0.76 to 1.22	1.42	
Robertson	QL Lock- rib	x		610* 914	0.76 to 1.52	1.42	
Westeel	T-30V	x		762 813	0.76 to 1.52	1.42	
Lorlea	D-300-C	x		900	0.76 to 1.52	1.42	

2. Replace Figure 2.6, page 34, with the new figure below.



Note: Screened areas above denote changes.

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Design and Construction of
COMPOSITE FLOOR SYSTEMS

By
E. Y. L. Chien, P. Eng.
Chief Engineer,
Canadian Steel Construction Council.

J. K. Ritchie, P. Eng.*
Director, Project Analysis Division,
Canadian Steel Construction Council
and
Vice President - Canadian Institute
of Steel Construction.

* Positions at time of writing. Now President J. Keith Ritchie Marketing Services Inc.

Canadian Institute of Steel Construction

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PREFACE

This book contains a compilation of the latest technology related to the design and construction of composite steel-concrete floor systems. Design rules and construction techniques for composite floor systems have probably approached the true limit states from both strength and serviceability aspects using CAN3-S16.1-M84. Thus, the purpose of the book is to provide an overview of the various composite floor systems, and particularly to correlate design procedures, construction techniques, and building details to ensure that both strength and serviceability criteria are met.

The seven chapters of the book deal with the utilization of the best qualities of two materials: Concrete – an ideal compressive material subject to variations in mix design, curing, creep and shrinkage, climatic conditions, and on-site construction techniques, and Steel – a mill produced material manufactured under controlled conditions to a guaranteed minimum strength level fabricated under temperature controlled conditions to specified fabrication tolerances and erected to specified construction tolerances. Composite action, usually achieved by site welding stud shear connectors through steel deck onto the steel structural member, provides another variable in the quality equation.

The potential for maximizing the efficiency of these two materials is considerable. However, to achieve maximum efficiency, understanding their complementary performance is essential. Similarly, the marriage of "guaranteed" quality material with a material subject to both short term and long term variations in geometry, strength and quality of finish, calls for understanding of the controls necessary for reinforcing, placing, finishing and curing of concrete to maximize its efficiency.

All the above subjects, including the interaction of structural concrete with deformed steel deck to create composite steel-concrete deck-slab systems, are addressed herein. Deck-slab systems are examined as integral components of various structural steel and deck-slab composite systems. Beams, girders, trusses and the stub-girder system are examined in great detail from both design and construction aspects.

Construction considerations are especially important because the moment of inertia of a bare steel section is significantly incremented by the addition of a composite concrete deck-slab. Thus, steel deflections or stresses during concrete placement may well be the critical criterion governing selection of the structural members.

Each major design consideration is supported by calculations, and each major construction type is illustrated by a design example. Tables have been especially designed to assist in the quick selection of preliminary sizes of members and are included in appropriate chapters for convenience.

The authors gratefully acknowledge the contributions of M.I. Gilmor, P. Eng., A. Wong, P. Eng., D.L.T. Oakes, P. Eng., K. Garlick, W. Kahl and B. Williamson to the writing and production of this manual. Constructive reviews by J. Springfield, P. Eng., Dr. D. Stringer, P. Eng., and Dr. D.E. Allen, P. Eng. are also sincerely appreciated.

June, 1984

E.Y.L. Chien
J.K. Ritchie

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FOREWORD

The Canadian Institute of Steel Construction is the national industry association representing the structural steel, open-web steel joist and steel plate fabricating industries in Canada. Formed in 1930 and granted a Federal charter in 1942, the CISC functions as a non-profit organization promoting the efficient and economic use of fabricated steel in construction.

For many years, the CISC has promoted the latest technology for the use of steel in construction through research, development, meetings, seminars, conferences, computer programs and the publication of other design aids such as the Handbook of Steel Construction and textbooks, such as Limit States Design in Structural Steel – SI Units and Calcul aux états limites des charpentes d'acier. The CISC is therefore pleased to publish Design and Construction of Composite Floor Systems.

Design and Construction of Composite Floor Systems is unique in its state-of-the-art view of steel-concrete composite floor systems and incorporates the latest results of research from many countries. The authors have provided many practical suggestions and details of construction as well as new selection tables for stub-girders and composite trusses to complement their detailed calculation procedures. This book provides a wealth of information for the designer, educator, student and contractor of steel structures.

Although no effort has been spared in an attempt to ensure that all data in this book is factual and that numerical values are accurate to a degree consistent with current structural design practice, the Canadian Institute of Steel Construction does not assume responsibility for errors or oversights resulting from use of the information contained herein. Anyone making use of the contents of this book assumes all liability arising from such use. All suggestions for improvement of this book will be forwarded to the authors for their consideration in future printings.

The Head Office of the CISC is located at 201 Consumers Road, Suite 300, Willowdale, Ontario, Canada M2J 4G8. Regional Offices are situated in Vancouver, Calgary, Winnipeg, Toronto, Montreal and Halifax.

CHAPTER 1

1.0 DECK-SLAB SYSTEMS IN STEEL FRAMED BUILDINGS

1.1 INTRODUCTION

In the construction of early skeletal steel framed buildings, various means of providing a working floor supported by the steel floor beams included clay tile formed-concrete infilled assemblies, timber formed reinforced concrete floor slabs and others. This chapter will address the evolution of the latest method of constructing floor components of skeletal steel framed structures and lead on to discuss the relationship of this method of floor construction with the other frame components. Very detailed discussion of the design of the steel floor framing components will also be addressed.

At the outset, it should be noted that comments throughout this and subsequent chapters relate primarily to uses and occupancies involving static loading, relatively light loading and, unless noted, dry service. Although assemblies discussed may be appropriate for other uses and occupancies, additional considerations may be required.

The use of steel deck with a concrete cover slab has gained almost universal acceptance in the construction of Canadian steel framed buildings. Such wide acceptance can largely be credited to over two decades of effort by structural researchers, deck manufacturers and practising structural designers in the improvement of the deck-slab product and its related design criteria. Some of the important improvements relating to deck-slab construction can be grouped into the following categories:

- deck profile optimization
- introduction of discrete embossments to provide mechanical shear connection with a concrete cover slab
- composite deck-slab design methodology
- deck and deck-slab diaphragm design information

Discussion of such improvements follows in appropriate sections of this chapter.

The selection of an appropriate deck-slab system is one of the keys to structural efficiency of a steel framed building. The importance of thorough understanding and proper evaluation of the application of deck-slab systems cannot be over-emphasized. It should be noted that the selection of the cheapest deck-slab system will not automatically lead to the lowest cost structure, especially when composite construction is the chosen design system.

A wide choice of deck profiles and steel thicknesses is available, although geographically dependent on manufacturers to some extent. Various concrete cover slab thicknesses, along with the type and strength of concrete to be used, may also be considered.

Considerations in selecting a deck-slab can be categorized into firstly, a list of deck-related considerations, and secondly, a list of slab-related considerations.

Some important deck-related considerations are as follows:

- deck depth selection

- material requirements (deck steel specification for roll forming, coating designation, base steel thickness)
- deck profile and embossment details
- composite deck-slab design methodology
- structural diaphragm capacity
- power and communications serviceability features
- flute closure and screed flash details
- edge support details
- trim angle supports (around columns and openings)
- deck installation considerations as well as shipping and handling.

Slab-related considerations are highlighted as follows:

- concrete cover slab thickness
- concrete density and type
- concrete strength
- concrete shrinkage and creep characteristics
- reinforcing requirements for shrinkage, temperature and strength
- construction practice (placement, finishing, curing and inspection)

1.2 STEEL DECK-SLAB TERMINOLOGY

Steel deck is a structural building product manufactured by roll forming light gauge zinc coated structural quality sheet steel into fluted elements to act as load carrying elements in roof and floor construction. Floor decks designed for composite deck-slab interaction employ specially designed embossments or indentations rolled into webs and flanges of the deck profile. These embossments act as mechanical connectors to transfer horizontal shear between the steel deck and a structural concrete cover slab and to prevent vertical separation of the two materials. This permits the steel deck and concrete slab to act compositely under load, resulting in an efficient one-way slab system. Embossed decks are commonly called ***composite*** steel decks, to differentiate from ***non-composite*** steel decks which are incapable of providing positive interaction with a structural concrete slab. Non-composite steel decks may be used in either floor or roof construction. If used in floor construction they would serve as a form for a concrete slab. As used in this publication the term steel ***form*** refers specifically to forms as defined by Cl. 17.2 of CSA Standard CAN3-S16.1-M84[1.1], and hereon referred to as S16.1.

Composite steel deck may be supplied in the form of cellular or non-cellular units. ***Cellular steel deck***, consisting of a fluted sheet element interconnected with a flat sheet on the underside, may be suitable for use in a composite deck-slab system, provided that the fluted sheet has the necessary embossments to achieve composite action. ***Non-cellular steel deck*** refers to the fluted sheet steel element alone, and is suitable for most common applications of deck-slab usage.

A combination of a particular steel deck and a selected concrete cover type and thickness results in a ***deck-slab*** system. Similarly, a combination of a steel form and a concrete slab produces what is called a ***form-slab*** system. Although permitted by Canadian design standard S16.1, the latter combination is used infrequently in the types of structures being discussed.

A deck-slab system, connected to supporting steel beams or trusses by means of shear connectors, is the most common method of achieving composite interaction of steel structural members with the deck-slab system. This method of construction maximizes structural efficiency of both the steel and concrete materials used.

When stud shear connectors are used to connect a deck-slab to steel supporting members, the effectiveness of the shear connection is largely dependent upon the shape and the direction of the concrete filled deck flutes. When concrete ribs parallel the steel member, providing continuous encasement of shear studs in the line of stress, full effectiveness of the stud shear connectors can

usually be attained. However, in applications where the deck runs perpendicular to the steel member, the width of the concrete filled deck flute, or concrete rib, becomes critical to the effectiveness of the stud shear connections. Full effectiveness of stud shear connection can be achieved by a so called ***wide-rib*** profile deck (see Fig. 1.1) connected to a steel member. By definition in S16.1, concrete ribs in ribbed slabs formed by wide-rib profile decks have average concrete rib widths equal to at least twice the depth of the steel deck. Decks producing a ribbed slab with narrower ribs are defined here as ***narrow-rib*** profile decks. Reduced capacity of stud shear connectors is necessary in such applications. For more specific discussion on the topic of stud shear connections, the reader is referred to the material presented in Chapter 2. Only some of the cellular deck profiles available offer a wide-rib format, while others are classified as narrow-rib profiles. Reference to individual manufacturers is necessary to ascertain the properties of their cellular products.

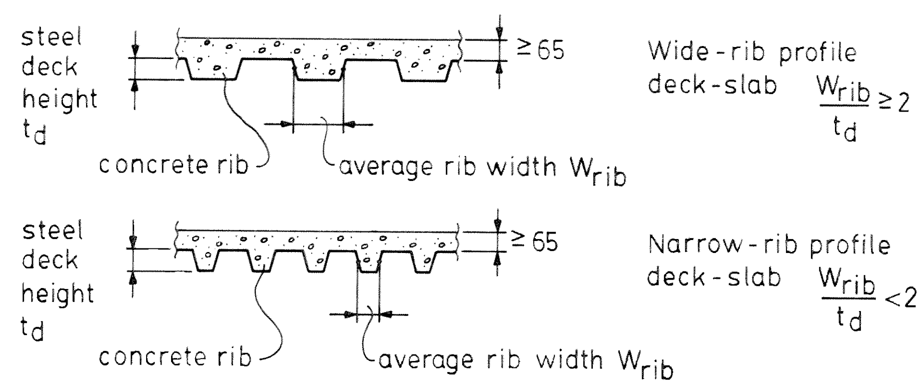


Figure 1.1
Concrete Ribbed Slab Formed by Steel Decks

1.3 STEEL DECK RELATED CONSIDERATIONS

a) *Material Requirements.* Sheet steel intended for the manufacture of steel deck is produced in the form of zinc coated structural quality steel sheet in coils and cut lengths. In Canada, the material is usually ordered to Canadian Sheet Steel Building Institute (CSSBI) Specification 101M "Zinc Coated Structural Quality Sheet Steel for Steel Deck"[1.2]. This specification provides:

- Limitations on base steel nominal thicknesses,
- Zinc coating designation applicable to steel sheets,
- Basis of purchase,
- Chemical requirements,
- Mechanical requirements,
- Coating bend test,
- Dimensions and tolerances (negative tolerance is restricted), and
- Order thicknesses.

Two steel grades are included in this specification. Grade A provides a minimum yield strength of 230 MPa and Grade B a minimum yield of 255 MPa. Technical information on sheet steel used for the production of deck is provided in a CSSBI publication, "Metric Zinc Coated (Galvanized) Sheet Steel for Structural Building Products" - Technical Bulletin No. 6[1.3]. In this publication, minimum standards relating to zinc coating designation and base steel nominal thickness for cellular and non-cellular deck applications are listed. It should be noted that all sheet steel thicknesses are expressed in millimetres to two decimal places. Base steel nominal thickness is used to establish section properties and for structural design calculations. Minimum requirements for zinc coating applicable to steel deck for various exposures are also tabulated. Note that a minimum zinc coating known as ***Wiped Coat*** under the coating designation ZF75 is widely used for steel floor deck in buildings conditioned for human comfort. The ZF75 zinc-iron alloy coating provides short term corrosion protection to the base steel during fabrication, shipment, site storage and erection.

Steel deck profile and thickness govern spanning capability of the deck-slab system either under the fresh-concrete condition load or under occupancy load. Thus, a deeper deck profile and/or a thicker deck sheet would permit wider support spacing. This would permit a reduction in the number of steel beams required, as well as a reduction in the steel unit price due to a reduction in the number of connections per tonne of steel. A thicker steel deck also provides more resistance to damage from accidental point loads during construction.

When shear studs are to be welded through steel deck to provide composite action between deck-slab and steel members, the weldability of studs through various combinations of single or double steel sheets should be checked. See Section 2.6.

b) *Deck Profiles and Embossment Details.* Profiles and embossments vary from manufacturer to manufacturer and are usually proprietary. Non-composite deck dates back to the 1930's while embossed composite deck made its debut in the early sixties. Other types of composite deck also were introduced in the same period but did not gain popularity in Canada; for example, steel deck with inverted pyramidal shaped ribs to provide composite action with a concrete cover slab. Another example of an unsuccessful entry in this field was the use of non-composite deck with welded wire reinforcing, transverse to deck flutes, to provide composite action with the concrete slab.

Through the use of embossed composite deck, positive moment slab reinforcing is replaced by the deck material, permitting efficient utilization of both steel and concrete as well as increasing the span of the deck-slab and spacing of supporting beams.

Earlier types of composite decks generally conformed to the narrow-rib profile type. The narrow concrete filled deck flutes in 38 mm deep decks are too flexible to develop the full shear strength of a stud, resulting in the need for an excessive number of stud connectors per composite member. Narrow ribs in deck-slabs using deeper decks (76 mm) are even less suitable for composite beam design. For 38 mm deep and 43 mm deep decks, wide-rib efficiency may be achieved by inverting a narrow-rib profile. However, one should note that special side-lap details should be worked out with the manufacturer, so that deck panels can be connected to adjacent panels from the top surface. See Figure 1.2. The increase in concrete quantities and dead load should also be accounted for in such a design.

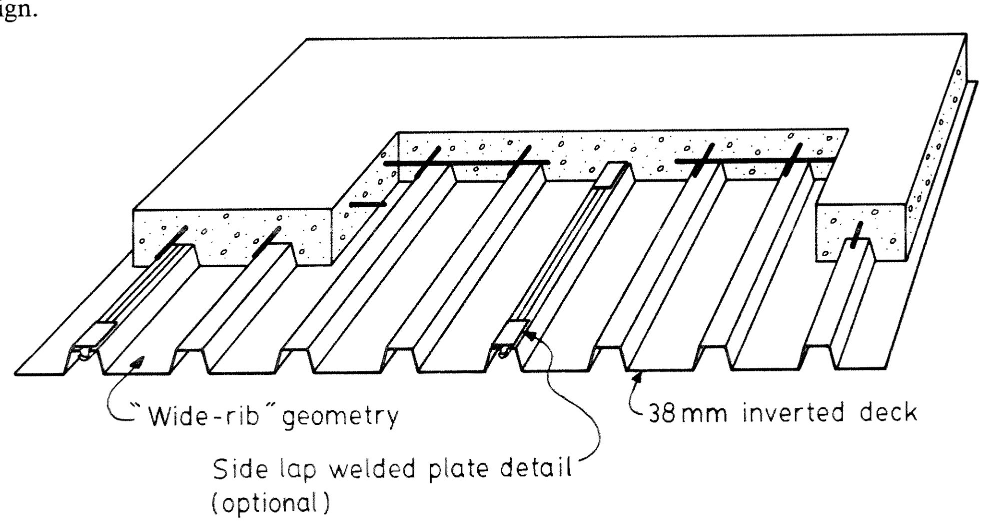
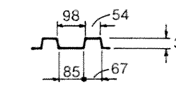
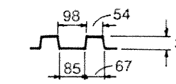
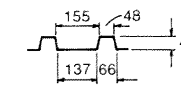
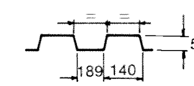
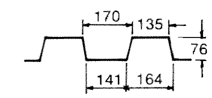
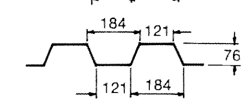
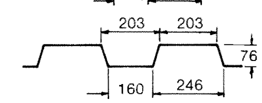
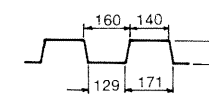


Figure 1.2
Wide-Rib Efficiency Achieved by Inverting Narrow-Rib Profile Deck

TABLE 1.1 WIDE-RIB PROFILE EMBOSSED STEEL DECK FOR COMPOSITE DECK-SLAB DESIGN

Producer†	Designation	Steel+ Grade A	Steel+ Grade B	Sheet Width (mm)	Thickness Range (mm)	F.P. Spray Contact Area (m^2/m^2)	Profile# Geometry
Westeel	T-15 INV	x		457 610	0.76 to 1.52	1.46	
Lorlea	D152CI		x	610	0.76 to 1.52	1.46	
Westeel	T-168 INV	x		1016	0.76 to 1.52	1.34	
Robertson	QL Span-rib	x		457* 914	0.76 to 1.52	1.34	
Canam	P2432	x		610	0.76 to 1.22	1.42	
Robertson	QL Lock-rib	x		610* 914	0.76 to 1.52	1.42	
Westeel	T-30V	x		762 813	0.76 to 1.52	1.42	
Lorlea	D-300-C	x		646	0.76 to 1.52	1.42	

† Company names abbreviated.
* Cellular deck using same deck-profile, available.
\+ As listed in CSSBI 101M-84.
\# Check profile geometry with deck producers prior to design use.

To increase the shear connection efficiency of stud shear connectors in deck-slab systems, deck profiles were redesigned and tested during the early seventies. One such example is an Inryco 76 mm deck, which was developed and tested for use in the Sears Tower[1.4]. Competition spawned another profile and both, with minor modifications, are currently being used in Canada.

Similarly, the competition for a deck profile design for the First Canadian Place, First Bank Tower in Toronto generated an all-Canadian 51 mm deep profile. Table 1.1 shows a partial list of wide-rib profile composite decks currently in use in Canada.

Other items to be considered when a deck profile is selected include:

- volume of concrete in the ribbed portion of deck-slab, and thus dead load,
- contact area of sprayed-on fire protection (see variation in profile surface areas Table 1.1), and
- compatible cellular decks to be blended with non-cellular decks for desired electrical power and communication systems.

c) *Composite Deck-Slab Design Methodology*. After developing a deck product, a composite steel deck manufacturer usually publishes product catalogues containing technical data, design procedures, and design load tables. Due to the proprietary nature of deck research and testing, a standard design procedure is still evolving. The earliest design procedures developed for composite deck-slab systems were based on the working stress design concept. Subsequently, extensive research by Ekberg, Porter and Schuster has led to numerous research papers[1.5 to 1.8]. Research work conducted by Ekberg and Schuster at Iowa State University since 1967 has led to the development of semi-empirical equations for the evaluation of ultimate shear-bond strength of composite deck-slabs.

Under static load conditions, two primary failure modes exist for composite deck-slab systems, i.e. flexural failure and shear-bond failure.

Design procedures published by steel deck manufacturers usually include the following steps:

i) check deflection under fresh concrete including ponding (accumulation of concrete due to deck deflection). A deflection of L/180 or 20 mm is a normal limitation.
ii) check effects due to construction load during slab pouring.
iii) check effects due to concentrated construction load.
iv) check shear-bond capacity of composite section.
v) check maximum concrete compressive stress in composite section.
vi) check maximum steel deck tensile stress in composite section.
vii) check live load deflection of composite section.

Continuity of deck spans may be considered in cases i) to iii). Cases iv) to vii) are checked based on simple span condition. A standard for composite steel deck is currently being prepared by the CSSBI[1.18].

There is limited information regarding the behaviour of composite deck-slabs under heavy moving loads. Existing research indicates that the mechanical bonding pattern can significantly affect behaviour of the composite slab. For example, one product may not sustain a repeated load at the level of the first end-slip load, while another product may be more susceptible to fatigue failure of the sheet steel[1.9]. Recent research projects on the behaviour of composite deck-slabs under repeated load[1.10,1.11,1.12] have produced useful information on some 38 mm and 76 mm deep decks. Some manufacturers, based on independent research studies, possess quantitative information on the performance of some of their products tested under repeated loads. It would suffice to say that dynamically loaded composite deck-slab applications should be approached with caution.

d) *Deck Depth Selection*. Steel decks produced in Canada can be grouped into four depths, i.e. 38 mm, 43 mm, 51 mm and 76 mm. Deep decks generally produce larger deck design spans, thus allowing larger, and frequently more efficient, beam spacings. However, the selection cannot be made independently because of the potential impact on other building components.

e) *Structural Diaphragm Capacity*. Steel decks or deck-slabs attached directly to the structural framing of a building can be designed to act as a horizontal shear diaphragm to transmit in-plane shear forces to lateral load resisting systems such as cores, or braced bents. The use of steel deck diaphragms in place of in-plane steel bracing has become an accepted practice in Canada, U.S.A., Australia, U.K. and many European countries. Many recent publications on deck diaphragm design are available[1.13 to 1.17]. For analysis purposes, a diaphragm can normally be considered analogous to a plate girder with the steel deck or deck-slab forming the web and the peripheral members serving as the flanges. The diaphragm girder is a field assembled unit and is totally dependent on the adequacy of the connections, both component to component connections and diaphragm to main structure connections. Three types of connections require special consideration. These are the arc spot welds that connect the deck to the intermediate members and peripheral members, the side-lap connections between deck units for diaphragm shear action of the plain deck, and the steel shear

connections that connect the deck or deck-slab to transmit shear to the lateral load resisting systems, and peripheral flanges. When compositely designed members are used, stud shear connectors will usually serve as boundary connectors and intermediate diaphragm-to-beam connectors.

Another use of the deck-slab diaphragm is to provide lateral support for columns of multi-storey buildings. In such cases, the shear required to be locally resisted by the floor diaphragm equals the force caused by the P-Δ effect of the vertical load in the column at the floor level under consideration. The stability force for the column may be transmitted directly to the deck-slab system through bearing; or gradually transferred into the deck-slab system via the floor framing connections, with gradual distribution to the slab through welds or shear studs (see Figure 1.3). A similar mechanism is available to provide column stability during erection, prior to concrete slab placement. Column loads are much lower and this condition would not often be critical as a local consideration. However, overall building stability may be a consideration, possibly requiring the steel deck diaphragm to be supplemented with a concrete cover slab at various height levels in the structure.

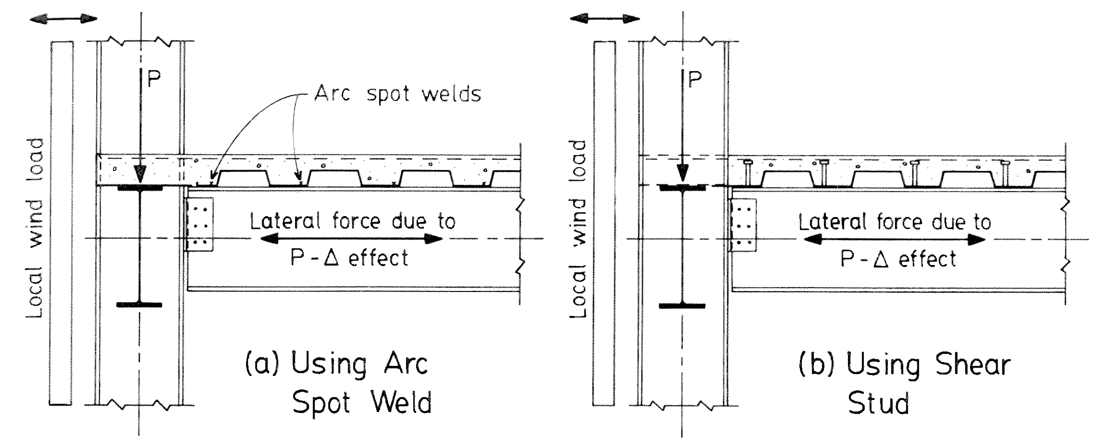


Figure 1.3
Deck-Slab Shear Diaphragm
Acting as Column Lateral Support

f) *Power and Communication Servicing Features*. In addition to serving as a load carrying platform, the steel deck-slab system can be designed to accommodate in-floor distribution systems for power and communication needs. The layout of the distribution system and the design of the structural floor and its supporting members are therefore necessarily integrated. Cellular floor deck units can be blended with non-cellular units to a designer-selected module, subject to product manufacturing width limitations, and can be designed compositely with a structural concrete cover slab. However, in areas where the concrete slab is interrupted by such items as trench header ducts, or wide runs of standard header ducts, non-composite deck-slab design is usually required (see Figure 1.4). In order to provide compatible loading capacities in these areas, it may be necessary to reduce deck span (maintaining the same deck thickness), or increase deck thickness. Composite action can sometimes be achieved in spans where standard header ducts are introduced by providing special reinforcing details (Figure 1.4a).

g) *Edge Details*. Steel deck edge details are dependent on the particular project. Decks may be interrupted at girders to permit shear stud placement. Change in direction of span requires a closure to prevent concrete leakage through ends of flutes, Fig. 1.5. Floor openings and exterior edges require a screed flash of sheet steel or wood forming. Floor openings may also require local strengthening and/or slab reinforcement. Decks abutting to core walls require intermittent support when deck span parallels wall, or continuous support when deck span is perpendicular to the wall. A designer must determine whether deck-to-shear-core connections are for temporary support for concrete placement, whether they form part of the gravity load carrying system or whether they

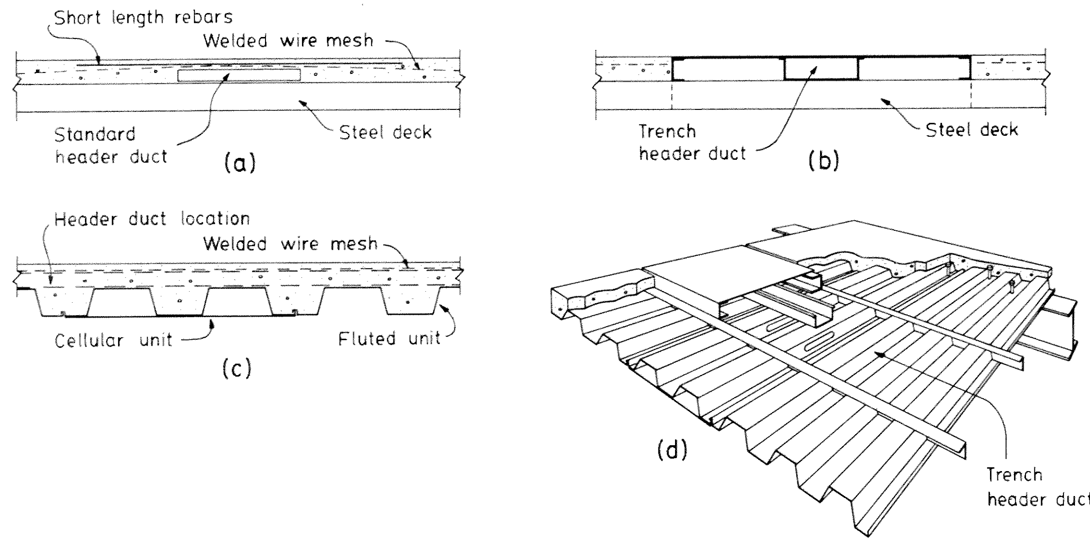


Figure 1.4
Power and Communication Serviceability (or Wire-Management) Features

provide the shear transfer mechanism for transmitting external lateral forces and column stability forces into the shear core. At column locations, closure plates may be required between flanges and trim members to pick up the deck edge around large columns. Figures 1.5 to 1.11 suggest typical details. More exterior edge details of deck-slabs when used with composite beams and girders are shown and discussed in Section 4.12.

h) *Deck Installation Considerations.*

- Skew cutting: This involves the cost of waste and cost for labour hours.
- Length of panels: Preferred lengths are in the range of 9 metres. Lengths beyond 12 metres are rarely handled.
- Width of Coverage: Wider units generally require fewer man-hours for installation and shipment, but width is usually tied to profiles available from the successful bidder.
- Hoisting: Cost enquiries to deck fabricators should include cost of deck hoisting.
- Quantity of Deck: Premium price for small jobs to be investigated.
- Shear Studs: Field installed stud shear connectors are generally supplied and installed by deck fabricator/erector.
- Painting of support beams: When stud shear connectors are used, welding is facilitated by eliminating shop priming of steel beams.

1.4 CONCRETE SLAB CONSIDERATIONS

In composite floor or roof design, concrete slabs or cover slabs play an important role in providing structural strength to the overall framing system. In addition to structural functions, slabs or cover slabs provide a working surface for other trade work as well as a sub-floor for floor finishes such as tiles, carpets, etc. It must be emphasized that concrete slabs or cover slabs for compositely designed floors or roofs should not be treated the same as a non-structural concrete fill. Proper care should be taken to ensure that the concrete is well proportioned, mixed, transported, placed, finished, protected, cured, and acceptance checked. Notwithstanding that a certain amount of cracking will occur in all forms of concrete construction, such types of construction remain structurally safe. Appropriate steel reinforcing will minimize and control cracks caused by shrinkage, flexural action of beam joints at girder locations, longitudinal shear, diaphragm stresses, and stress concentrations around openings, at corners of concrete cores or at supports of cantilevered slabs.

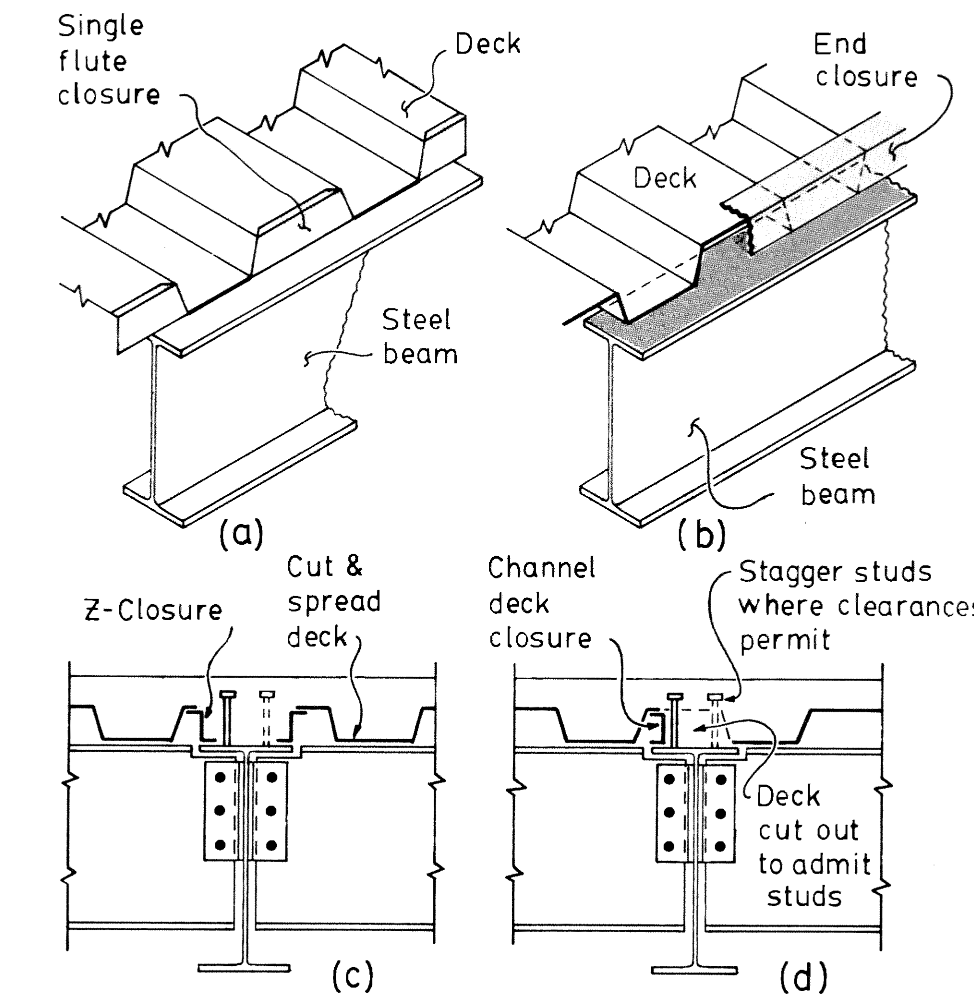


Figure 1.5
Deck Flute Closure Details

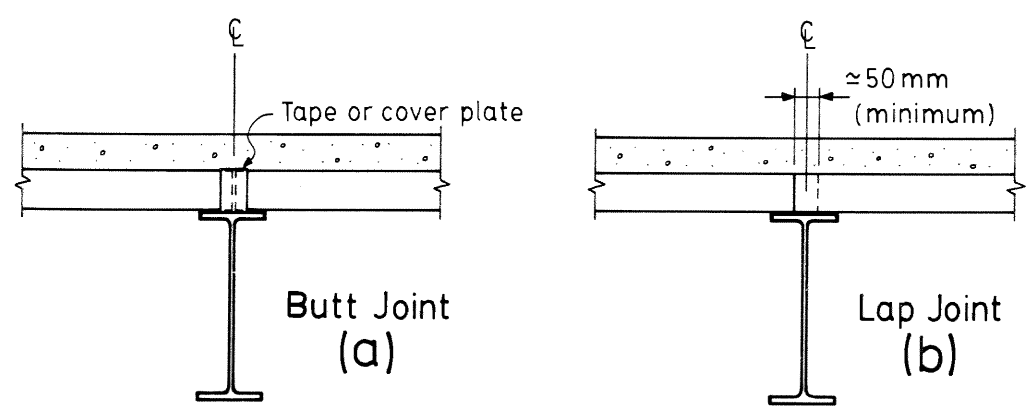


Figure 1.6
Deck End-Joint Details

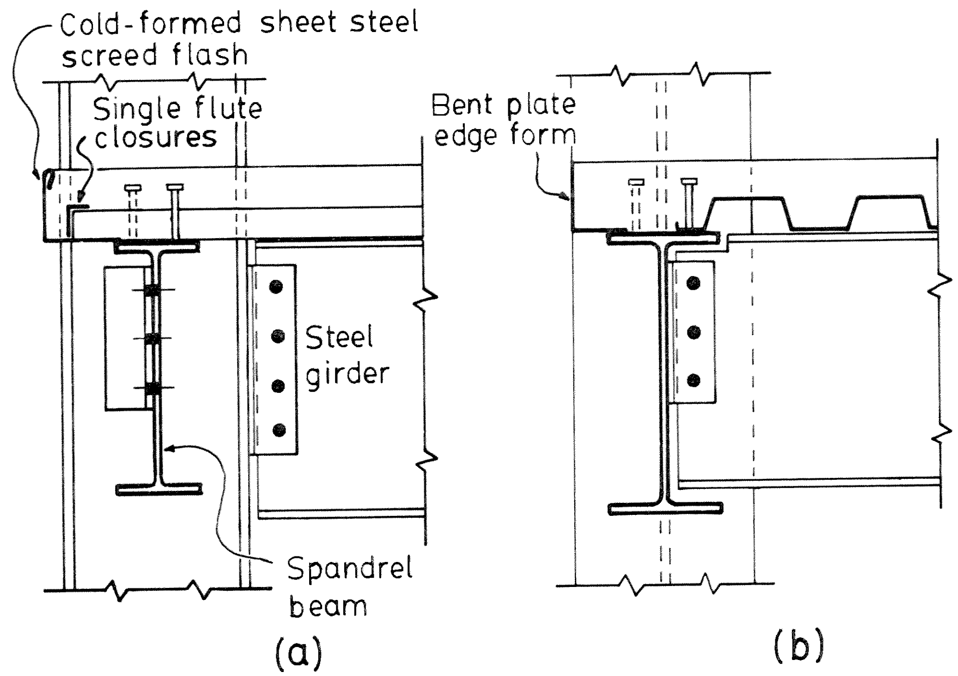


Figure 1.7
Spandrel Edge Details Showing
'Screed Flash' and 'Edge Form' Angles

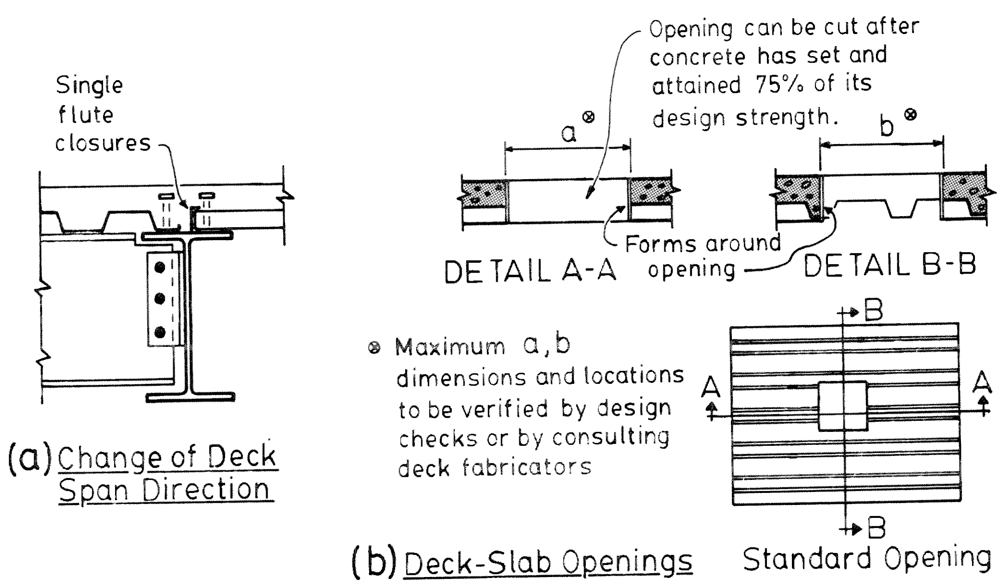


Figure 1.8
Details Showing Deck Span Direction Change
and Unreinforced Deck-Slab Opening

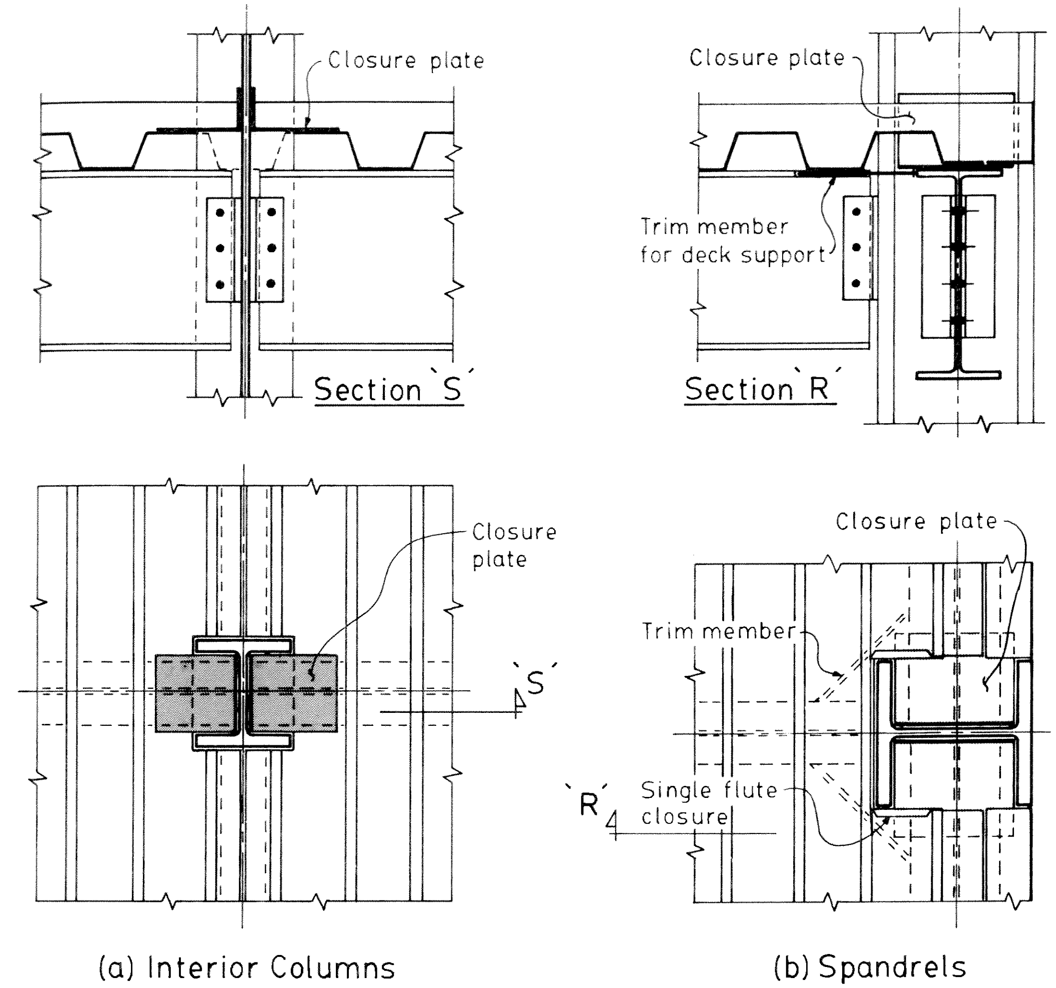


Figure 1.9
Details of Closure Plates at Column Locations
and Trim Members for Deck Support

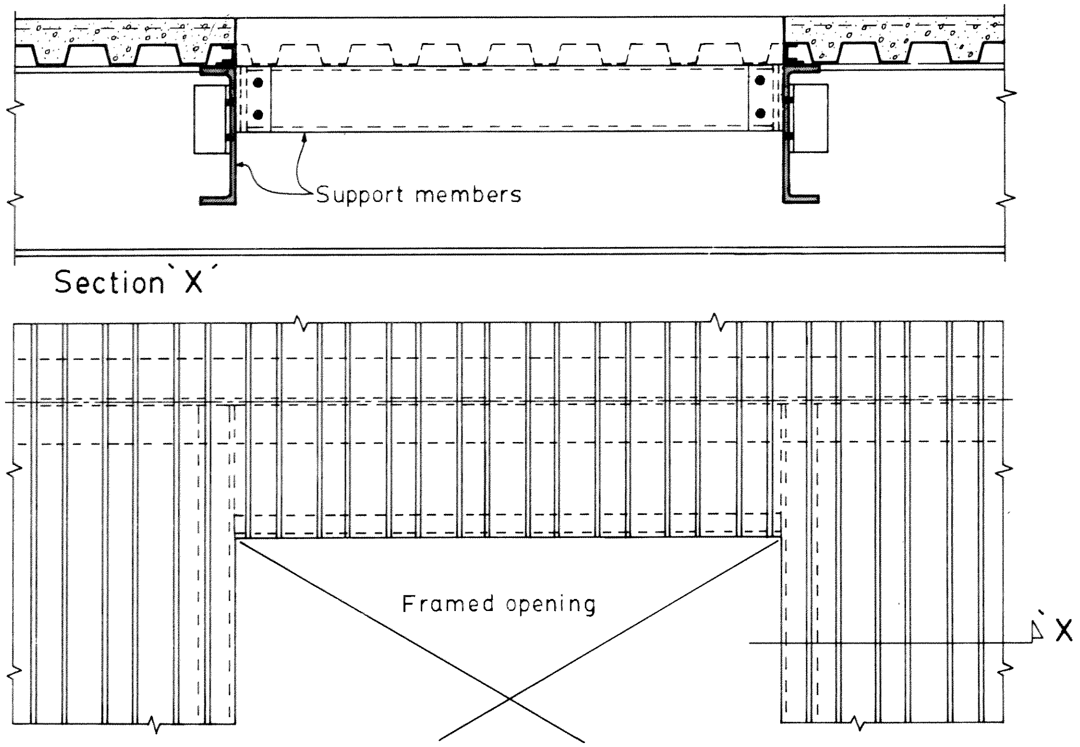


Figure 1.10
Detail at a Large Framed Opening

The following are some of the definitions of the key words or phrases used in this publication to describe concrete slab or slab materials. In general, they conform to those in S16.1, with appropriate clarifying descriptions and/or figures where necessary.

Concrete for slab or cover slab construction shall conform to Portland cement concrete in accordance with CSA Standard CAN3-A23.1-M Concrete Materials and Methods of Concrete Construction.

Concrete slab or slab is a reinforced cast-in-place concrete slab at least 65 mm in effective thickness. The design area, equal to the design effective width times effective slab thickness, shall be free of voids or hollows except for those specifically permitted in the definition of effective slab thickness.

Concrete cover slab or cover slab is that portion of a cast-in-place reinforced concrete slab above the flutes of steel deck. All flutes shall be filled with concrete so as to form a ribbed slab.

Effective slab thickness, t_c, should be taken as overall slab thickness, t_o, provided that:

- the slab is cast with a flat underside, Figure 1.12a; or
- the slab is cast on corrugated steel forms, Figure 1.12b, having height of corrugation not greater than 0.25 tmes the overall slab thickness; or
- the slab is cast on fluted steel forms, Figure 1.12c whose profile meets the following requirements:
  - the minimum width of concrete ribs (the part of the form filled with concrete) shall be 125 mm.

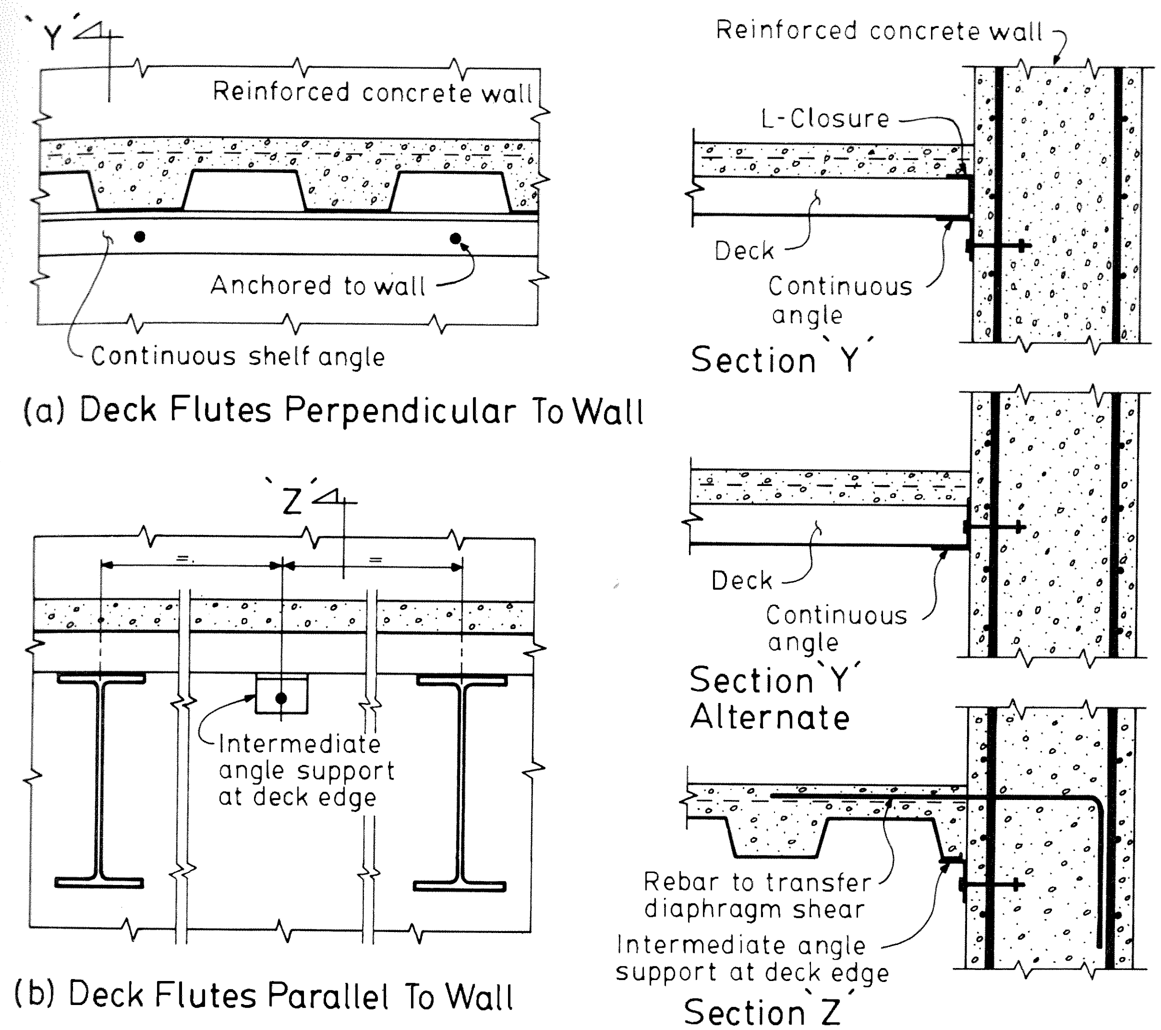


Figure 1.11
Deck Edge Support Detail at Reinforced Concrete Walls

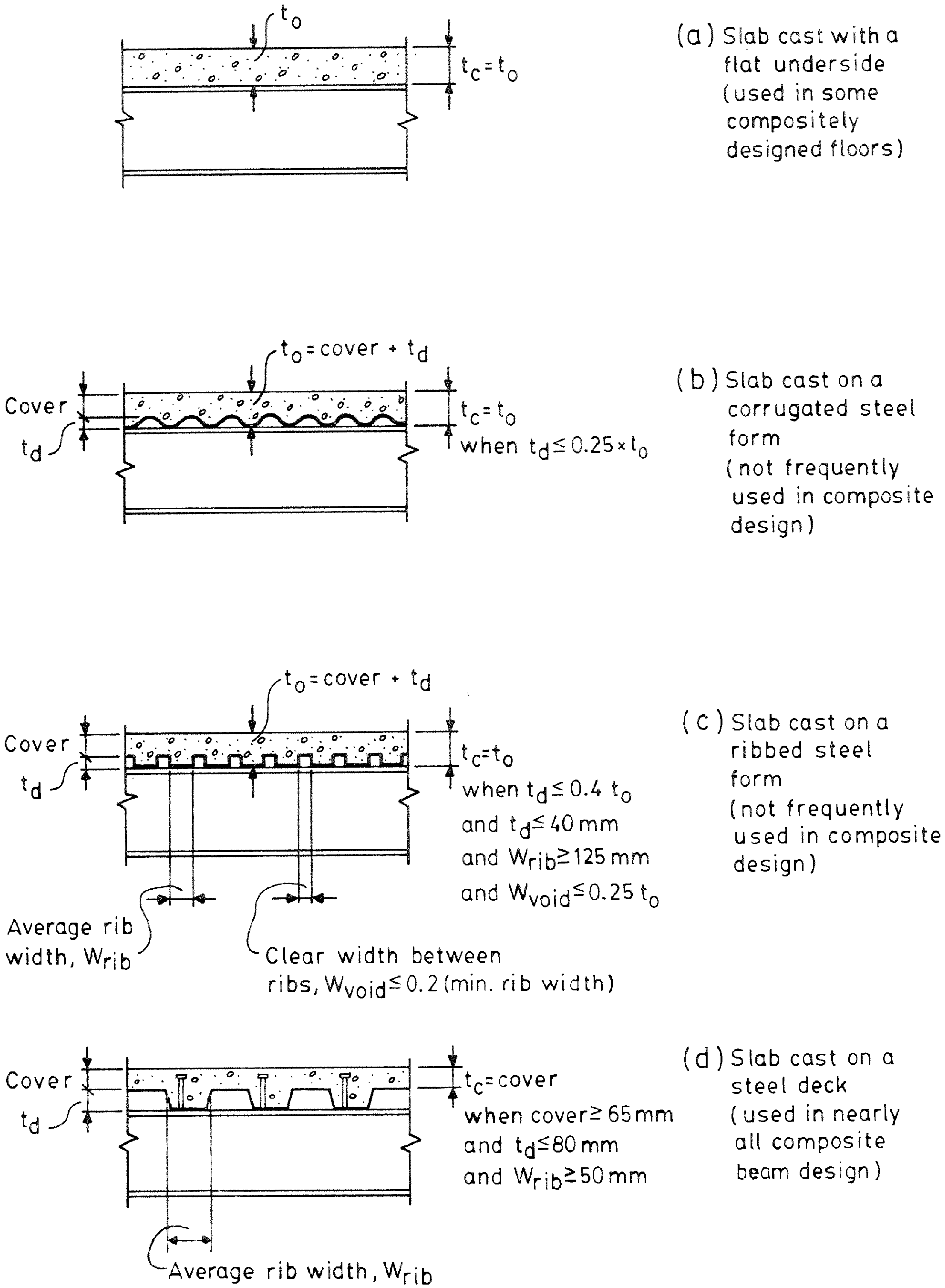


Figure 1.12
Effective Cover Slab Thickness, t_c
for Composite Floor Member Design

- the maximum form height shall be 40 mm but not more than 0.4 times the overall slab thickness.
- the average clear distance between concrete ribs shall not exceed 0.25 times the overall slab thickness.
- the average clear distance between concrete ribs shall not exceed 0.20 times their minimum width.

In all other cases, effective slab thickness, t_c, means the overall slab thickness, t_o, minus the height of form or deck, t_d.

Effective cover slab thickness, t_c, is the minimum thickness of concrete measured from top of concrete cover slab to the top of steel deck, see Figure 1.12d. This thickness shall not be less than 65 mm unless the adequacy of a lesser thickness has been established by appropriate tests.

Design effective width of concrete is the width of slab or cover slab deemed to be effective when computing the composite sectional properties for strength and stiffness calculations. The S16.1 rules for calculating design effective width, Figure 1.13, of slab or cover slab are given as follows:

Slabs or cover slabs extending from both sides of the steel section, truss, or joist shall be deemed to have a design effective width, b_1, equal to the least of:

- 0.25 times the composite beam span
- 16 times the overall slab thickness, or overall cover slab and steel deck depth, plus the width of the top flange of the steel section or top chord of the steel truss or joist, and
- the average distance from the centre of the steel section, truss or joist to the centres of adjacent parallel supports.

Slabs or cover slabs extending from one side only of the supporting steel section, truss or joist shall be deemed to have a design effective width, b_1, not greater than the width of top flange of the steel section, or top chord of the steel truss or joist, plus the least of:

- 0.1 times the composite beam span,
- 6 times the overall slab thickness or overall cover slab and steel deck depth, and
- 0.5 times the clear distance between the steel section, truss or joist and the adjacent parallel support.

Some concrete slab or cover slab considerations are as follows:

a) *Slab Thickness or Cover Slab Thickness*
During a building design, a decision on the type of a cover slab for floor or roof construction is required.

Items governing the selection of slab type and thickness include:

- the spanning capability of the deck-slab system (which governs beam spacing and in turn affects unit price of steel),
- the selection of temperature and shrinkage reinforcement (welded wire mesh) in accordance with the chosen concrete type and thickness,
- the volume of concrete used (thus, the cost),
- the mass of structural floor (which influences gravity column sizing, earthquake design loads, and lateral load resisting system sizing, and vibration characteristics, see Chapter 7),
- the fire resistance rating required[1.19,1.20,1.21],
- the sound transmission and impact noise rating, and
- the floor to floor height and/or clear height of floors.

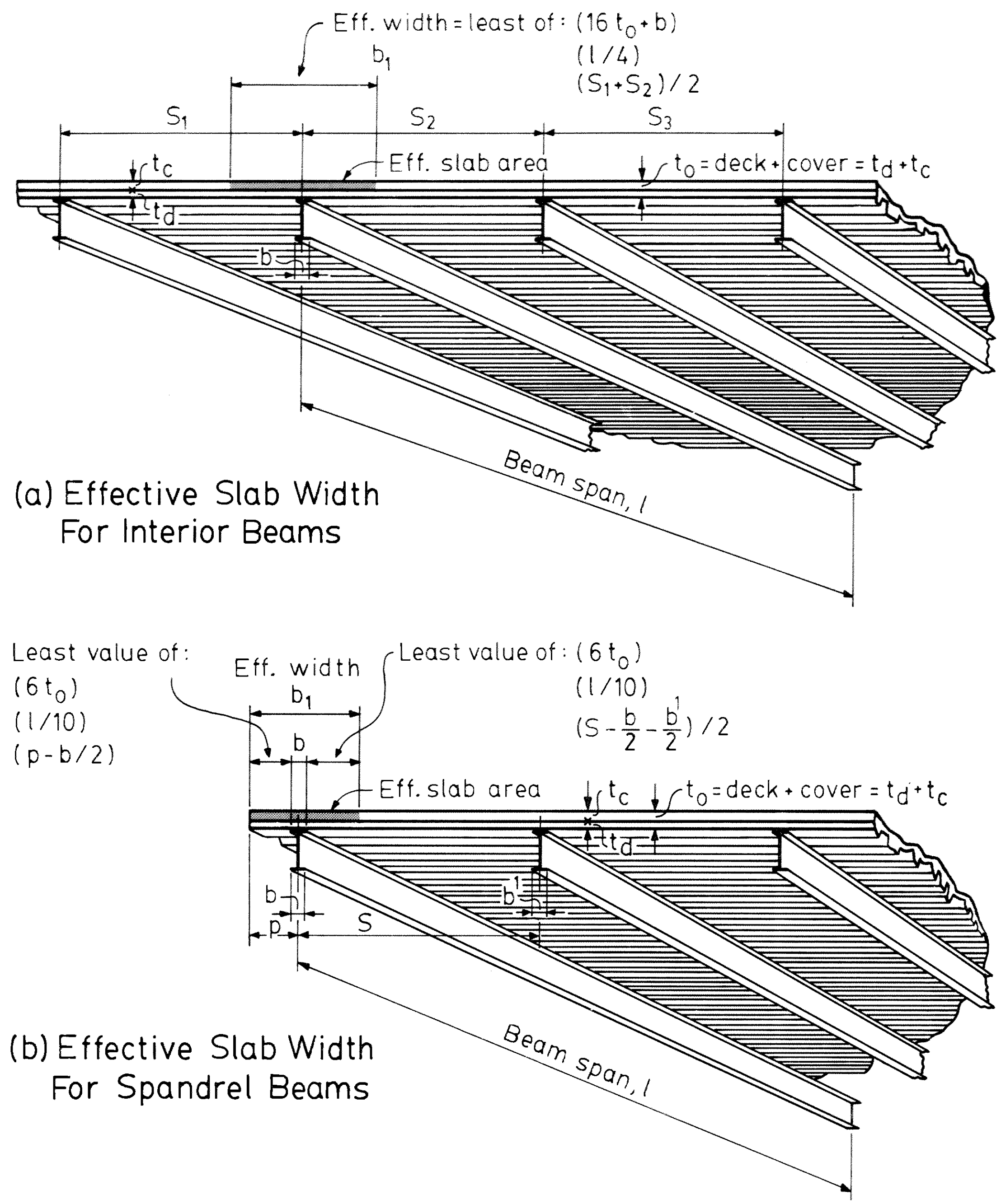


Figure 1.13
Effective Slab Width of Composite Members using Deck-Slabs

b) *Slab or Cover Slab Density*

Unfortunately, for designers whose projects range across the vast North American continent, general rules of thumb on concrete density selection may not be applied due to variation in availability and price of low density aggregate. Furthermore, project size and "local expertise" enter into the decision. A large project may provide sufficient incentive for a local ready-mix operator to deviate from his normal operations, assigning storage space to low density aggregates and becoming involved in a "special mix". Once the hurdle of the first project is out of the way, the "premium" for semi-low density concrete may approach the differential in aggregate costs. In many cases, a designer can still revert to normal density concrete for cover slab or slab construction, if the effect on total building cost becomes unfavourable using semi-low density concrete. The influences of slab concrete density include:

- the elimination of the requirement for sprayed-on fire protection to the underside of the deck-slab system, if slab thickness and density permit this substitution,
- the change in dead load in column design and earthquake lateral load resisting system design (insignificant if a slightly thicker low density slab has been selected, in lieu of the standard 65 mm cover slab, to provide a fire resistance rating),
- the inherent insulation value of low density concrete in a roof application, and
- the cost differential and degree of difficulty of concrete pumping and placing.

c) *Concrete Strength*

Concrete mixes are normally supplied in even 5 MPa strength increments. Concrete strengths of 20, 25 and 30 MPa are common. In general, concrete strength does not have much influence on the overall structural strength of the deck-slab system. However, it does have a direct bearing on shear stud capacity, and in some instances, on composite beam capacity or stub-girder capacity. For example, deeper composite beam assemblies which become slab-critical may benefit from higher strength concrete. Similarly, longer span stub-girder assemblies subject the deck-slab system to high shear, bending and compressive forces. They will likewise benefit from an increase in concrete strength.

d) *Reinforcing*

Slabs or cover slabs should be adequately reinforced to support all specified loads and to control cracking.

– *Shrinkage control reinforcement*

Of the several types of volume changes that occur in concrete during hardening, the most extensive is shrinkage due to dehydration. The most influential single factor governing the drying shrinkage of concrete is the unit water content, i.e. the amount of mixing water. Other factors such as cement content and the size, shape, and grading of aggregates are important but largely because of their effect upon the amount of water required to bring the mix to a "workable" consistency. It follows that any means of reducing the amount of water required for a workable mix will assist in cutting down shrinkage. Additional shrinkage crack control can be achieved by the use of shrinkage control reinforcement, commonly referred to as temperature and shrinkage reinforcement. In the case of solid slabs, CAN3-A23.3 rules on shrinkage reinforcement should be followed. In the case of deck-slab systems, some deck manufacturers provide information on minimum welded wire mesh reinforcement configurations appropriate for the respective cover slab thicknesses. Welded wire mesh, placed in the concrete, approximately 25 mm from the top surface, distributes shrinkage strains in a series of small cracks rather than permitting the accumulation of shrinkage strains over greater distances, permitting uncontrolled cracking. Cover slabs thicker than 65 mm, slabs expected to see heavier than normal office loading or slabs requiring better crack distribution, such as slabs which will receive a tile finish, should have more attention placed on mesh sizing and mesh placement. Shrinkage strains tend to accumulate in areas of least restraint. Such areas are thus vulnerable to cracking. (See also comments on reinforcement at beam girder joints which follow).

Although specific research verifying these suppositions has not been found, it is apparent that the use of a composite steel deck in fact provides considerable shrinkage restraint in the longitudinal direction, that is, in the direction of the deck span. For good quality concrete, properly placed and cured, cracks caused by shrinkage strain accumulation will occur perpendicular to the supporting beams. These cracks are magnified by negative bending of the deck-slab system caused by creep and shrinkage of the concrete and construction loading in the construction phase, and by superimposed in-service loads. Thus, chairing of the mesh over the girder supports provides assurance that the mesh is placed "as per plans and specs" but more importantly provides specific resistance to cracking at this critical location. This additional care in installation of mesh reinforcement provides very satisfactory results at a small cost premium.

Reinforcement of a deck slab system should not be deemed to replace the use of good quality structural concrete. However, in selecting reinforcement details for a deck-slab system, a designer must consider deck orientation, areas of major shrinkage restraint and local anomalies. Low rise large plan buildings usually involve multiple bays in each direction and some means of two directional crack control must be examined. Multi-storey steel framed structures frequently involve a clear span-to-core framing system. In a framing plan as shown in Fig. 1.14, composite deck plus mesh would normally control cracking in regions where deck flutes are parallel to the core. Major cracking would be expected over the girders and a solution to inhibit such cracking is proposed elsewhere in this chapter. Since this configuration is often used with cellular deck for electrical and communication services, it might be noted that the stiffer cellular units will usually produce reflection cracks at their edges, i.e. at the edges of the flat bottom plate, but because the module will usually be 1.5 to 2 m, the cracks will not usually be large enough to cause concern.

In a structural framing plan as shown in Fig. 1.17, composite deck supplemented by chaired mesh provides good crack control in the long direction of the structure. The relatively short dimension from core to exterior free edge (commonly 10-12 m) usually allows sufficient unrestrained shrinkage to avoid major cracks.

To reduce the effect of shrinkage cracks to a minimum, additional measures should be taken to ensure that:

- the concrete cover slab is treated as structural concrete,
- there is no segregation of mixed concrete during slab pour,
- the concrete surface is not impaired by over-working during floating and trowelling by causing excessive amount of fines and water to be brought up to the surface, and
- the concrete is properly cured, by maintaining temperature and moisture content at appropriate levels, before loading.

- *Slab reinforcement over beam-girder joints of simply supported floor members*

Two-directional considerations of composite construction incorporating a deck-slab system warrant some discussion. It is appropriate at this point to note the definition of a ***girder*** as a principal framing member normally supporting secondary ***beams*** and with the deck normally paralleling the girder. Further, it is assumed in the following discussion that the girder and its carrying beams are simply supported members without applied end moments. Reinforcement of a concrete cover slab over a girder is a subject which has been ignored in the bulk of composite research to date. There is a tendency for various crack phenomena to focus at the girder location. For example, there is frequently a discontinuity in the deck as there is in shear stud distribution if all floor members are compositely designed. Thus, shrinkage strains tend to accumulate here in the area of least restraint. Furthermore, there is negative flexural action at the girder caused by secondary beam deflection and thus end rotation in so called simple framing beam members. This end rotation is dependent upon the amount of superimposed load applied and is amplified by creep and shrinkage of the slab. The type of beam end connection will also influence the amount of end rotation, both before and after concrete placement. Shoring of beams during concrete placement can have a significant impact on

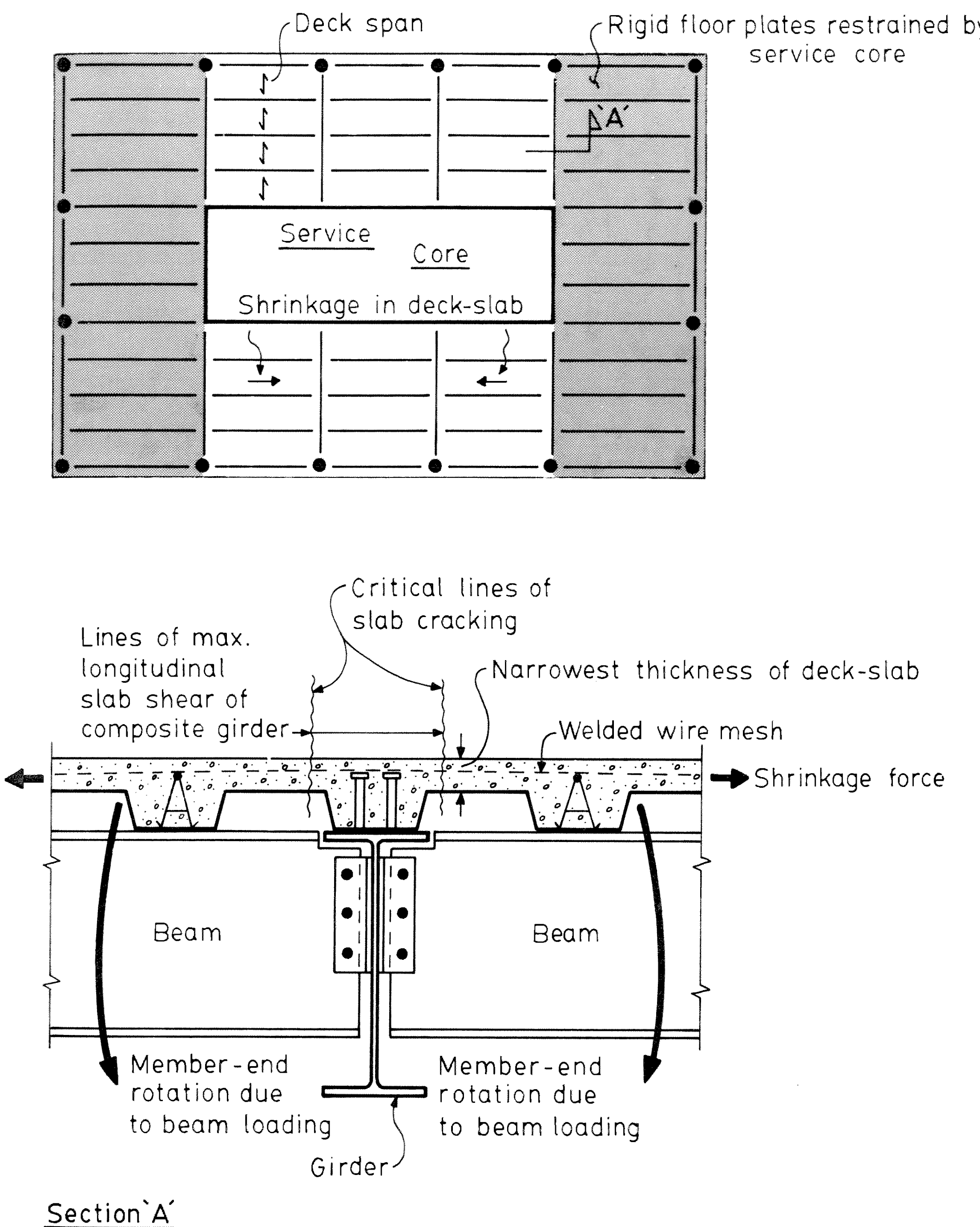


Figure 1.14
Example Floor Plan Showing Locations of Stress Concentration in Deck-Slab

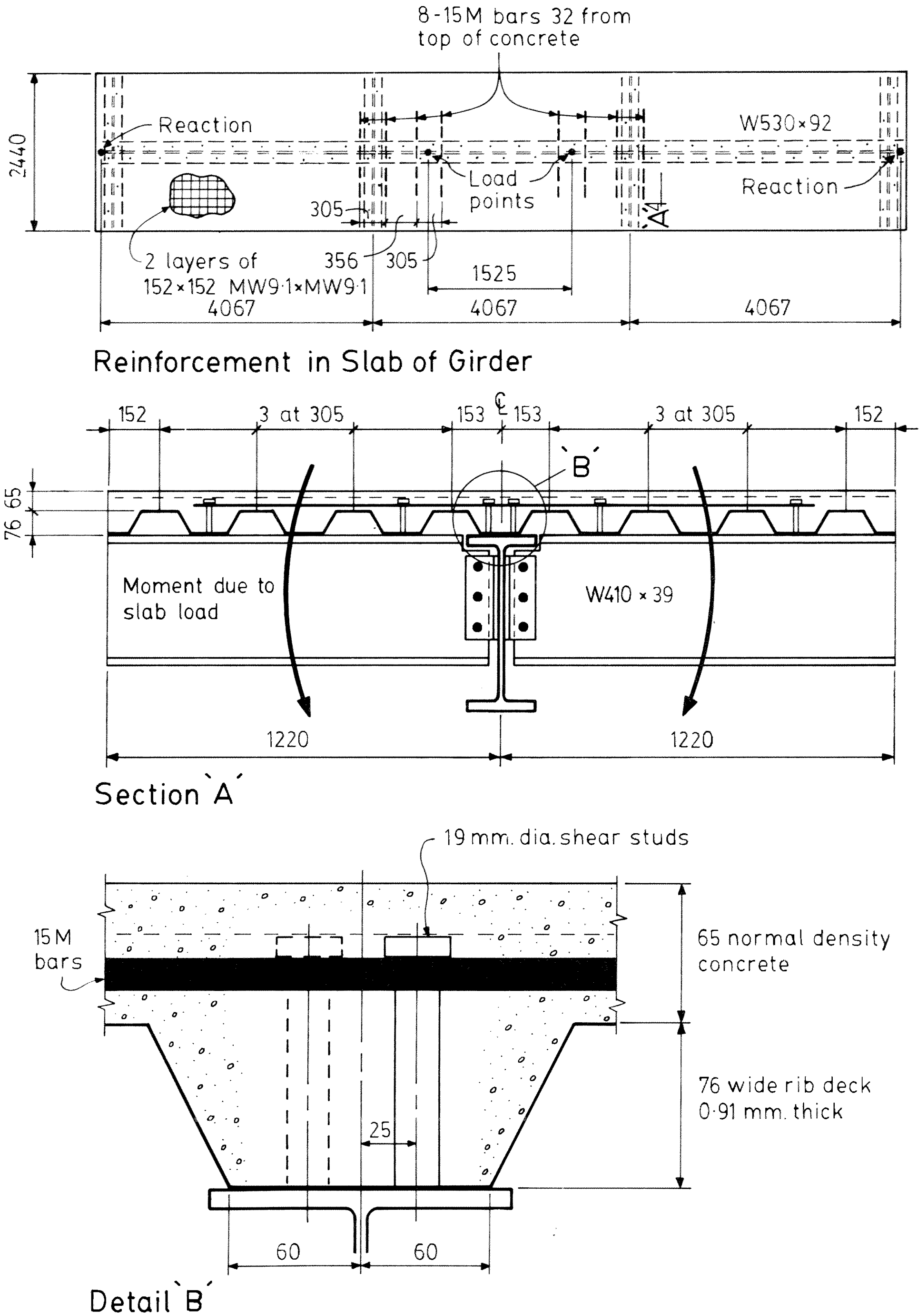


Figure 1.15
Composite Girder Test Specimen
(Tested at McMaster University)

slab cracking over girders, since all beam end rotation, albeit possibly a smaller angular rotation (due to stiffness of the composite section) than the initial end rotation during concrete placement in the unshored case, takes place after the concrete is set.

One additional factor, the tendency of longitudinal shear cracking of the slab above the girder due to compressive force in the slab, would further accentuate the probability of cracking if the slab (over girder) is left without a proper amount of longitudinal shear resisting reinforcement, Figure 1.14.

Recent research tests by Robinson[1.22] have shown that slab cracks above a composite girder can be reduced by using the detail shown in Figure 1.15. Testing of a girder with this reinforcing detail was compared to the test results of a girder with only minimal welded wire mesh reinforcement. Both girders reached and exceeded the factored moment capacity predicted by limit states design rules. However, the girder without additional rebars over beam-girder joints failed in a less ductile manner. Also, much wider longitudinal cracks appeared on the deck-slab surface of this girder as compared to the girder incorporating short length rebars over beam-girder joints.

Since there is no evidence of a practical quantitative procedure for computing the size and number of such reinforcing, a qualitative solution is therefore presented in Figure 1.16 for consideration[1.19].

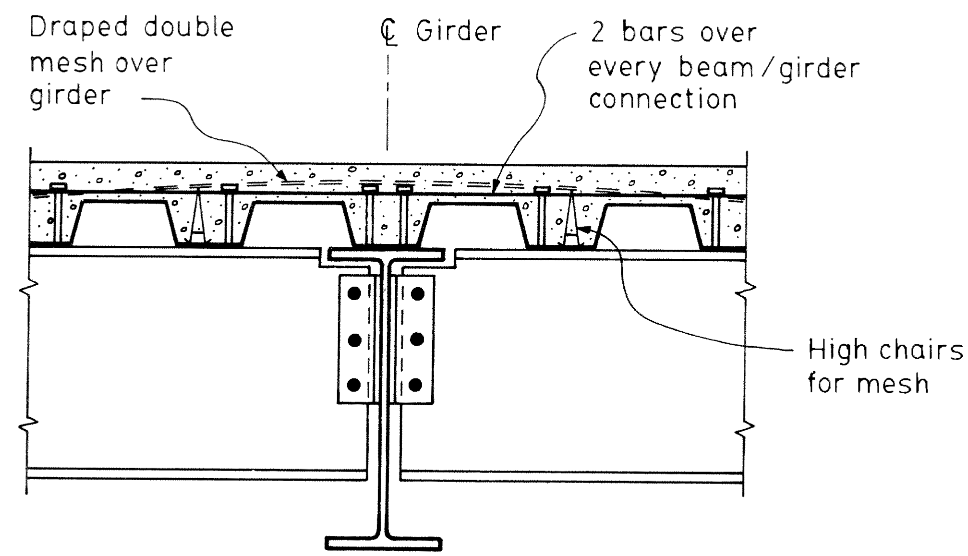


Figure 1.16
Proposed Deck-Slab Reinforced at
Beam-to-Girder Joints

– *Other crack control reinforcement*

Shrinkage stresses and diaphragm stresses tend to accumulate also at areas of high restraint or stress concentration such as the corners of service core walls and floor openings. Extra reinforcement in the form of short-length bars should be included where necessary. See Figure 1.17.

– *Structural reinforcement of slabs*

Design of reinforcement may be necessary to comply with localized structural requirements of slab reinforcing for composite action of floor members, such as stub-girders, structural diaphragm action or edge slab projections, Figure 1.18.

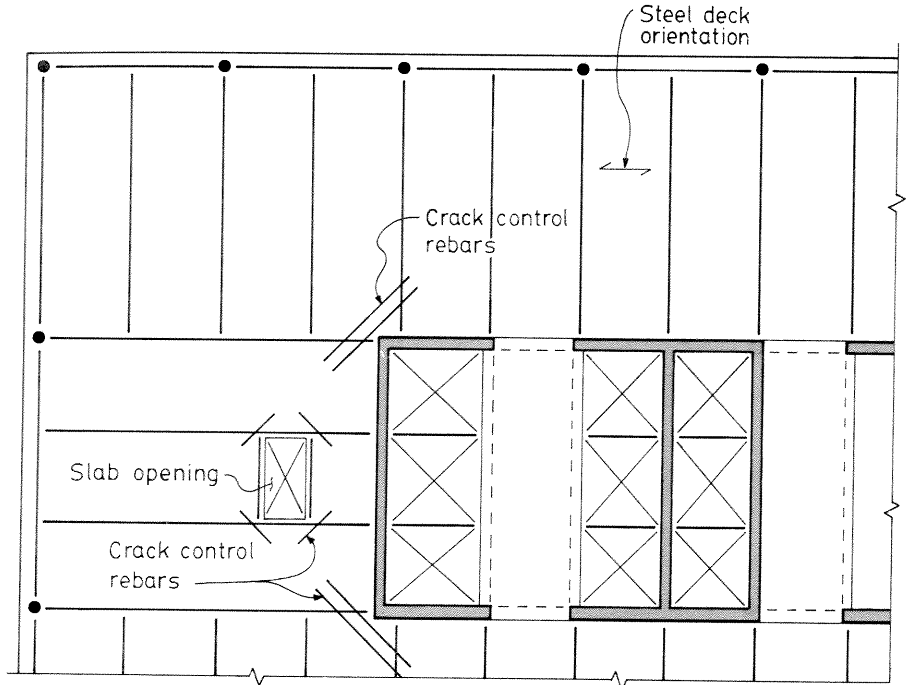


Figure 1.17
Crack Control Rebars in Deck-Slab Floors

e) *Other Considerations*

– *Expansion (and contraction) joints in composite steel-concrete structures*

Placement of expansion joints to completely separate structure segments is a function of structure configuration, construction sequence and building structural system. For example, industrial building structures exposed to the elements have been built close to 500 metres in length without expansion joints in the steel superstructure. Such a structure must be considered unique and outside the scope of this discussion. The absence of architectural finishes and the use of a substantial bracing system to resist both "natural" lateral forces and traction forces resulting from crane loadings would in all likelihood provide sufficient restraint against temperature strains in the steel structural frame.

The introduction of suspended floors and an insulated roof membrane, as in more common-place structures, simplifies joint requirements in some respects and adds complexity in others. A steel skeletal frame to be fully enclosed and occupied at constant temperature will only be subjected to significant temperature strains during construction and, at first glance, short term temperature strains are of little concern.

Many designers arbitrarily select a cutoff of 90-100 metres as the maximum length of a steel framed building enclosed for occupancy to be built without expansion joints. Since the sequencing of steel erection, and full enclosure and heating of the structure may be beyond the control of the designer, such a limitation is not unreasonable, although greater limits could be practically achieved for a long building, framed in the spring, and fully enclosed and heated before winter.

The steel skeletal frame is a known stable material with predictable thermal characteristics and with no time dependent dimensional changes. However, the introduction of concrete slabs or cover slabs adds a new dimension. Concrete shrinkage is inevitable, as discussed earlier. Some composite action in steel framed structures with concrete slabs or deck-slab systems is also inevitable whether or not the floor members are compositely designed. Steel deck-to-beam attachments, either in the form of arc spot welds, or more positive connection via stud shear connectors will produce some degree of composite action.

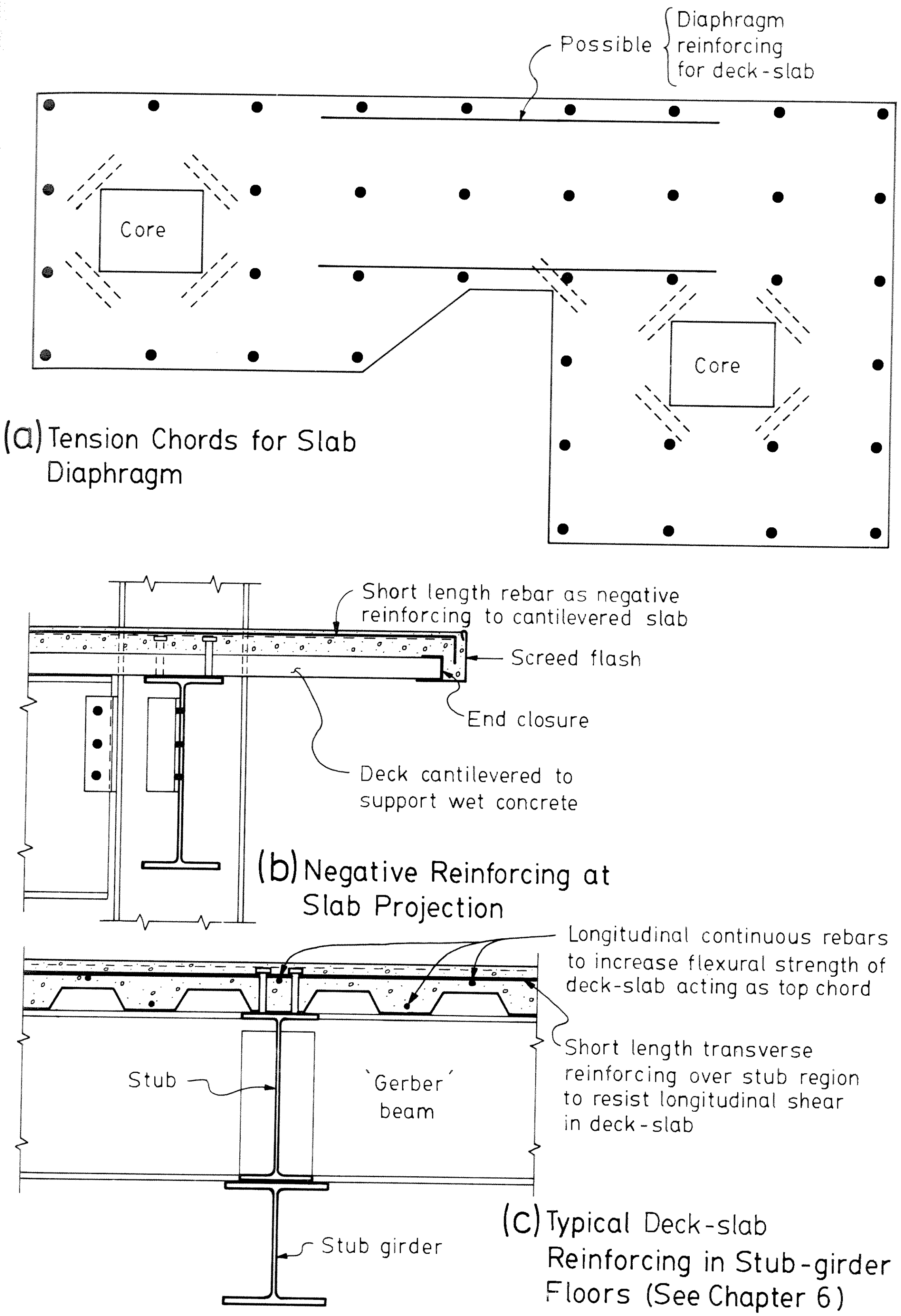


Figure 1.18
Examples of Structural Reinforcing in Deck-Slabs

Incremental attachment of a slab or deck-slab system to a steel framing system creates a natural resistance to concrete shrinkage and thus induces compressive forces to the top flanges of the steel supporting members, concrete shrinkage is thus reduced from a free shrinkage strain of about 600×10^{-6} to a restrained shrinkage strain of about 200×10^{-6} to 300×10^{-6}. The net result is crack distribution and deflection of the supporting beams.

The use of control strips, as used in a concrete structure to permit dissipation of time dependent shrinkage strains prior to placing the closure concrete slab (forming full continuity), is not completely applicable in steel structures due to the skeletal nature of the frame. Nevertheless, both control strips and control joints can be used effectively in deck-slab applications.

In unusually long or large plan area structures, control strips can be used, with the concrete simply stopped back during the pour. Rebar and deck continuity through the strip are maintained, with the control strip cover slab placed later. Minimum reinforcing through the control joints is required to ensure full slab continuity of compositely designed members.

Saw cutting of the cover slab after concrete set-up, followed later by filling with resilient sealers can be used effectively where slab cracks would be considered to impair future serviceability. Care must be taken not to destroy the effective slab width if composite members are involved.

These measures are relatively costly and disruptive to construction and should only be considered when it has been determined that the reinforcing details and good concrete mixing, placing and curing practices mentioned previously will not produce a satisfactory structure.

– *Construction Methods*

One final consideration in the construction of deck-slab systems in a composite floor is the design of shoring in cases where composite members are shored. Design calculations should be made to determine the size of shoring members, sequence of shoring, effects of shoring load on the structure under construction and effects of shore removal on structural slabs and structural members.

1.5 OPEN AIR PARKING STRUCTURES

Structural framing systems for open air passenger car parking structures have been built with composite deck-slabs, with embossed steel decks acting as principal positive moment reinforcement. Such applications require careful consideration vis-à-vis drainage, concrete cracking and steel corrosion. Throughout southern Canada, heavy use of road salt rapidly accelerates the deterioration of concrete slabs of highway structures and, indirectly, parking decks. Many parking structures without proper protection, in the space of a number of years, are now encountering high maintenance costs for major repair or even replacement.

Should steel deck be considered as a framing component in a parking garage, the use of a heavy zinc coating will minimize corrosion from the underside. There is little evidence to show that the zinc coating will protect the deck from salt attack from above. Therefore, additional steps should be taken. Waterproofing of slabs and good drainage are also required. In addition, asphalt topping in areas of heavy wear and high concentration of salt such as approach ramps (often with additional slab reinforcing) are necessary. A thicker cover slab than that used for office occupancies should be considered along with additional reinforcement.

Finally, transverse rebars over girder-beam joints may be necessary to control slab cracking. It is only under these conditions that a composite deck-slab system should be considered as a viable solution for parking garages. The steel deck may also be used as a stay-in-place form. The above considerations would then be more visual and maintenance oriented, rather than structural in nature.

REFERENCES

1.1 "Steel Structures for Buildings – Limit States Design", CAN3-S16.1-M84.

1.2 "Zinc Coated Structural Quality Steel Sheet for Steel Deck", CSSBI 101M-84.

1.3 "Metric Zinc Coated (Galvanized) Sheet Steel for Structural Building Products – Technical Bulletin No. 6", CSSBI, Oct. 1979.

1.4 Iyengar, S.H., and Zils, J.J., "Composite Floor Systems for Sears Tower", AISC Engineering Journal, Third Quarter, 1973.

1.5 Ekberg, Jr., C.E., and Schuster, R.M., "Floor Systems with Composite Form-Reinforced Concrete Slabs", Final Report, International Association for Bridge and Structural Engineering, 8th Congress, New York, Sept. 1968.

1.6 Porter, M.L., and Ekberg, Jr., C.E., "Summary of Full-Scale Laboratory Tests of Concrete Slabs Reinforced with Cold-Formed Steel Decking", Preliminary Report, International Association for Bridge and Structural Engineering, 9th Congress, Amsterdam, The Netherlands, May 1972.

1.7 Porter, M.L., and Ekberg, Jr., C.E., "Design Recommendations for Steel Deck Floor Slabs", Journal of the Structural Division, Vol. 102, Nov. 1976.

1.8 Schuster, R.M., "Composite Steel-Deck Concrete Floor System", Journal of Structural Division, ASCE, Vol. 102, May 1976.

1.9 Salmon, C.G., and Fisher, J.M., Chapter 3 – Applications of Light-Gauge Steel in Composite Construction, "Handbook of Composite Construction Engineering", Van Nostrand Reinhold Company, 1979.

1.10 Temple, M.C., and Abdel-Sayed, G., "Fatigue Experiments on Composite Slab Floors", Journal of the Structural Division, July 1979.

1.11 Suleiman, R.E., "Behaviour of Composite Slabs Subjected to Repeated Point Loading", Master of Applied Science in Civil Engineering Thesis, University of Waterloo, 1983.

1.12 Porter, M.L., and Ekberg, Jr., C.E., "Behavior of Steel-Deck Reinforced Slabs", Journal of the Structural Division, March 1977.

1.13 "Structural Diaphragm Design, Steel Roof Deck", Supplementary Technical Publication No. 1, fourth edition, Robertson Building Systems, 1982.

1.14 "Steel Deck Shear Diaphragm Design Manual", Westeel-Rosco Limited, 1975.

1.15 "Diaphragm Action of Cellular Steel Floor and Roof Deck Construction", CSSBI Information Bulletin No. 3, Dec. 1972.

1.16 Luttrel, L.C., "Steel Deck Institute Diaphragm Design Manual", First Edition, 1981.

1.17 Bryan, E.R., and Davies, J.M., "Steel Diaphragm Roof Decks", Granada Publishing Limited, 1981.

1.18 "Standard for Composite Steel Deck – Draft", CSSBI, May, 1984.

1.19 Ritchie, J.K., and Chien, E.Y.L., "Innovative Designs in Structural Systems for Buildings", Canadian Structural Engineering Conference, 1978.

1.20 "List of Equipment and Materials – Vol. II Building Construction", Underwriters' Laboratories of Canada, Scarborough, Ontario, Canada.

1.21 "Fire Resistance Directory", Underwriters Laboratories Inc., Northbrook, Illinois, U.S.A.

1.22 Robinson, H., "Resistance to Longitudinal Cracking in Composite Girders", McMaster University Report, October 1982.

CHAPTER 2

2.0 HEADED STUD SHEAR CONNECTORS FOR COMPOSITE FLOOR MEMBER DESIGN

2.1 INTRODUCTION

Early steel buildings utilized encasement concrete to provide fire protection and to achieve composite structural interaction. Steel beams without mechanical shear connectors were found to act compositely with concrete encasement, provided that only static loads were applied, and only when the shear stress at the interface of steel and the encasing concrete did not exceed the bond strength. Although concrete encasement offered some advantages, two distinct disadvantages, cost of forming and additional dead load, encouraged a trend to use formed flat slabs without encasement. Early tests demonstrated that there was considerable bond strength between the unpainted surface of top flange of steel shape and a concrete slab. However, such bond strength was not as reliable as in the encased beam case, due to the lack of positive vertical attachment, and the limited amount of shear transfer. As a result, embedded mechanical shear connectors evolved.

Since the early 1930's, numerous types of mechanical shear connectors have been tested by a large number of researchers around the world[2.1]. Such mechanical connectors as spiral shear connectors, T-connectors, channel connectors, hook connectors, angle connectors and more, were found to be structurally effective, though not necessarily economically viable.

Several research studies on stud shear connectors were carried out by Viest and Thurlimann[2.2,2.3,2.4], beginning in 1954. The types of shear studs tested included bent studs and straight studs with upset heads (the common Headed Stud or simple shear stud of today). Tests carried out included static and fatigue tests of studs in push-out specimens, and static and fatigue tests of studs in solid concrete slabs. In 1965, a series of beam and push-out tests were reported by Slutter and Driscoll[2.5], who developed a functional relationship between the shear connector strength and the concrete compressive strength. These researchers also developed a method of calculating ultimate bending capacity of a composite beam with weak shear connections (or partial shear connection).

2.2 STRENGTH OF STUD SHEAR CONNECTORS EMBEDDED IN SOLID CONCRETE

By 1971, the capacity and behaviour of headed stud shear connectors embedded in solid concrete slabs with both normal density and semi-low density concretes were well established and reported by Ollgaard, Slutter and Fisher[2.6]. The following equation was derived:

$$q_u = 0.5\, A_{sc} \sqrt{f'_c\, E_c} \qquad 2.1$$

where q_u = ultimate strength of a stud connector (N)
A_{sc} = normal area of stud shear connector (mm^2)
E_c = modulus of elasticity of concrete (MPa)
f'_c = specified concrete compressive strength at 28 days (MPa)

When used with limit states design as in S16.1 the factored ultimate shear resistance q_r of a stud shear connector embedded in solid concrete can be expressed as

$$q_r = 0.5\, \phi_{sc}\, A_{sc} \sqrt{f'_c\, E_c} \qquad 2.2$$

where $\phi_{sc} = 0.8$ = performance factor for shear connectors.

We can see that stud connector strengths are given in terms of apparent shear strengths, although connectors generally exhibit a tension failure in beam tests. As concrete pushes against the stud, the stud eventually begins to bend over and develop a tensile resistance(2.7); therefore, a limiting value of 415 ϕ_{sc} A_{sc} (the tensile strength of the commonly available stud is 415 MPa), is given in S16.1.

$$q_r = \text{the lesser of } 0.5\, \phi_{sc}\, A_{sc} \sqrt{f'_c E_c}, \text{ and } 415\, \phi_{sc}\, A_{sc} \qquad 2.3$$

where E_c may be expressed as

$$w_c^{1.5}\, 0.043 \sqrt{f'_c} \qquad 2.4$$

as given by CAN3-A23.3-M77(2.8) and the term w_c = mass density of concrete (kg/m^3).

For q_r values of stud connectors of various diameters embedded in several types and strengths of concrete, see Table 2.1.

TABLE 2.1 FACTORED SHEAR RESISTANCES OF END WELDED HEADED STUDS, q_r, FOR SOLID SLABS AND COVER SLABS WITH WIDE-RIB PROFILE DECKS.

Stud Diameter in	Stud Diameter mm	Mass Density of Concrete kg/m^3	Factored Shear Resistance$^+$, q_r in kN for Various Concrete Strengths, f'_c: 20 MPa	25 MPa	30 MPa
	12	2300	29.5	34.8	37.5
		1850	25.0	29.6	33.9
½	(12.7)	2300	33.0	39.0	42.1
		1850	28.0	33.1	38.0
⅝	(15.9)	2300	51.6	61.0	65.7
		1850	43.8	51.8	59.4
	16	2300	52.4	61.9	65.9
		1850	44.5	52.6	60.3
¾	(19)	2300	74.3	87.8	94.6
		1850	63.1	74.6	85.5
	20	2300	81.9	96.8	104.3
		1850	69.5	82.2	94.2
	22	2300	99.0	117	126.2
		1850	84.1	99.4	114
⅞	(22.2)	2300	101	119	129
		1850	85.9	101	116

$^+$ $q_r = 0.5\phi_{sc}A_{sc}\sqrt{f'_c\, E_c} \leq 415\phi_{sc}A_{sc}$
where $E_c = w_c^{1.5} 0.043\sqrt{f'_c}$, and $\phi_{sc} = 0.8$

Although headed studs in composite floor members generally take shear forces, they may also be used to carry co-existing tensile forces such as in the case of stub-girder construction. Information on tensile, shear, and combined tensile and shear behaviour of headed studs embedded in concrete may be found in work by McMackin, Slutter and Fisher(2.9). This work led to the publication of some comprehensive and design-oriented references(2.10,2.11).

The above shear connector tests dealt with the situation where the failure of stud connectors occurred either by the stud pulling out of the concrete or by shearing of the connectors. However, if very thin steel beam flanges are used, a reduction of ultimate shear capacity can be observed, and a third failure mode may occur in the form of pulling-out of connectors from the thin flange(2.25). For this reason, S16.1 stipulates that the stud diameter shall be limited to 2.5 times the thickness of the part to which it is welded, unless a lesser thickness can be justified. A practical consideration also bears on this thickness/diameter relationship. The energy required to weld larger studs may be excessive for thin flange material, particularly where additional power is required in situations where the studs are applied through the floor deck, and can result in the stud burning through the flange.

The American Institute of Steel Construction specification waives the restriction on stud diameter to flange thickness ratio, if the stud is located directly over the beam web. In the authors' opinion, a stiffer stud may result and the risk of burn-through is reduced; however, the practical field application problem of ensuring that the stud coincides with the web below, and the different welding equipment setting for stud locations off the beam web, may reduce the potential benefits of this approach.

2.3 STRENGTH AND BEHAVIOUR OF STUD SHEAR CONNECTORS USED WITH DECK-SLAB SYSTEMS

When steel deck flutes are oriented parallel with a steel girder, steel-concrete interaction can be achieved either by discontinuing the deck above the girder top flange, allowing stud shear connectors to be applied directly to the flange or with stud shear connectors welded through the steel deck if a steel deck flute coincides with the girder top flange. With the exception of certain limitations such as when using very narrow deck flutes, shear values assigned to studs may be the full solid values, in accordance with design rules as described in Section 2.2. However, when a steel deck is placed with flutes perpendicular to a steel member, the behaviour of weld-through stud shear connectors embedded within concrete ribs, and the appropriate shear value assigned may differ substantially from that of the previous case. Stiffness of the composite beam assembly may also vary from the "deck parallel" case. The following discussion of stud connector strength and behaviour are particularly addressed to the "deck perpendicular" case.

Stud Connectors used with Deck-Slab Incorporating Narrow-Rib Decks.

The ultimate shear capacity of stud connectors in a deck-slab system is a function of the rib geometry. This relationship was first identified and reported by Robinson(2.12). He deduced that the degree of interaction achieved and particularly the mode of cracking of the deck-slab was largely influenced by the width to height ratio of the concrete ribs formed by filling steel deck flutes with concrete. His study and several others(2.13,2.14,2.15,2.16) produced test results of stud performances, using narrow-rib profile decks, i.e. the average width of the concrete rib divided by the height of the deck is less than 2. These research tests have also provided sufficient data to illustrate the fact that, under the working load condition, there is usually no significant reduction of composite beam stiffness by using a deck-slab instead of a solid concrete slab. The ultimate factored shear resistance values of studs for use with 40 mm (nominal depth) decks as published in Table 8 of S16.1, were obtained as a direct result of some of the above research. Deeper decks with narrower ribs exhibited a substantial decrease in stud shear strength. This reduction in strength can be attributed primarily to punch-through or cracking of the concrete ribs encasing the studs. The fact that the ultimate shear strength of studs in a deck-slab system may be increased by increasing the width of the push-specimen was also noted by Fisher(2.17) through tests by Inland-Ryerson(2.18).

Stud Connectors used with Deck-Slab Incorporating Wide-Rib Decks

The calibration of stud connector shear strength versus concrete rib geometry and concrete type was provided by Fisher[2.17]. Using a factor of safety of 2.0 against flexural failure of test beams, Fisher proposed a stud shear connector allowable load formula,

$$Q_{rib} = 0.5 \frac{W_{rib}}{t_d} Q_{sol} \sqrt{\frac{E_{c\text{-}l}}{E_{c\text{-}n}}} \qquad 2.5$$

where Q_{rib} = Allowable load for stud shear connector embedded in a concrete rib,
Q_{sol} = Allowable horizontal stud shear resistance when a stud connector is embedded in solid slab of normal density concrete,
W_{rib} = Average rib width,
t_d = Rib height, or deck height, and
$E_{c\text{-}l}$, $E_{c\text{-}n}$ = Modulus of elasticity of low density and normal density concrete respectively.

Since the same factor of safety should apply to both Q_{rib} and Q_{sol}, equation 2.5 can be rewritten as,

$$q_{u(rib)} = 0.5 \frac{W_{rib}}{t_d} q_u \sqrt{\frac{E_{c\text{-}l}}{E_{c\text{-}n}}} \qquad 2.6$$

where $q_{u(rib)}$ = factored ultimate strength of a stud shear connector embedded in a concrete rib,
q_u = As defined by equation 2.1 using a normal density concrete.

In a case where a wide-rib profile deck is used with a normal density concrete, equation 2.6 can be expressed as,

$$q_{u(rib)} = q_u$$

assuming $\frac{W_{rib}}{t_d} = 2$ and $q_{u(rib)} \leq q_u$

Fisher also pointed out that equation 2.5 can only be used when,
- steel decks of up to 76 mm deep are used,
- the diameter of studs is less than or equal to 20 mm,
- the extension of the head of the studs above deck flutes is about 2 × diameter of the studs, and
- top rib width is equal to or greater than the bottom rib width.

It is also important to know that stud shear values obtained from equation 2.5 apply to situations with,
- single-stud-per-rib type of connection, and
- interior beam conditions (see following explanations).

Following 17 full-scale beam tests (which were carried out at Lehigh University), and the study of 58 existing beam test results (reported by other investigators), an improved version of the stud shear capacity formula (as compared to equation 2.5) was proposed by Grant, Fisher and Slutter[2.19].

$$Q_{rib} = \frac{0.85}{\sqrt{N}} \left(\frac{H-t_d}{t_d}\right) \left(\frac{W_{rib}}{t_d}\right) Q_{sol} \leq Q_{sol} \qquad 2.7$$

where N = number of stud connectors embedded in a concrete rib,
H = height of stud connector,
t_d = height of steel deck.

In the above formula, two more variables are introduced for the determination of stud shear capacity, i.e. the ratio of stud embedment length to deck height and the number of studs in a concrete rib (Fig. 2.1). Through the use of equation 2.7, the scatter of $M_{u(tested)}/M_{u(theoretical)}$ ratios of all the test beams can be brought to near unity. In addition, Fisher claimed that push-off tests reported by Iyengar[2.20] and further tests carried out at University of Texas[2.21] and at Lehigh University[2.22] verified the effects of multiple-stud grouping and stud embedment lengths.

It can be shown that formula 2.7 can be rewritten as,

$$q_{r(rib)} = \frac{0.85}{\sqrt{N}} \left(\frac{H-t_d}{t_d}\right) \left(\frac{W_{rib}}{t_d}\right) q_r \leq q_r \qquad 2.8$$

where $q_{r(rib)}$ is the factored ultimate shear resistance of the connector in a concrete rib and q_r is as defined in equation 2.3.

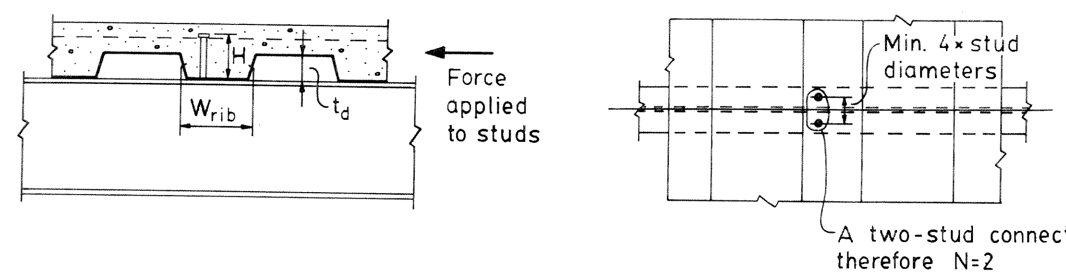


Figure 2.1
Multiple Stud Application in a Wide-Rib Profile Deck ($W_{rib}/t_d \geq 2$)

The example, given below, is intended to illustrate the use of equation 2.8.

DESIGN DATA:
N = 1, (W_{rib}/t_d) = 2, wide-rib profile, H = 120, t_d = 76 mm

SOLUTION:

$$q_{r(rib)} = \frac{0.85}{\sqrt{1}} \left(\frac{120-76}{76}\right) (2)\, q_r$$

$$q_{r(rib)} = 0.98\, q_r \simeq q_r$$

Provided that $(H-t_d)/t_d$ is not less than approximately 0.6, the S16.1 method of computing stud shear values of studs embedded in deck-slab of wide-rib profile decks would produce results equal to that computed for solid slabs, i.e.

$$q_{r(rib)} \simeq q_r$$

2.4 SLAB EDGE DISTANCES AND DISTANCES BETWEEN STUDS IN PAIRS

The discussion to this point has centered on studs applied singly on interior framing members. Every building has edge conditions, at the perimeter, at atriums, and/or interior stairwells, and at other conventional openings for vertical services. When compositely designed members are used under such edge conditions, the capacity of the member may be impaired by the effective width of slab available. This situation is covered by S16.1 and by design tables.

Other implications of edge conditions are not so clearly defined or understood. For example, shear studs applied to spandrel beams or girders, where the deck-slab does not project beyond the steel flange, may require evaluation of the impact of this edge condition on the capacity of shear connectors as follows:

- The reduction of shear cone due to the proximity of a free edge affects shear resistance of studs in both solid-slab and deck-slab systems.
- The effect of encroachment of a free edge on rib failure, Fig. 2.2, when a narrow-rib profile deck is chosen.
- The occurrence of a free edge reduces the failure plane in the cover slab, Fig. 2.3. This type of failure is known to occur in push-out tests of studs in deck-slabs incorporating wide-rib profile deck with a thin cover slab, but less frequently in beam tests.

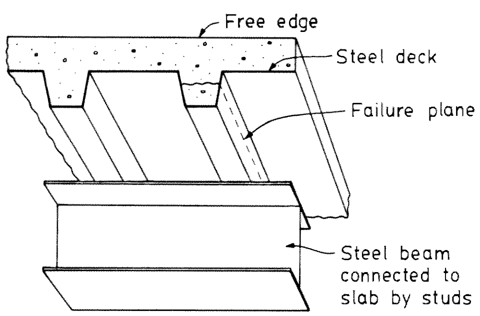


Figure 2.2
Effect of Free Edge on Rib Strength

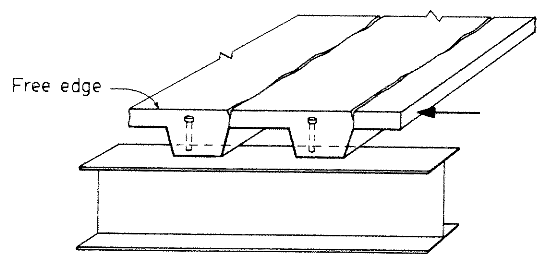


Figure 2.3
Effect of Free Edge on Cover-Slab Strength

A comprehensive test program conducted by McMackin et al[2.9] to determine the behaviour of headed studs, when embedded in plain unreinforced concrete, provided sufficient data to reliably forecast anchor strengths of studs in both shear and tension. Design aids[2.10,2.11] provided by stud producers, facilitate the computation of ultimate anchor strengths for studs embedded in plain solid concrete. Recent push-out tests by Robinson[2.23] for stud connectors in a wide-rib profile deck-slab designed to simulate a spandrel beam edge condition (although in the absence of a final report) have indicated that Fig. 2.4 can be used to produce conservative values for connector designs. Other edge distances for studs in shear can be conservatively derived from the design information above. See Fig. 2.5 for more detail.

When shear studs are placed on a 'girder' with concrete ribs parallel to the 'girder' span, the amount of concrete cover at the sides of studs generally does not affect stud shear strength. Thus it is not a critical factor governing the composite member strength[2.30]. In this situation, studs may be installed as close as is permitted by field welding practice towards the wall of deck flutes. However, a proposed minimum edge dimension for practical reasons is shown in Fig. 2.5c.

In a Hollow Composite 'beam' situation, see Section 4.2, when more than one stud is provided in a concrete rib, the respective shear cones overlap; as a result, the ultimate shear resistance per stud is decreased. The amount of overlap of shear cones increases when transverse spacing between studs decreases, and hence the reduction of shear resistance. It is interesting to note that multiple studs

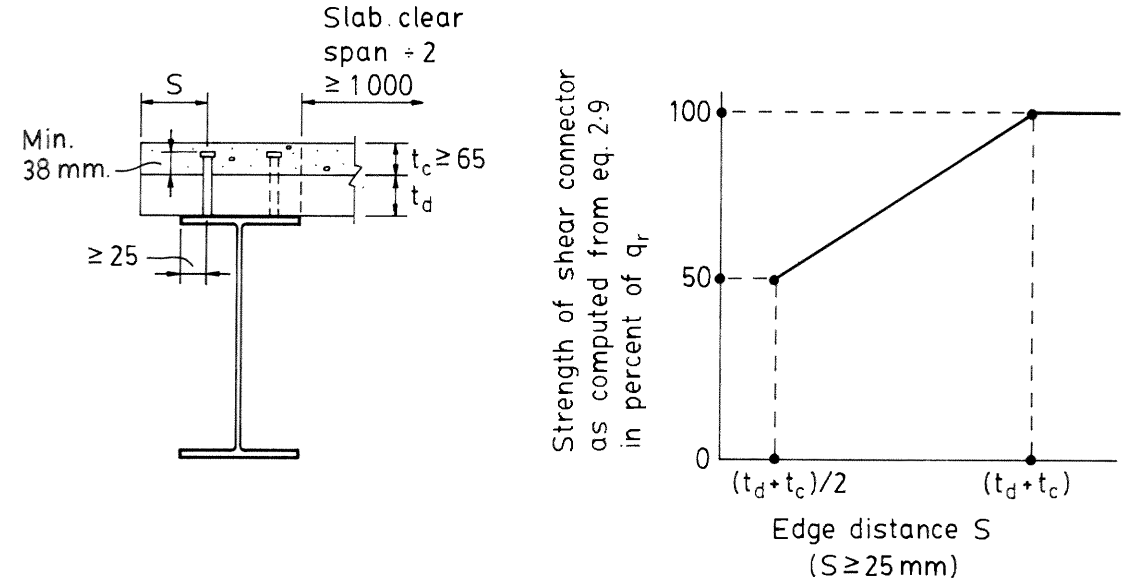


Figure 2.4
Proposed Stud Shear Resistance in
Hollow Composite Spandrel Beams

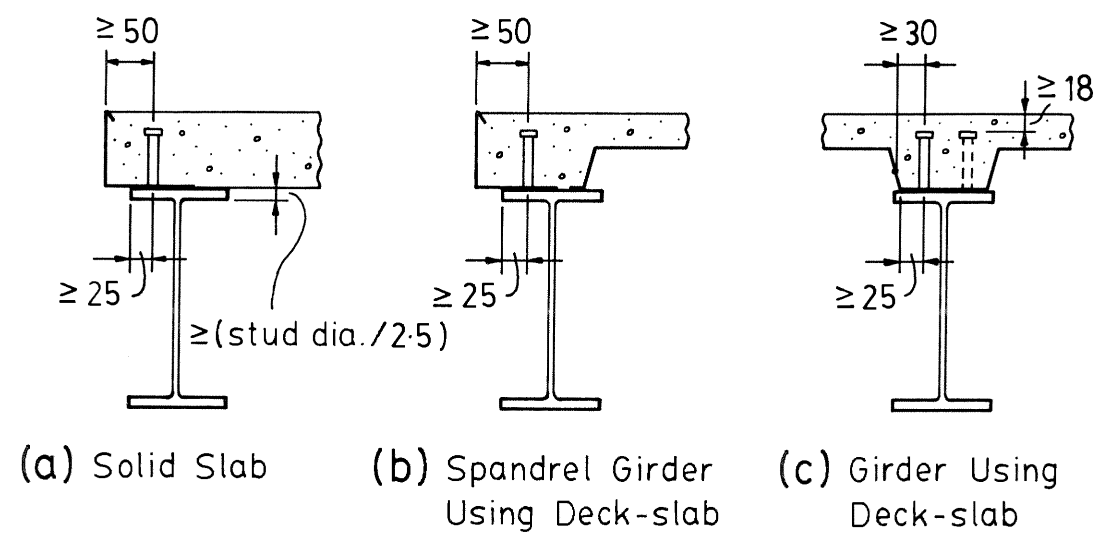


Figure 2.5
Proposed Minimum Edge Distances
for Stud Shear Connectors

tested by Grant et al[2.19,2.24] had transverse spacing of approximately 100 mm; however, a slight variation of transverse spacing is not known to significantly affect the ultimate shear resistance of multiple stud groups. Furthermore, minimum transverse spacing of studs in an actual application can also be determined by physical restrictions such as welding equipment, stud size and stud layout. It can be shown that a distance of about 75 mm between a pair of 19 mm studs can be conveniently achieved. Figure 2.1 illustrates the minimum recommended stud transverse spacing to be used with the multiple stud formula 2.8.

For hollow composite beams incorporating deck-slabs of wide-rib configuration, the placement of shear studs to the side of the rib closest to the support of the beam (or nearest zero moment location) appears to improve shear stud capacities[2.23]. This will reduce the possibility of studs punching through the side of the concrete rib. Hence, a minimum edge distance to minimize the effect of punch-through failure of studs (applicable only to single-stud-per-rib condition in wide-rib profile deck) is proposed in Fig. 2.6.

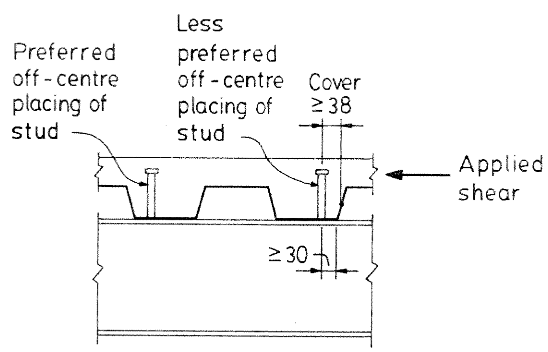


Figure 2.6
Minimum Cover to Resist Punch-Through Failure of Stud Connection (for Single Stud per Rib Connections)

2.5 SPACING OF STUD SHEAR CONNECTORS IN COMPOSITE BEAMS AND GIRDERS INCORPORATING ROLLED AND WELDED H-SHAPES

Information given in Sections 2.2 and 2.3 permits a reasonable estimate of ultimate strengths of stud connector groups either in single stud groups or multiples, in solid slabs or in deck-slabs. In addition, Section 2.4 provides general recommendations of minimum edge distances for studs and minimum spacing of studs within a multiple stud group. One would then compute the sum of horizontal shear between the points of maximum and zero moment, for members designed applicable to one of the three cases; neutral axis in concrete (Full Connection), neutral axis in steel (Full Connection) or neutral axis in steel (Partial Connection). See Section 4.4 for full details. Studs are then placed into connector groups (either singly or in multiples if necessary) and are then spaced along the span of the composite beam according to the rules provided and explained below.

Composite Members under Uniformly Distributed Load

Figure 2.7 shows equilibrium diagrams for a uniformly loaded composite member together with the ultimate stress distribution at the maximum moment location (which, in these cases, occurs at mid-span). The value of horizontal shear at the interface of steel and concrete between maximum and zero moment locations can then be determined from the concrete stress block (which can be obtained using procedures outlined in 4.4). At first glance, it might be concluded by elastic analysis that connectors would be spaced in accordance with the shear diagram for the case of uniform loading. In this instance, the variation of stud spacing to satisfy interface shear would resemble a so called "triangular" distribution. However, it has been shown[2.5] that redistribution of loads on shear connectors prior to ultimate failure permits shear studs to be spaced uniformly and still obtain the

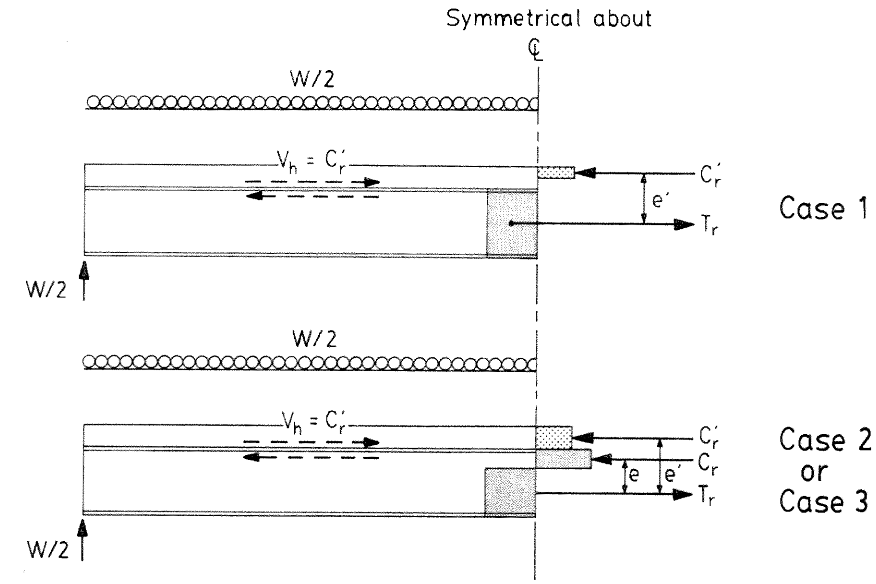


Figure 2.7
Equilibrium of a Uniformly Loaded Composite Member

desired ultimate beam strength. One must also recognize that code-prescribed "uniform" loading does not often occur in many real structures. Therefore it is quite evident that uniform spacing of connector groups is structurally appropriate as well as desirable during detailing, particularly when a deck-slab is chosen as the top flange of the composite section.

Composite Members with Heavy Point Loads

Certain loading conditions, such as the occurrence of heavy point loads or heavy partial-uniform loading, see Figure 2.8(a) and (b), may require a closer connector spacing over part of a member length. The procedure, given by S16.1 Cl. 17.4.8 (as shown below) should be followed for computing the distribution of connectors under this loading condition. The number of connectors required between the point of zero moment and its adjacent point of concentrated load or heavy local uniform load, shall be not less than n′.

$$n' = n\left(\frac{M_{f1} - M_r}{M_f - M_r}\right) \qquad 2.9$$

when n = the number of shear connectors to be distributed within the region of zero moment and the nearest maximum positive moment (M_f).
M_{f1} = factored positive moment at the location considered.
M_r = factored moment resistance of steel section alone.
M_f = maximum positive factored bending moment.

The value of n is represented by V_h/q_r, where V_h = factored shear force at the steel-concrete interface, between the point of maximum moment and the point of zero moment, and q_r = factored shear resistance of a connector. The computation of V_h is presented in Section 4.4 and the formulas for computing q_r, or $q_{r(rib)}$, are given in Equations 2.3 and 2.8. A worked example on stud distribution in a composite girder is given at the end of Chapter 4.

2.6 STUD APPLICATION AND QUALITY CONTROL

Stud shear connectors are now used on virtually every commercial steel building in Canada. With the advent of limit states design and the use of wide-rib profile decks, stud shear connectors are relied upon to carry very heavy loads. Thus, using a partial-connection composite beam and girder design, it is not uncommon to find only 16 to 20 studs on a 9 metre span floor member spaced at 3 metre centres. As a result, we rely on 8 to 10 stud shear connectors, at each side of the point of maximum moment, to carry the maximum shear force in such a member. Consequently, a designer must define sufficient installation and inspection procedures in the construction specification to ensure that each stud installed will deliver its assigned ultimate shear resistance.

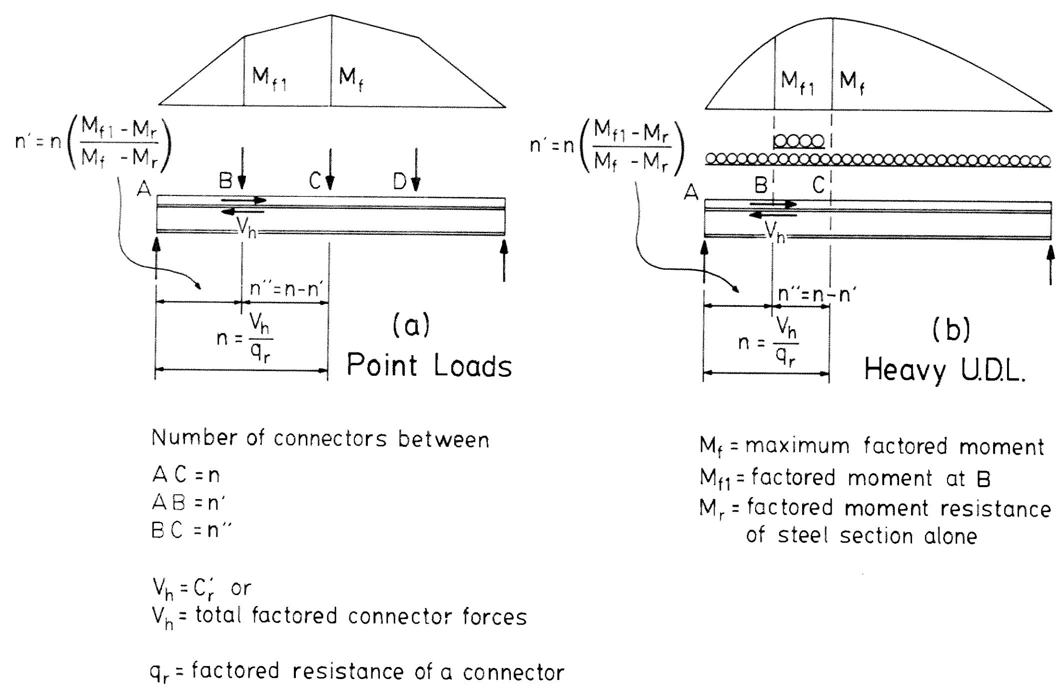


Figure 2.8
Distribution of Connectors as Prescribed by S16.1

Stud and Stud Welding Quality Control

S16.1 states that the welding of studs shall meet the requirements of CSA Standard W59, Welded Steel Construction (Metal Arc Welding)(2.29).

Under W59-Clause 5.5.6 "Stud Welding", the stud material requirements in relation to mechanical requirements of stud steel, and the qualification requirements of "stud base", must be met.

In addition, Clauses 5.5.6.5.1 and 5.5.6.5.3, under the topic "stud installation procedure control", must be followed. Briefly, this procedure control involves the testing by bending of 2 consecutive studs on a test piece, immediately followed by the testing through bending of two consecutive studs on the actual member for each welding production period as well as after any change in the welding procedure. In this case, studs are bent to an angle of 30 degrees off perpendicular.

Appendix H of W59 also states that at least one stud in every 100 shall be bent to an angle of 15 degrees off perpendicular and left in the bent position when no sign of weakness is evident.

Installation Recommendations for Field Welded Headed Studs

To ensure the uniform performance of field welded headed studs, a number of field conditions is

desirable. When studs are to be welded through single or double layers of steel deck, these conditions are more critical. A list of recommended conditions is shown below:

- Top flange of steel sections shall be free of heavy rust and mill scale and shall be left unpainted.
- The interface areas between steel sheets and the steel section shall be free of dirt, sand or other foreign materials.
- Water on the deck or between deck and the steel section must be removed prior to welding of each stud.
- Deck steel must rest tightly upon top flange of the steel section during welding.
- Ferrules and studs should be kept dry suitable for welding.
- After welding, ferrules shall be broken free from studs to permit visual inspection of welds and to ensure proper embedment of studs during slab pouring.
- When base steel nominal thickness greater than 1.52 mm for single thickness or 1.22 mm each in double thickness, or when the total thickness of galvanized coating of sheet(s) exceeds 380 g/m^2, procedures recommended by the stud manufacturer shall be followed(2.27).
- S16.1 states that studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA Standard W59, Welded Steel Construction (Metal-Arc Welding).
- For stud welding at low temperatures, Kennedy(2.28) found that welding procedure could be modified to produce acceptable stud weld quality at temperatures down to −40 degrees Celsius. In addition, stud manufacturer's welding setup and power requirements must be reviewed with proper adjustment for cold weather application. Stud testing should also be modified as required by W59.

REFERENCES

2.1 Viest, I.M., "Review of Research on Composite Steel – Concrete Beams", Journal of the Structural Division, June 1960.

2.2 Viest, I.M., "Investigation of Stud Shear Connectors for Composite Concrete and Steel T-Beams", Journal of the American Concrete Inst., Vol. 27, 1956.

2.3 Viest, I.M., "Test of Stud Shear Connectors, Parts I and IV", Engineering test data, Nelson Stud Welding, Lorain, Ohio.

2.4 Thurlimann, B., "Fatigue and Static Strength of Stud Shear Connectors", Journal of the American Concrete Institute, Vol. 30, June 1959.

2.5 Slutter, R.G., and Driscoll, Jr., G.C., "Flexural Strength of Steel-Concrete Composite Beams", Journal of the Structural Division, ASCE, April 1965.

2.6 Ollgaard, J.G., Slutter, R.D., and Fisher, J.W., "Shear Strength of Stud Connectors in Light-weight and Normal-weight Concrete", AISC Engineering Journal, April 1971.

2.7 "CISC Commentary on CAN3-S16.1", Handbook of Steel Construction", 1984.

2.8 "Code for the Design of Concrete Structures for Buildings", CAN3-A23.3-M77, Canadian Standards Association.

2.9 McMackin, P.J., Slutter, R.G., and Fisher, J.W., "Headed Steel Anchor Under Combined Loading", AISC Engineering Journal, Second Quarter 1973.

2.10 "Embedment Properties of Headed Studs", TWR Nelson Division, Design Data 10, 1977.

2.11 "Structural Engineering Aspects of Headed Anchors and Deformed Bar Anchors in the Concrete Construction Industry", KSM Welding Systems Division, 1974.

2.12 Robinson, H., "Tests on Composite Beams with Cellular Deck", Journal of the Structural Division, ASCE, August 1967.

2.13 Slutter, R.G., "Test of Cincinnati Centre Composite Beam Report No. 200.67.458.2, Fritz Laboratory, Lehigh University, January 1968.

2.14 Robinson, H., "Composite Beam Incorporating Cellular Steel Decking", Journal of the Structural Division, ASCE, March 1969.

2.15 Robinson, H., and Wallace, I.W., "Composite Beams with 1½ inch Metal Deck and Partial and Full Shear Connection", The Engineering Journal, Vol. 16, Sept. 1973.

2.16 Azmi, M.H., "Composite Open-Web Trusses with Metal Cellular Floor", Master Thesis, McMaster University, April 1972.

2.17 Fisher, J.W., "Design of Composite Beams with Formed Metal Deck", AISC Engineering Journal, July 1970.

2.18 Thelm, A.B., Weiler, T.J. and Wiker, R.L., "Push-Off Tests with ¾ inch, and ⅝ inch Studs, 3000 psi Solite Concrete", Inland-Ryerson Report 7065.5, November 1968.

2.19 Grant, J.A., Fisher, J.W., and Slutter, R.G., "Composite Beams with Formed Steel Deck", AISI Engineering Journal, First Quarter, 1977.

2.20 Iyengar, S.H., Zils, J.J., "Composite Floor System for Sears Tower", AISC Engineering Journal, Third Quarter, 1973.

2.21 Furlong, R.W., and Henderson, W.D., "Report of Load Tests on Composite Beams of Lightweight Concrete in Three-Inch Metal Deck with Stud Length as the Principal Variable", University of Texas, Austin, August 1975.

2.22 Allan, B., Yen, B.T., Slutter, R.G., and Fisher, J.W., "Comparative Tests on Composite Beams with Formed Metal Deck", Fritz Engineering Laboratory Report 200.76.458.1 Lehigh University, Bethlehem, December 1976.

2.23 Robinson, H., "Performance of shear studs in wide-rib profile deck-slab system through push-out and beam tests". Current research project, being tested at McMaster University, 1983-.

2.24 Grant, J.A., Fisher, J.W., and Slutter, R.G., "Tests of Composite Beams with Formed Metal Deck", Fritz Engineering Laboratory Report No. 381.2.

2.25 Goble, G.G., "Shear Strength of Thin Flange Composite Specimens", AISC Engineering Journal, April 1968.

2.26 Robinson, H., "Resistance to Longitudinal Cracking in Composite Girders", CSCC Research, McMaster University, Report to CSCC, October 1982.

2.27 "Nelson Weld-Thru Deck Application, Design", TWR Nelson Division, 1980.

2.28 Kennedy, D.J.L., "Stud Welding at Low Temperatures", Canadian Journal of Civil Engineering, Sept. 1980.

2.29 "Welded Steel Construction (Metal Arc Welding)" Canadian Standards Association W59 – 1982.

2.30 "Commentary on the AISC Specification", AISC Steel Construction Manual, Eighth Edition, 1980.

CHAPTER 3

3.0 LOADING CONSIDERATIONS FOR SHORED AND UNSHORED COMPOSITE FLOOR MEMBERS

3.1 INTRODUCTION

A compositely designed floor framing system acquires a significant portion of its final strength and stiffness from the concrete slab or deck-slab system. Thus, during member selection, consideration must be given to the fact that steel framing members may be susceptible to instability, excessive deflection or overstressing during construction. In this chapter we will explore the various loading stages experienced in composite construction and the implications for designer, fabricator, constructor, and owner/tenant.

3.2 DEAD LOADS AND LIVE LOADS

Gravity loads due to the structure and other building components that are "constant" throughout the life of the structure are referred to as dead loads. In composite floor design, structural dead loads consist of loads due to the mass of the concrete slab, steel deck (if applicable), and structural steel.

The slab load may include an allowance for concrete accumulation due to elastic deflection of the supporting members. If the members are unshored and uncambered, a level screeded floor can attract significant additional load due to concrete accumulation at mid span. Such addition of concrete to "level" the floor increases deflections which in turn requires more concrete - thus the term "ponding", normally used to describe the cumulative loading phenomenon created by rainwater on a roof, has been chosen to describe this condition. Steel deck deflections can contribute to this condition and in turn may attract self-loading of sufficient magnitude to be of concern. Table 3.1 illustrates the additional load applied to the structural frame due to deck deflections only. Cambering of steel framing, equivalent to steel elastic deflections under these theoretical concrete loads has been assumed. Single, double and triple span conditions are illustrated. Triple span deck, providing the least amount of erection joints, is the most common case for conventional buildings.

As an alternative to cambering, steel members may be shored at their theoretical elevation, thus restricting any concrete overage in quantities to that required to compensate for deck deflection. There are other implications to shoring of simple composite beams which will be discussed in Chapter 4. However, as a general rule, shoring of beams is undesirable for reasons which will be discussed in following paragraphs and in other chapters. Stub-girders must be shored during slab placement and this subject is discussed in Chapter 6.

Deck may also be shored. Shoring to reduce concrete quantities would rarely, if ever, be cost effective. Shoring to control temporary construction stresses on the steel deck might be considered on a special long span, but again, this approach would constitute an unusual application.

Dead loads due to other building components include loads produced by both moveable and fixed walls or partitions, floor finishes, fire-protective materials, mechanical-electrical distribution systems, lights and ceiling materials, etc. Because of the significant increase in strength and stiffness after composite action is achieved, it is usually desirable to keep the two groups of dead loads separate during the design of a composite floor member in order to facilitate the evaluation of structural effects under fresh-concrete condition loads or final occupancy-loading condition.

Under limit states strength design, all dead loads are multiplied by a load factor of 1.25, to take into account the variability of loads and load patterns and the analysis of their effects. Dead loads, having a counteracting effect or causing reversal of design forces, must be multiplied by a load factor of 0.85.

Gravity loads acting on a building frame due to occupancy, as well as snow on roof surfaces, are regarded as vertical live loads. Minimum specified live loads for floors of various occupancy types can be found in Part 4 of the National Building Code of Canada, 1985. Occupancy live loads (excluding snow) may be reduced for tributary area effects. Under the NBC 85, live load reduction due to tributary area effects is permitted for two categories of floor uses:

$$RF_1 = 0.5 + \sqrt{20/A} \leq 1.0 \qquad 3.1$$

where RF_1 = Live load reduction factor for floors or roofs used for storage, manufacturing, retail stores, parking garages or assembly halls, ($\geq$ 4.8 kPa)

A = Tributary area in square metres relating to the type of use and occupancy under consideration.

$$RF_2 = 0.3 + \sqrt{9.8/A} \leq 1.0 \qquad 3.2$$

TABLE 3.1 DEAD LOAD PER SPAN INCLUDING CONCRETE PONDING ON UNSHORED STEEL DECK SUPPORTED BY CAMBERED BEAMS AND GIRDERS+

DECK SPAN CONDITION	APPROXIMATE* DL PER SPAN INCLUDING PONDING (W)
Single Span	$\left[1.0 + \dfrac{0.4\ w_c\ s^4}{I_d}\right] s\ q$
Double Span	$\left[1.0 + \dfrac{0.15\ w_c\ s^4}{I_d}\right] s\ q$
Triple Span	$\left[1.0 + \dfrac{0.20\ w_c\ s^4}{I_d}\right] s\ q$

Where W = total slab load per span including ponding per metre width of deck (kN)
w_c = mass density of concrete (kg/m^3)
s = deck span (m)
I_d = moment of inertia per metre width of steel deck (mm^4)
q = Theoretical uniformly distributed slab load on unsagged steel deck (kPa)

Note: + If an unshored member is not cambered, its ponding effect should be investigated in conjunction with the ponding of concrete slab on steel deck.
* Computed by neglecting second order effect at span producing maximum deflected shape.

where RF_2 = Live load reduction factor for floors or roofs for use and occupancy other than specified above (and excluding snow).

Under NBC 85, floor members are to be designed for live loads applied uniformly over the entire tributary area or any portion of the area, whichever produces the most critical effects in the members concerned. (See design application, example problem Chapter 6.)

The gravity live loads acting on a structural floor member for a particular occupancy type can usually be divided into two distinct parts. First, a more sustained or long-term part, which may represent the loads due to the furniture, bookcases, desks, filing cabinets and safes with their contents in the case of an office occupancy. These loads would rarely exceed the dead load component of a steel framed structure. In the case of a retail store, warehouse, or a library, the specified live load frequently exceeds the dead load, and the amount of sustained live loads can represent a significant portion of the total specified design live load. The second part of live load consists of loads of short duration which might be caused by an extraordinary gathering of people, and stacking of building contents during renovation, etc.

In an office occupancy, the split of sustained versus short duration live loads (in an unshored composite member design) is of little significance as far as creep deflection is concerned. In a store or warehouse application, sustained live load may approach total design live load (which in turn may represent a significant portion of the total dead and live load); creep deflections in unshored composite floor members under these occupancies may be worthy of consideration.

When composite floor members are shored during construction, the total design long term loading includes sustained live loads as well as total dead loads. Hence, creep deflection can become a critical consideration during the design of shored composite floor members.

The effect of concentrated loads on a deck-slab system must also be evaluated. NBC 85 specifies appropriate concentrated loads, depending upon the type of occupancy. An area of floor or roof measuring 750 mm by 750 mm located so as to cause maximum design forces must support the specified concentrated load. This form of live load checking generally enables a small span floor member to support accidental overloading. It can be shown through existing load test data that, in the case of a one-way composite deck-slab, varying degrees of lateral redistribution of the concentrated load can occur depending upon the amount of transverse slab reinforcing[3.1]. In the case of a deck-slab with minimum slab reinforcing, it is conservative to assume a redistribution area equal to (750 + $2t_o$) by (750 + $2t_o$), see Figure 3.1.

Under limit states strength design, a live load factor of 1.5 is applied to the specified live loads to take into account the increased variability of the loads as compared to dead loads.

3.3 LOADING CONSIDERATIONS DURING CONSTRUCTION

In general, construction loading has been treated more as a construction safety feature rather than a code specified minimum load. Selection of a compositely designed steel beam is often governed by loading during construction. Thus, a careful review of both loading sequence and magnitude is warranted.

Loading of a compositely designed steel deck has similar ramifications. Shoring of deck is usually impractical, thus in a design situation where negative bending over the supports governs, a different profile may be tried, beam spacing may be reduced, deck material thickness may be incremented to the next standard thickness or, if the project size warrants, a special thickness supplying the appropriate section properties may be considered. Concentrated loads, particularly those resulting from careless handling of other construction materials, may also create difficulties and, in projects where significant usage of the bare deck as a working surface is anticipated prior to concrete placement, a deck material minimum thickness of 0.91 mm is sometimes selected as partial protection from this type of damage. In addition, local protection against abuse (in the form of temporary platforms and planking), such as at the entrance to a material hoist or elevator, or on the intended path of concrete buggy traffic, should be used.

Construction dead loads normally become part of the permanent dead loads in the structure and are therefore modified by an appropriate load factor of 1.25 in this case. Timing of application of these loads must be considered in the design.

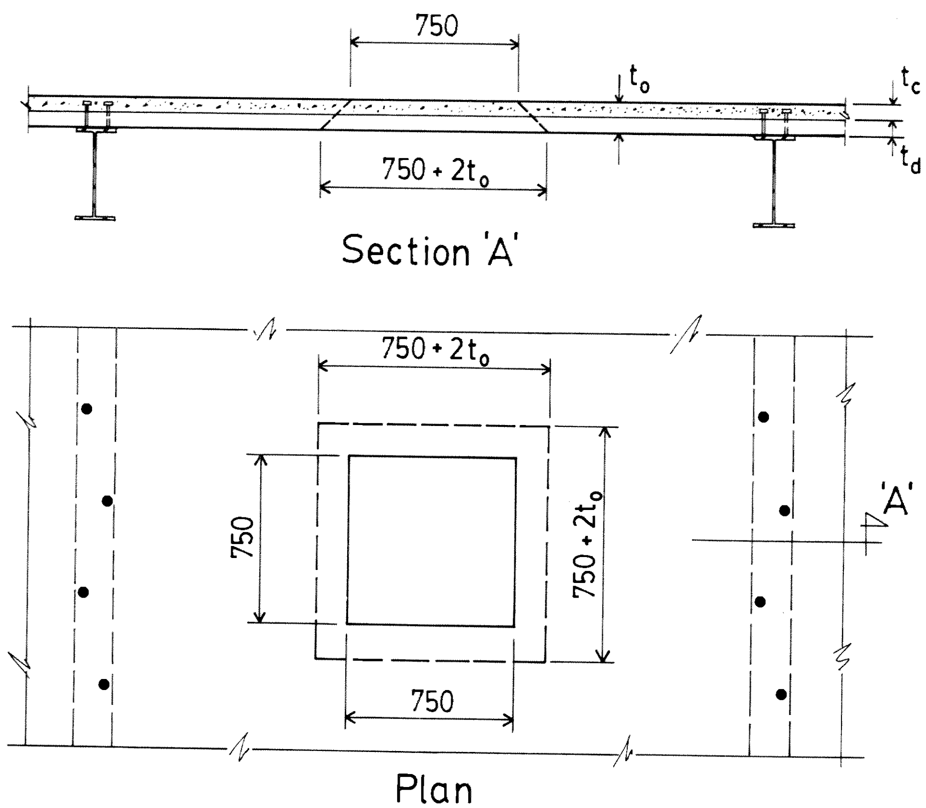


Figure 3.1
Assumed Distribution of
NBCC Specified Concentrated Load

Construction live loads are rather more difficult to define and are dependent upon the method of concrete placement. For example, the use of a concrete pump can restrict loading of the steel deck to a load very little higher than the fresh concrete except for the deck under and adjacent to the concrete pump distribution pipe. Concrete buggying can cause greater local loads and also may present a more serious risk of local damage due to accidental or careless dumping of buggies. The construction live loads that are assumed in this publication are shown in Table 3.2. A load factor of 1.5 is applied to these loads for design purposes.

3.4 LOAD COMBINATION CONSIDERATIONS FOR CONSTRUCTION AND OCCUPANCY

During the design of composite floor members, critical conditions under various load combinations and member support configurations are to be considered. In general, up to four distinct stages of loading are considered, i.e. deck placing, concrete placing, shoring (if applicable) and occupancy. Table 3.3 provides a quick reference for loads, load combinations and load factors to be considered during various stages of construction. In addition, concentrated live loads, for alternate design checks to account for accidental localized overloading, are also included for consideration.

TABLE 3.2 CONSTRUCTION LIVE LOAD FOR COMPOSITE FLOOR DESIGN+

Structural Element – Construction Event	Specified Minimum Construction Live Load (worst case of:)	Remarks
Steel deck – during concrete placing	a) 1 kPa b) 2 kN/m, 300 mm in width	– Uniform load – line load transverse to deck flutes
Steel member – during deck placing	a) 0.5 kPa b) 0.3 kPa c) Varies linearly from 0.5 to 0.3 kPa d) 4 kN	– for tributary area $\leq$ 27 m² – for tributary area $\geq$ 54 m² – when tributary area is > 27 and < 54 m² – concentrated load over an area 0.3 x 0.3 m, for beam area < 16 m²
Steel member – during concrete placing	a) 1.0 kPa b) 0.6 kPa c) varies linearly from 1.0 to 0.6 kPa d) 4 kN	– for tributary area $\leq$ 27 m² – for tributary area $\geq$ 54 m² – when tributary area is > 27 and < 54 m² – concentrated load over an area 0.3 $\times$ 0.3 m, for beam tributary area < 8 m²

+ Assumed for design calculations in Chapters 4, 5 and 6

TABLE 3.3 LOADS, LOAD FACTORS AND LOADING CONDITIONS FOR DESIGN OF COMPOSITE FLOOR MEMBERS

TYPE OF LOAD AND LOAD FACTORS	DECK PLACING	CONCRETE PLACING	MAXIMUM SHORED CONDITION*	OCCUPANCY CONDITION
Dead load $\alpha_D = 1.25$	– deck – steel member	– deck – concrete slab[+] – steel member (fresh-concrete condition) loading	– deck – concrete slab[+] – steel member – other building finishing components if applicable	– deck – concrete slab[+] – steel member – floor finish – partitions/walls – ceiling and mechanical/distributing systems – fire protection materials
Live load $\alpha_L = 1.5$	– construction materials	– due to heaping of concrete – due to construction equipment and material, etc.	– due to floors above (under construction) – due to temporary storage of materials – due to construction live loads	– specified LL (from NBC 85 or by designer), reduced based on tributary area where applicable – may include computed service equipment loading on floor
Alternate live load in the form of a concentrated load $\alpha_L = 1.5$	See Table 3.2	See Table 3.2		See NBC 85 Table 4.1.6.B

Notes: * maximum shored condition may occur when maximum number of levels of shored members are situated above the member under consideration.
[+] effects of slab load should include ponding of concrete.

REFERENCE

3.1 Porter, M.L., Ekberg, Jr., C.E., "Behavior of Steel-Deck-Reinforced Slabs", ASCE, Journal of the Structural Division, March 1977.

CHAPTER 4

4.0 COMPOSITE BEAMS AND GIRDERS

4.1 INTRODUCTION

One of the major developments in steel design during the last two decades has been a significant trend to compositely designed rolled or welded beams with a deck-slab system. Structural steel design standards governing design of composite members have evolved using primarily an ultimate strength approach, leading up to the limit states concept used in S16.1, the Canadian Standard.

The study of composite structural interaction between steel beams and concrete dates back to as early as 1922 by Mackay et al[4.1] in Canada, later followed by researchers in U.S.A., England, Switzerland, France, Belgium and Germany[4.2]. During the mid 30's, a series of tests on composite beams was conducted at the University of Toronto under the auspices of the Canadian Institute of Steel Construction[4.3]. These tests gave an indication of the combined strength of concrete slabs bearing on top of steel beams, with and without shear connectors, as well as some with partial encasement of the top flange and a portion of the web. As a result, design tables were printed in the 1937 edition of the CISC Steel Handbook.

In 1941, clauses governing composite beam design for buildings were incorporated in the National Building Code of Canada, as a direct result of the earlier research and testing. It was not until 1944, that the first North American specification for composite bridge girder design was issued[4.4,4.5]. In the U.S.A., development of composite building design followed somewhat later. Composite beam design provisions were first introduced in the 1952 edition of the AISC Specification.

Structural designers have long been aware of the advantages of composite beam construction such as the saving of steel, reduction of overall structural depth, and the increase in floor stiffness and load capacity. Prior to the early 1960's, steel beams of rolled and welded 'H' shapes were designed to act compositely with poured-in-place flat-bottom slabs of various thicknesses with connection provided by means of either member embedment or mechanical shear connectors, herein after referred to as "Solid Composite Construction". See Figure 4.1. However, the continuing search for improved material and manpower utilization as well as construction economics has resulted in the evolution of a method referred to herein as "Hollow Composite Construction". See Figure 4.2. Beginning around the mid-sixties, structural designers began to use composite beams incorporating composite deck-slab systems[4.6,4.7,4.8,4.9]. The use of steel decks in building construction supplanted the traditional temporary timber planking previously required for building access and for the safety of workers both above and below, offering as well immediate access to other building trades. The economy of this method of floor construction was enhanced during the same period, with the introduction of headed stud shear connectors welded through the fluted or cellular deck units into the top flange of the steel beams. Interim steps on some early projects called for pre-punching of the steel deck, either for direct field welding of studs to the steel beams, through the openings, or for shop installation of the studs to a grid, matching the pre-punched deck holes. Neither practice proved to be very satisfactory and the weld-through practice evolved.

The following sections of this chapter are dedicated to a more detailed review of design methodology for composite beam members as permitted by S16.1, including the latest amendments to the standard.

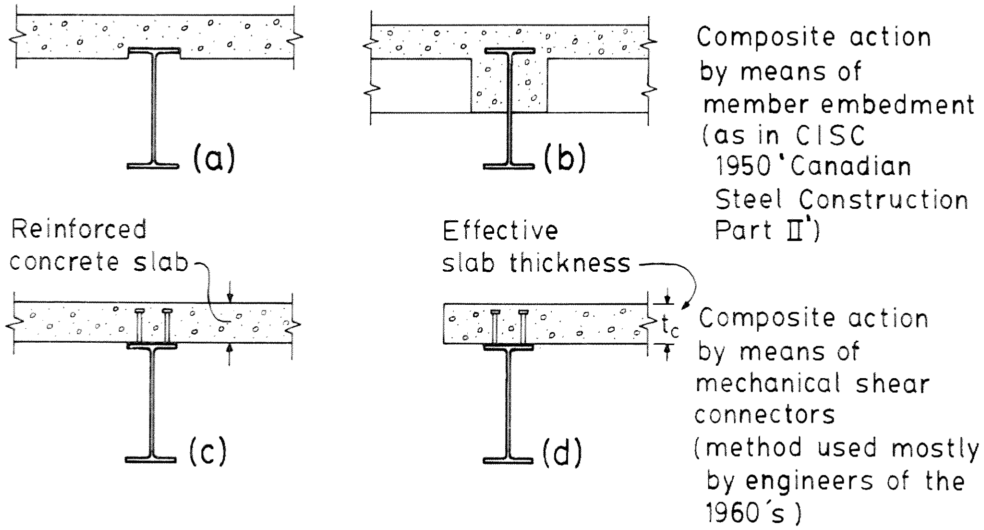


Figure 4.1
Solid Composite Construction

Composite Beam and Girder Floor Systems

In a composite beam or girder, the steel shape is used primarily to resist tension and shear while the concrete slab acts primarily as a compression resisting element. Thus, the composite beam or girder is usually designed as a simply supported flexural member. Composite action can be achieved in the beams alone, in the girders alone, or in both. The steel members are usually W-shapes, although welded W-shapes and HSS shapes may also be used. Trusses are specifically dealt with in Chapter 5 and stub-girders are discussed in considerable detail in Chapter 6.

4.2 EFFECTIVE THICKNESS OF CONCRETE SLAB

The concrete slab may be cast on temporary forms, sheet steel forms or steel deck. If the concrete slab is cast with a flat underside, or on a corrugated or fluted sheet steel form, the strength and stiffness of the composite member can be calculated based on the overall slab thickness in accordance with Clause 17.2 of Standard S16.1 (also see definitions in Sections 1.2 and 1.4) and a floor so constructed is known as ***Solid composite*** construction. However, if concrete is cast on steel deck (See Figure 1.12d) only the concrete cover thickness above the top of the steel deck is effective for composite action with the steel shape. This type of configuration is referred to here as ***Hollow composite*** construction.

Hollow composite floors have become a very commonly used gravity load resisting system for four main reasons.

- The use of steel deck eliminates the need for formwork shoring and provides a wide effective width of concrete slab for composite interaction with the steel shape.
- Composite steel decks also serve as positive concrete reinforcement.
- Welding of headed studs directly through the steel deck provides economical interconnection of beam and deck-slab.
- Steel decks used in a cellular configuration allow the passage of in-floor electrical and communication services.

Although most of the design tables and the worked example in this chapter apply specifically to hollow composite floor construction, the design considerations and methodology outlined also apply for the most part to solid composite floor construction.

4.3 EFFECTIVE WIDTH OF CONCRETE TOP FLANGE

For a T-beam formed by a steel section and a concrete cover slab, only part of the concrete top flange is effective. The effective width, under elastic conditions, is a function of beam span, Poisson's ratio, and the shape of the moment diagram[4.10,4.11]. Based on elastic theories, semi-empirical design rules for determining effective slab width have been adopted by S16.1. These rules may be applied to hollow composite members as well as solid composite members because strain measurements across the slab width have indicated that shear lag is no more severe in a steel deck-slab than in a formed solid slab[4.12]. The applicability of the existing design rules for computing effective slab widths of composite beams designed by ultimate strength method is explained by the Commentary to S16.1 as published in the CISC Handbook of Steel Construction, 3rd Edition. The design effective width of concrete specified by Clause 17.3.2 of S16.1 is described in Section 1.4 and illustrated in Figure 1.13.

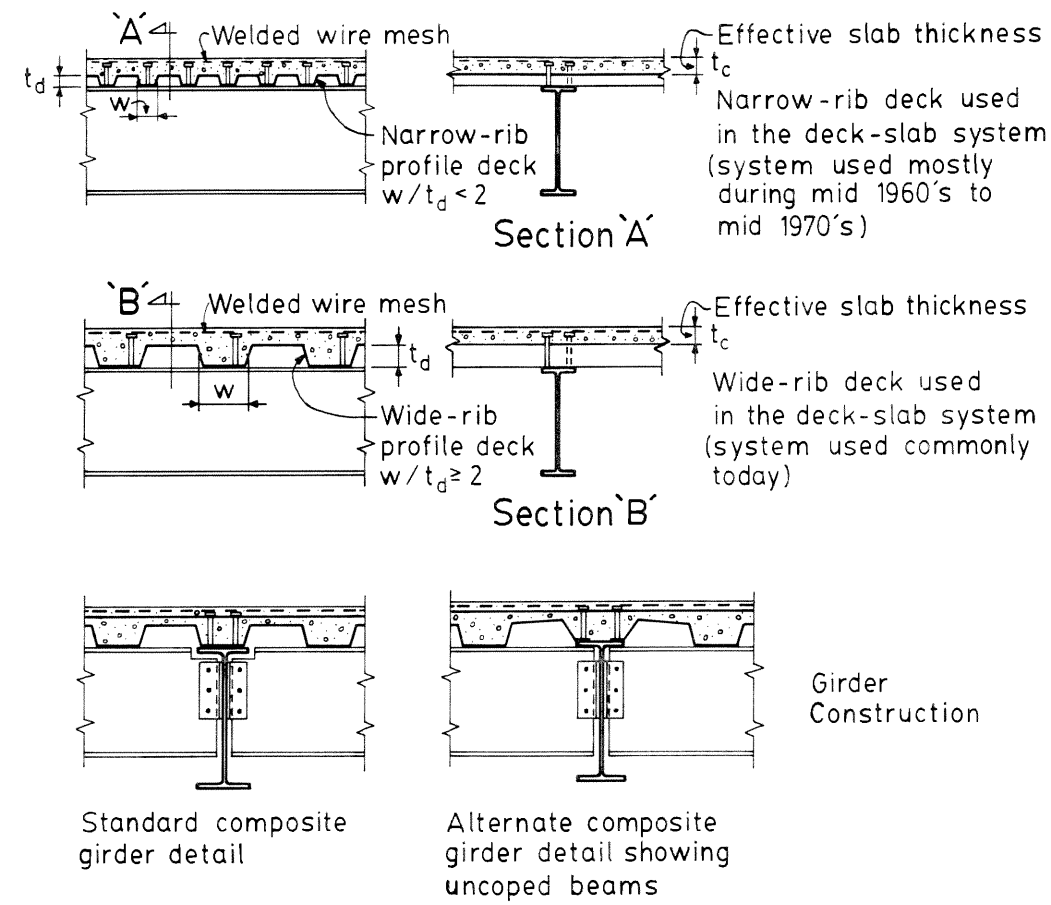


Figure 4.2
Hollow Composite Construction

The design of a composite floor member involves the assessment of its performance at various limit states including evaluation of:

- the ultimate strength of the composite section,
- the ultimate capacity of the bare steel section during construction stages, and
- the performance of the composite member both as an individual member and as part of the overall floor system, when subjected both to specified loads and during construction.

4.4 FLEXURAL STRENGTH OF A COMPOSITE SECTION

The ultimate flexural capacity of a composite member depends on the degree of shear connection provided, the compressive resistance of the effective concrete slab (plus any steel in compression, in those instances where the neutral axis falls within the steel section), and the tensile yield resistance of the steel shape. Shearing of the concrete adjacent to the steel/concrete connection will be discussed in Section 4.9. Three flexural modes of failure therefore exist:

- shear connection failure,
- crushing of concrete, and
- full yielding of steel section.

Composite members with full shear connection can exhibit full composite action. Composite members with partial shear connection lack the strength to provide full composite action and will likely fail at the shear connections. When failure occurs either by crushing of concrete and/or full yielding of steel, large deflections prior to failure are the norm.

Limit states design of a composite beam requires a designer to satisfy strength and stability criteria such that the factored member resistance under the ultimate limit state condition is greater than, or equal to, the effect of factored external loads. In general, factored load means the product of a specified load and its load factor and factored resistance means the product of member resistance and the applicable performance factor[4.13,4.14]. By neglecting concrete tensile strength, the factored ultimate limit state moment resistance, M_{rc}, can be calculated by the following procedure.

Locate Plastic Neutral Axis

The plastic neutral axis of a composite section can be located by comparing the factored compressive resistance of the effective concrete slab, $0.85\ \phi_c\ b_l\ t_c\ f'_c$, and the factored tensile resistance of the steel shape, $\phi\ A_s\ F_y$, where

ϕ_c = performance factor for concrete, 0.60
ϕ = performance factor for steel, 0.90
b_l = effective width of concrete slab, mm
t_c = effective thickness of concrete slab, mm
f'_c = specified compressive strength of concrete at 28 days, MPa
A_s = cross-sectional area of steel shape, mm^2
F_y = specified minimum yield strength of steel, MPa

Case 1 – Neutral Axis in Concrete (full shear connection)

If $\phi\ A_s\ F_y$ is less than $0.85\ \phi_c\ b_l\ t_c\ f'_c$, the plastic neutral axis (P.N.A.) falls within the concrete as shown in Figure 4.3. The ultimate flexural capacity is reached when the steel shape is fully yielded.

The depth of the rectangular concrete stress block, 'a', can be found by equating the factored ultimate compressive component to the tensile component of the moment couple of the composite section,

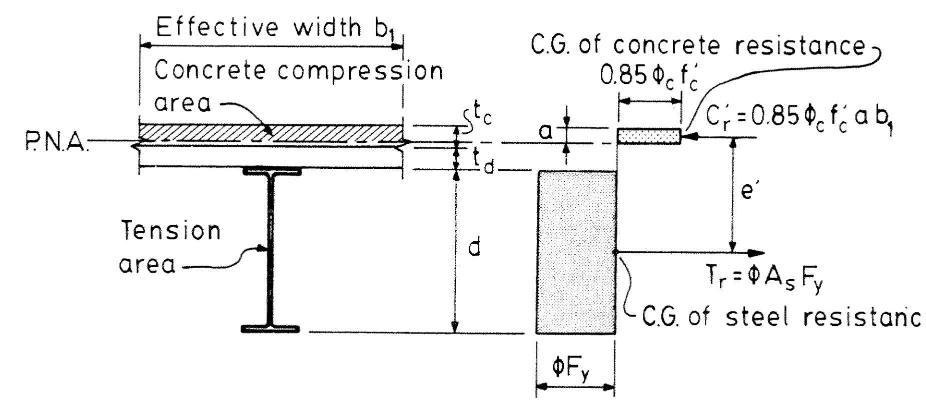


Figure 4.3
Neutral Axis Falls within Effective Slab Thickness ($a \leq t_c$) (Case 1)

$$0.85\ \phi_c\ f'_c\ b_l\ a = \phi\ A_s\ F_y$$

and solving for 'a',

$$a = \frac{\phi\ A_s\ F_y}{0.85\ \phi_c\ f'_c\ b_l} \qquad 4.1$$

Note: Value of 'a' should always be less than or equal to t_c.

M_{rc} can then be computed by finding the internal moment arm, e', as

$$M_{rc} = e'\ \phi\ A_s\ F_y \qquad 4.2$$

in which

$$e' = \frac{d}{2} + t_o - \frac{a}{2} \qquad 4.3$$

where d = depth of steel section in millimetres
t_o = overall depth of deck-slab in millimetres

In the case of solid composite construction, $t_c = t_o$

In order to achieve composite action concrete and steel must act as a single unit. If headed studs are used as the shear transfer device, the principal force that must be resisted by these stud connectors is the sum of the factored horizontal shears between the points of maximum and zero moment, V_h, as illustrated in Figure 4.4.

If full shear connection is required the designer must ensure that

$$Q_r \geq \phi\ A_s\ F_y \qquad 4.4$$

Where Q_r = sum of the factored resistances of all shear connectors between the point of maximum moment and the adjacent point of zero moment (also see Section 2.5, stud shear connectors)

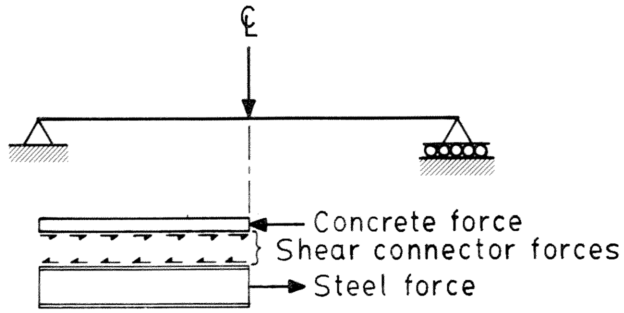


Figure 4.4
Concrete-Steel Interface Shear Forces

Case 2 – Neutral Axis in Steel (full shear connection)

Case 2 refers to the situation where the plastic neutral axis lies in the steel section when full shear connection is provided. This occurs when the resistance of the composite section is governed by concrete compressive capacity, i.e.,

$$0.85\ \phi_c\ f_c'\ b_1\ t_c < \phi\ A_s\ F_y$$

The factored concrete compressive resistance is computed as,

$$C_r' = 0.85\ \phi_c\ f_c'\ b_1\ t_c \qquad 4.5$$

As shown in Figure 4.5, part of the steel section is now in compression.

The factored compressive resistance of the steel area in compression can be written as,

$$C_r = \frac{\phi\ A_s\ F_y - C_r'}{2} \qquad 4.6$$

By taking moments about the centroid of the steel area in tension, the factored moment resistance can be found:

$$M_{rc} = C_r'\ e' + C_r\ e \qquad 4.7$$

where e′ and e are the lever arms as shown in Figure 4.5. The lever arms can be computed once the exact location of the P.N.A. is found.

a) P.N.A. in steel flange i.e. when $C_r \leq \phi\ b\ t\ F_y$

$$e = \frac{(A_s d - b t_1^2)}{2(A_s - b t_1)} - \frac{t_1}{2} \qquad 4.7a$$

$$e' = e + \frac{t_1}{2} + t_o - \frac{t_c}{2} \qquad 4.7b$$

where $t_1 = \dfrac{C_r}{\phi\ b\ F_y}$

b) P.N.A. in steel web, i.e. when $C_r > \phi\ b\ t\ F_y$

$$e = d - d_2 - d_3 \qquad 4.7c$$

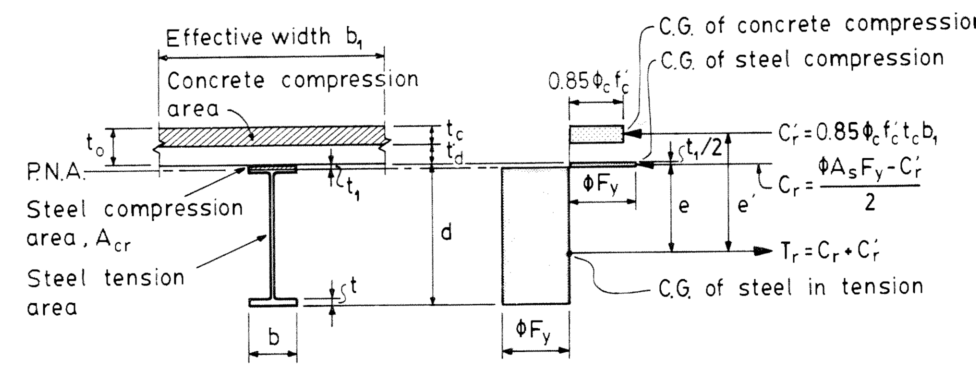
$$e' = d + t_o - d_2 - \frac{t_c}{2} \qquad 4.7d$$

where $d_2 = \dfrac{A_s d/2 - A_{cr}(d - d_3)}{A_s - A_{cr}}$

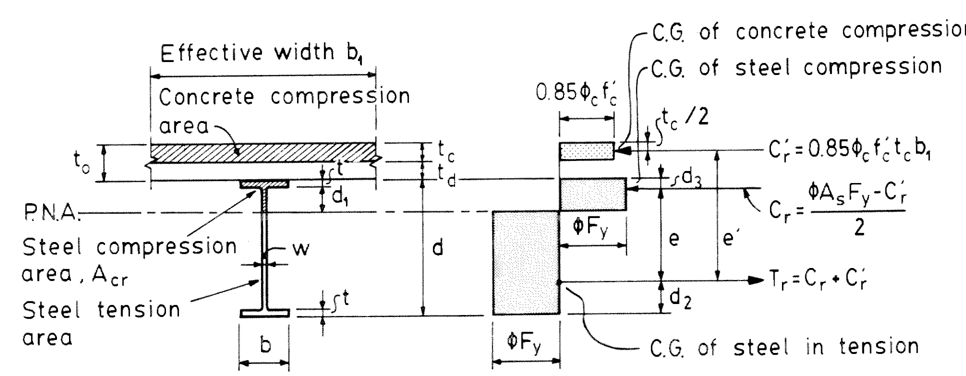
$$d_3 = \frac{bt^2/2 + d_1 w(t + d_1/2)}{A_{cr}}$$

$$d_1 = (A_{cr} - bt)/w$$

$$A_{cr} = \frac{C_r}{\phi\ F_y}$$



(a) Plastic Neutral Axis in Steel Flange



(b) Plastic Neutral Axis in Steel Web

Figure 4.5
Force Equilibrium of Composite Section with Full Shear Connection (Case 2)

For Case 2, V_h is equal to C_r'. Since full shear connection is required, the following requirement must be satisfied.

$$Q_r \geq 0.85\ \phi_c\ f_c'\ b_1\ t_c \qquad 4.8$$

Case 3 – Partial Shear Connection

In many design situations, full shear connection is not necessary. In fact, most composite floor beams are constructed with partial shear connection in the range of 50% to 70% of that theoretically required for full composite action (i.e. Q_r required for Case 1 or Case 2).

There are three main reasons for this:

- 50% or higher shear connection usually provides an ultimate flexural capacity greater than 80% of the full composite flexural capacity(4.15).
- Construction considerations such as deflection under fresh concrete and beam depths available, particularly in the case of unshored construction, may govern the selection of the steel beam section.

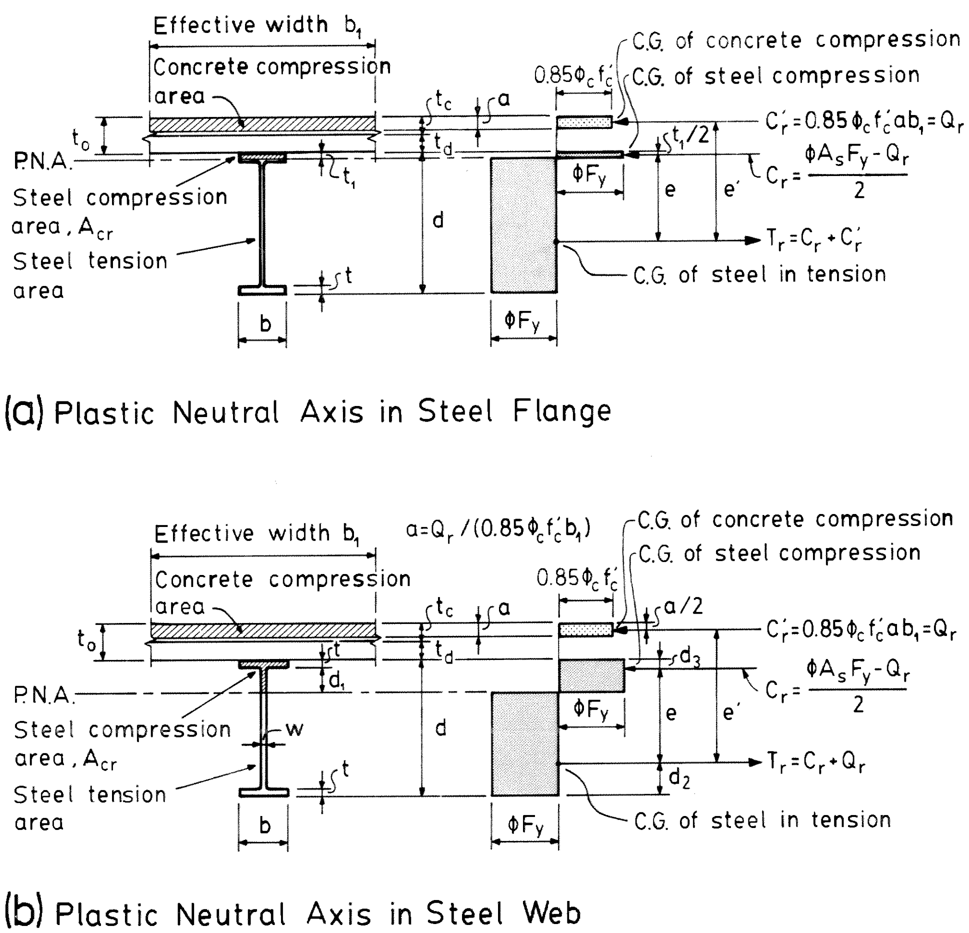


Figure 4.6
Force Equilibrium of Composite Section with Partial Shear Connection (Case 3)

- Physical shape and layout of deck flutes on a composite beam often prevent the economical distribution of sufficient stud connectors for the attainment of full shear connection without installation of studs in pairs and thus reducing their efficiency.

Partial shear connection is calculated as the ratio of Q_r to the lesser of $\phi\ A_s\ F_y$ and $0.85\ \phi_c\ f_c'\ b_1\ t_c$ expressed as a percentage. For flexural resistance calculations, a partial shear connection lower limit of 50% is specified in Clause 17.4.4 of S16.1. Composite beams with less than 50% shear connection may not behave as a composite member through the entire loading range up to the ultimate state as predicted by the method described herein(4.15). If composite action is required for deflection considerations only, the lower limit is reduced to 25%, since composite behaviour up to specified load level only is of concern. Considering the model as shown in Figure 4.6, the plastic neutral axis of a Case 3 member always falls within the steel shape. C_r' is restricted to the total horizontal shear provided by the shear connectors, thus

$$C_r' = Q_r = V_h \qquad 4.9$$

C_r' is assumed to act at the centroid of the assumed concrete stress block whose depth, 'a', can be determined by writing

$$0.85\ \phi_c\ f_c'\ b_1\ a = Q_r$$

and solving for 'a',

$$a = \frac{Q_r}{0.85\ \phi_c\ f_c'\ b_1} \qquad 4.10$$

C_r can then be computed,

$$C_r = \frac{\phi\ A_s\ F_y - Q_r}{2} \qquad 4.11$$

followed by locating the plastic neutral axis (in steel).

M_{rc} for partial shear connection is calculated using

$$M_{rc} = C_r e + Q_r e' \qquad 4.12$$

Equations 4.7a to 4.7d are applicable, ***provided*** t_c is replaced by 'a'.

Detailed discussions on stud shear connector strengths and spacings for use with solid or hollow composite floor members have been covered in Chapter 2.

4.5 SHEAR STRENGTH

It is assumed that the web of the steel shape carries all the vertical shear force of the composite section. The factored shear resistance can be computed as:

$$V_r = \phi\ A_w\ F_s \qquad 4.13$$
where $A_w = d\ w$

All Canadian rolled W-shapes with F_y = 300 MPa (except W410×39) have web slenderness conforming to Cl. 13.4.1a of S16.1, hence $F_s = 0.66\ F_y$. For all other shapes and for sections rolled in higher strength steel, refer to Clause 13.4.1 of S16.1 for the computation of factored shear resistance.

4.6 LATERAL SUPPORT OF UNSHORED MEMBERS

Since composite action cannot be realized before concrete hardens, only the bare steel flexural capacity is available for load support during construction. The load carrying capacity of a steel shape therefore is assessed based on the sequence of construction as described in Chapter 3.

In the case of solid composite floor construction, unshored construction is possible if the forms carrying the fresh-concrete are supported by the steel beams and girders. The flexural strength of a simply supported steel member depends on the lateral support provided by the formwork and/or the framing members. Concrete forms may be designed to provide this lateral support to the steel beam. In accordance with Clause 19.3.1 of S16.1, each of the bracing members that are spaced at intervals should be designed to resist 1% of the compressive force in the top flange at the point of support. In the event that steel deck or form is used to provide a continuous lateral support, it should be designed and connected to the top flange in such a way that it can resist a uniformly distributed lateral force for the length of the compression flange equal to 5% of the maximum axial force in the flange. See Clause 19.3.2 of S16.1.

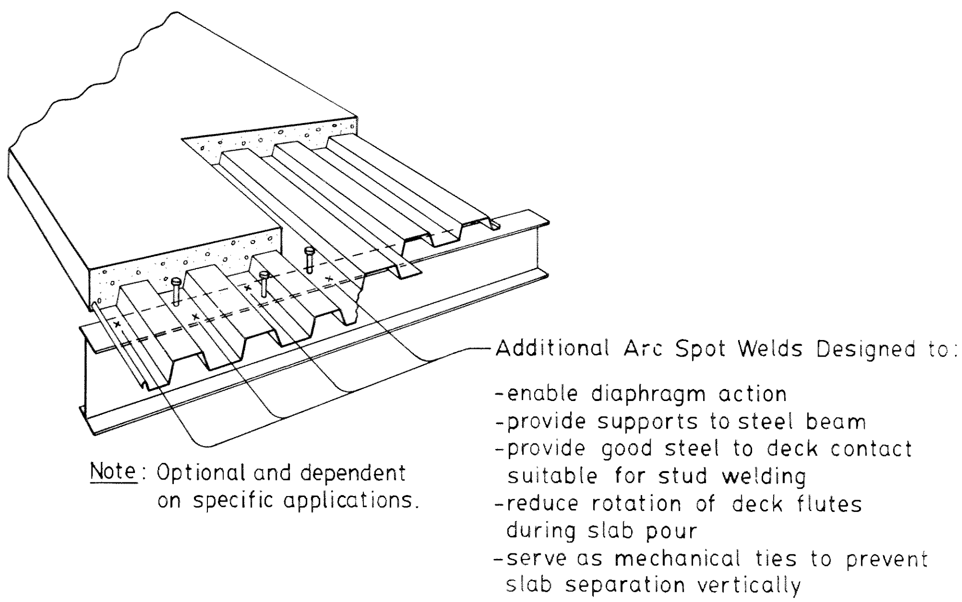


Figure 4.7
Functions of Arc Spot Welds (Puddle Welds) in Hollow Composite Floors

4.7 DESIGN OF UNSHORED HOLLOW COMPOSITE MEMBERS

Temporary shoring and formwork can be eliminated in the construction of hollow composite floor systems if the steel deck and all the bare steel members are adequately designed to carry the fresh-concrete condition loads (inclusive of construction live loads) throughout various construction stages.

In the case of a hollow composite beam, the steel deck is usually connected to the steel beam by means of arc spot welds[4.16] (or puddle welds). These welds, 1) enable the deck and beams to act as a diaphragm as described in Chapter 2, 2) permit the steel deck to act as continuous lateral and torsional support to the steel beams, 3) assist in preventing vertical separation between the deck-slab system and the steel shape, and 4) serve other construction purposes. See Figure 4.7. Stud shear connectors will perform a duplicate function and are frequently used in lieu of some of the deck-to-beam arc spot welds.

Before the steel deck is welded to the steel beam's top flange, the unbraced length, L′ is equal to the beam span. The steel beam is required to carry the mass of the steel deck itself plus a nominal amount of construction load while the steel deck is being placed. After the steel deck is connected to the top flange, the steel beam may be considered continuously braced and hence it can be designed as a laterally supported beam to carry the fresh-concrete condition loads (inclusive of construction loads during concrete placement); see Fig. 4.8, and 4.9b.

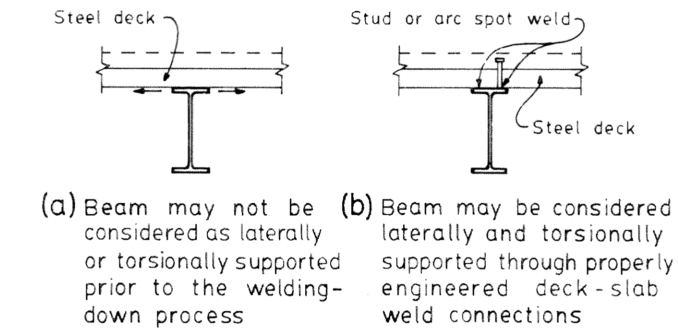


Figure 4.8
Lateral Support Conditions of Hollow Composite Beams under Construction

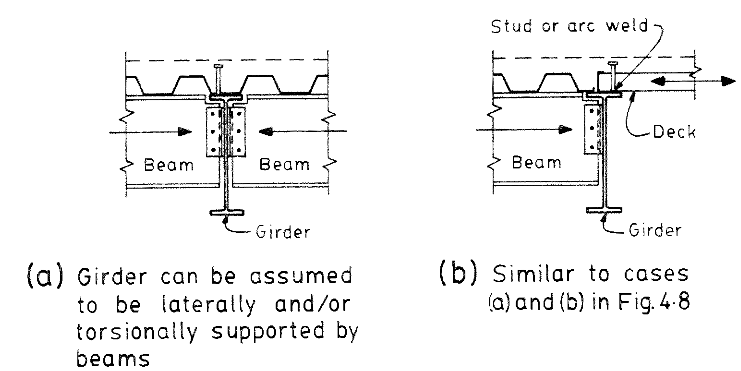


Figure 4.9
Lateral Support Conditions of Hollow Composite Girders under Construction

For a hollow composite girder, the steel deck is usually installed with flutes parallel to the girder span and hence continuous lateral and torsional supports cannot be assumed. In this situation, lateral support of a girder is provided at intervals where beams are framed into it (Figure 4.9a). The unbraced length, L′ to be used for design is the spacing of two adjacent beams near the mid span as shown in the example of Figure 4.10. Undoubtedly, there will be applications where the stiffness of deck and the deck-to-girder connection could be considered to provide additional lateral support to the girder top flange, if required.

Laterally Unsupported Members

When continuous lateral support is not provided to a steel W-shape which is subjected to in-plane bending, lateral-torsional instability may govern the flexural capacity of the member. The factored moment resistance of a laterally unsupported W-shape, M_r', can be determined in accordance with Clause 13.6 of S16.1 as follows (also see Table 4.1).

$$M_r' = \phi\, M_u \quad \text{if} \quad M_u \le \frac{2}{3} M_p \qquad 4.14$$

$$M_r' = 1.15\, \phi\, M_p \left(1 - \frac{0.28 M_p}{M_u}\right) \text{ but not greater than } \phi\, M_p \qquad 4.15$$

$$\text{if } M_u > \frac{2}{3} M_p$$

$$\text{where } M_u = \frac{\pi}{\omega\, L'} \sqrt{E\, I_y\, G\, J + \left(\frac{\pi E}{L'}\right)^2 I_y\, C_w} \qquad 4.16$$

$M_p = Z_x\, F_y$ for Class 1 and Class 2 sections
$\omega = 0.6 + 0.4\, M_1/M_2$ (see Figure 4.10)

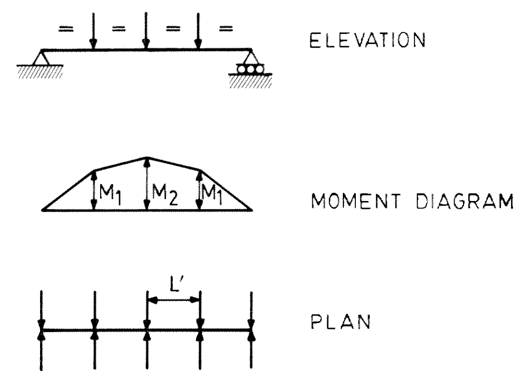


Figure 4.10
Unbraced Length of a Composite Girder
Prior to Composite Action

The equivalent bending coefficient, ω, is used to modify the unbraced length, L', depending on the shape of the moment diagram within the unbraced length. ω assumes a value of 1.0 where the moment diagram within the unsupported length is uniform or nonlinear as shown in Figure 4.11.

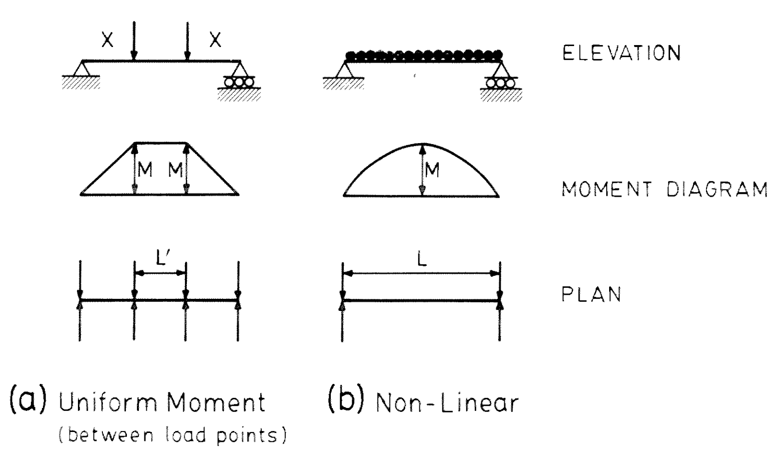


Figure 4.11
Shapes of Moment Diagrams
when Equivalent Bending Coefficient = 1.0

In order to calculate M_u, material and sectional properties including elastic and shear moduli of steel (E,G), moment of inertia about the y-axis, I_y, and the torsional constants of the W-shape, J and C_w, are required. However, M_r' values for various unbraced lengths are listed in Tables 4.1 to 4.8. The L_u values listed in the tables represent the maximum effective unbraced lengths at which lateral-torsional instability is not a problem. Since most L_u values are lower than beam spans of practical design range, one cannot afford to ignore stability investigation of the bare steel member before the steel deck is welded to its top flange in unbraced and unshored composite construction.

4.8 SERVICEABILITY REQUIREMENTS

While an acceptable level of safety against collapse is ensured by procedures discussed elsewhere, the performance of a structural system in various service conditions should also be considered. Serviceability limit states design of a composite beam and girder frame include 1) calculation of vertical deflection under short term and long term specified loads, 2) evaluation of construction deflections, and consideration of possible camber requirements in the case of unshored construction, 3) computation of beam deflection caused by slab shrinkage, and 4) assessment of vibration characteristics of the floor system (see Chapter 7). In addition, some measures should be taken to control concrete cracking as described in Section 1.4d. If unshored construction is selected, yielding of the bottom flange must also be prevented (see S16.1 Clause 17.6).

Although it is not a requirement of S16.1, maximum span-to-depth ratios of 24 and 30, for composite section depth and steel member depth respectively, were recommended in a CSSBI publication[4.9]. It is believed that these criteria will provide a good starting point in the selection of satisfactory composite members. Consideration of the above serviceability criteria, consideration of the standard depths of sections available, and any special design criteria such as high code-specified design loads in areas of anticipated low actual loads (e.g. shopping mall rest areas) will prompt further consideration of these criteria and further vibration analysis as noted in Chapter 7.

Vertical Deflection

Floor structural members deflect or sag under load. This vertical deflection, if excessive, can cause cracking of ceilings and partitions or prevent proper fit of doors. Visible distortions may cause undesirable psychological feelings to some people. If dead load deflection (especially member deflection due to fresh-concrete condition loads) is found to be excessive, it can usually be controlled by shop cambering the steel members or by shoring.

In order to prevent excessive deflection due to specified live loads (and a portion of superimposed dead load, e.g. partitions), the composite beam should be proportioned with adequate flexural stiffness. For simple span members of floors and roofs supporting construction and finishes not susceptible to cracking, a deflection limit of 1/300 of span is suggested as acceptable in Appendix I of S16.1. The appropriate deflection limit depends primarily on the type of ceiling and partitions supported by the floor. A limit of 1/360 of the span under full specified live loading is generally considered acceptable for simple span floor members supporting finishes susceptible to cracking. This maximum limit is also suggested in Appendix I of S16.1.

Since a floor member is usually subjected to occupancy loads after the concrete has attained its specified strength, member deflection should be calculated based on the elastic sectional properties of the composite section. The elastic stiffness of a composite section can be computed by means of the transformed section method. It is customary to calculate the moment of inertia of a composite section, I_t, by regarding the effective concrete cross-section (in compression) as equivalent steel cross-section, using the elastic modular ratio E/E_c. The estimate of live load deflection is complicated by the fact that concrete creeps under sustained loads. These long-term loads include dead loads (that are carried by the composite section) and a portion of the service live load. If the member is shored during construction the composite section must sustain all the fresh-concrete condition loads, superimposed dead loads, and a portion of the live load for a long period in service. In the case of unshored construction, the fresh-concrete condition loads are carried by the steel shape and hence only the sustained superimposed dead and live loads are responsible for concrete creep deflection. The amount of live load that contributes to creep depends on the type of floor occupancy. The sustained live load of a typical floor in an office building is a small fraction of its total occupancy load (about 25%)[4.37], but in a warehouse, a large portion of the service load may stay in place throughout its useful life.

For a compositely designed beam or girder, creep deflections occur when the effectiveness of the concrete slab under long term loading decreases, leading to an increase in steel stresses. For this reason, stresses and deflections caused by dead load on the composite section are usually determined with composite section properties calculated using a factored steel to concrete modular ratio equal to approximately 2.5 times the elastic modular ratio[4.17,4.18].

Steel-concrete interface slip and partial shear connection may reduce the stiffness of a composite member[4.12,4.15]. In the case of a hollow composite beam (ribs of slab running perpendicular to the beam) the ribbed profile of the deck-slab may also increase flexibility of the composite member. In lieu of tests or analysis, S16.1 provides a rule for the computation of effective moment of inertia, I_e, such that the reduction of composite member stiffness due to these effects can be accounted for during the member deflection computation.

$$I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s) \qquad 4.17$$

where I_s = moment of inertia of steel beam
I_t = transformed moment of inertia of composite beam
p = fraction of full shear connection
(use $p = 1$ for full shear connection)

If the effect due to concrete creep is NOT accounted for in a deflection computation or via a test, the estimated deflection under specified loads (using effective composite moment of inertia I_e, calculated on the basis of the unfactored elastic modular ratio E/E_c) must be increased by a minimum of 15%, in accordance with Clause 17.3.1.1(b) of S16.1.

Composite Beam Deflection due to Shrinkage

Tests[4.19] of small plain concrete specimens, exposed to air at 50 percent relative humidity, produced unit length changes due to drying shrinkage strains in the range of 600 to 800 micro strain. Since the plain concrete specimens were free to shrink without any restraint, the measured shrinkage values are referred to as free shrinkage of concrete. It was found that an average of one third of the estimated free shrinkage occurred within the first month and that 90% of the estimated free shrinkage had taken place at the end of an 11 month period. (Shrinkage measurements in these tests were terminated at the end of 38 months.)

The rate and total amount of drying shrinkage in a concrete specimen depend on factors such as:

- curing practices,
- surface/volume ratio,
- average relative humidity of surrounding air,
- average surrounding air temperature,
- water-cement ratio of the concrete mix (slump),
- size and type of aggregate,
- time from initial set of concrete, and
- amount of restraint from steel reinforcement.

See references [4.20,4.21,4.22].

Volumetric reduction of concrete due to drying shrinkage may also contribute to flexural deflection of composite members. Since the concrete slab may shorten but the steel shape does not, therefore, shrinkage due to drying of the concrete slab is partially restrained by the existence of steel reinforcement and by attachment to the steel shape. The slab shrinkage measured in such a test specimen is referred to as restrained shrinkage.

A drying shrinkage deflection pilot test was carried out by Robinson[4.23]. The test specimens consisted of two simply supported W410×54 hollow composite beams, 9 metre span, using 76 mm deep wide-rib profile composite decks (from two deck manufacturers) with ribs running perpendicular to the beam span. A 65 mm thick cover slab of normal density concrete with 152×152 MW9.1×MW9.1 welded wire mesh, forming a flange width equal to 2290 mm, was used on each test beam. The slump of the concrete mix was measured at 75 mm. Instrument readings of beam deflections due to slab shrinkage, together with room temperature and relative humidity of the laboratory atmosphere, were taken for a period of 50 days. Shrinkage readings along the length of concrete top flange, as well as those of a small unreinforced calibration specimen, were taken for the two test beams. Unlike the free shrinkage measurements previously discussed, a measurable increase in beam deflections due to shrinkage of the concrete slab was noted during the first 40 days of recording. After this, no discernible increase of beam deflection was observed for the remainder of the test period (see Figure 4.12). For the beams tested, the measured slab shrinkage amounts to about 2/3 of the free shrinkage measurement of the calibration specimen. Using these results to calculate slab shrinkage of the test specimen gave a computed slab shrinkage strain of about 350 micro strain.

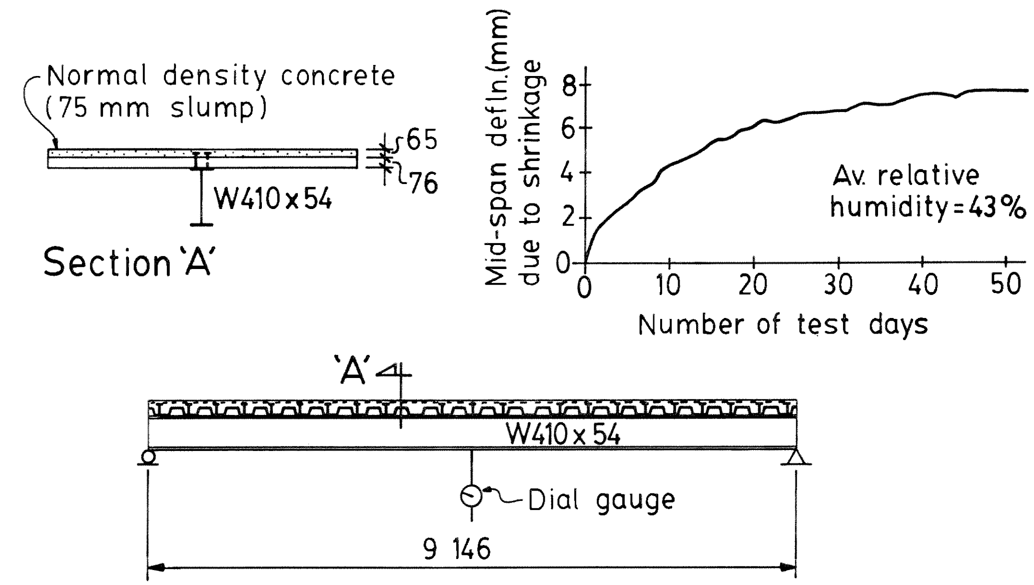


Figure 4.12
Shrinkage Deflection Tests at McMaster University

In the case of a hollow composite girder, where the deck flutes run parallel to the span of the girder, a reduction in slab shrinkage (due to additional restraint of deck steel) should be assumed during the estimation of deflections due to drying shrinkage of slabs. The amount of restrained shrinkage in slabs of hollow composite spandrel members should generally be less than that found from interior members, due to the lowering of effective slab to steel area ratio. Thus, in computing deflections of spandrel members due to restrained slab shrinkage, values lower than 350 micro strain may be considered appropriate.

Drying shrinkage of slabs at beam supports of interior bay composite beams can cause a tension stiffening effect[4.21]. The use of rebar details as in Figure 1.16, and member-end connection details further reduce the beam deflection. (Also see worked example, 4.14.) It is fair to conclude that the proposed restrained shrinkage strain of 200 micro strain for use with composite member design as given by Viest[4.4] appears to be appropriate.

Control of Yielding Deformation

If no temporary shores are provided, the bare steel shape must support the fresh-concrete condition loads (see definition Table 3.3), and hence its bottom flange experiences higher tensile strains than that of a shored member. In order to prevent permanent deformation due to steel yielding under service load conditions, the total bottom steel tensile strain due to full specified loads must be within the elastic range[4.24]. In accordance with Clause 17.6 of S16.1, stresses in the tension flange due to loads applied before the concrete strength attains 0.75 f'_c plus superimposed stresses due to the remaining specified loads considered to act on the composite section shall not exceed $0.9F_y$, if the member is unshored during construction. Assuming elastic behaviour, this requirement can be written as

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} \leq 0.9\ F_y \qquad 4.18$$

Where

M_b = moment due to specified fresh-concrete condition load acting on bare steel section,
M_t = moment due to specified superimposed loads acting on composite section,
S_x = elastic section modulus of bare steel section,
S_t = elastic section modulus with respect to bottom flange of composite section.

Camber Requirements

Deflection of an unshored steel member during construction depends on the member's stiffness and support conditions. Deflection during concrete placement can result in higher concrete consumption (if concrete is screeded level) and thus can lead to additional loading similar to 'ponding' on a roof member. Secondary members are supported by primary members which also deflect, and deflections of primary and secondary flexural members are interactive and additive. Thus the final steel framed floor, including steel deck, beams and girders, may be subjected to more than the calculated load.

Cambering of beams and/or girders for fresh-concrete condition loads is a common method of avoiding this problem when the calculated member deflection during concreting exceeds 20 mm. Cold bending in a gag press, hydraulic jacking at third points in a simple jig, or heat cambering are common and acceptable methods of shop cambering. In addition to cambering for fresh-concrete condition loads, the camber allowance for longer span members, say in excess of 10 to 12 metres, may include the calculated deflection of the **composite** member due to superimposed dead loads and some shrinkage. However, this approach can create a condition opposite to the 'ponding' condition and care should be taken to ensure uniform concrete cover slab thickness in critical areas. One frequently finds that the "calculated" deflections due to concrete placement do not occur. This phenomenon can be partially explained by the fact that the steel beam may have been specified with a slight over camber. Also, end restraint is provided by almost any connection and some "supporting members" are stiffer than others. One should also consider that hot rolled steel products are not precisely straight. Permissible tolerances for out-of-straightness are published in CAN3-G40.20 "General Requirements for Rolled or Welded Structural Quality Steel"; and using a 9 m W shape as an example, the maximum deviation from "straight" may be 9000/1000 = 9 mm. Clause 26.9.5 of S16.1 requires that beams with bow within straightness tolerance shall be fabricated so that after erection the bow due to rolling or fabrication shall be upward.

4.9 INTERACTION OF COMPOSITE BEAMS OR GIRDERS WITH DECK-SLAB SYSTEMS

Several phenomena related to the interaction between compositely designed steel beams and their load-sharing partner, the deck-slab system, require discussion. Performance and load capacity of stud shear connectors have already been discussed in Chapter 2. Local response of the deck-slab system to the forces produced by the stud shear connectors is an aspect requiring further discussion. Likewise, the response of the deck-slab system to external forces, such as negative bending over the girder caused by secondary framing and deck deflections due to applied loads, concrete shrinkage accumulations at areas of least restraint, and changes in deck-slab cross-sectional area due to the fluted nature of the steel deck profile, all may influence local performance.

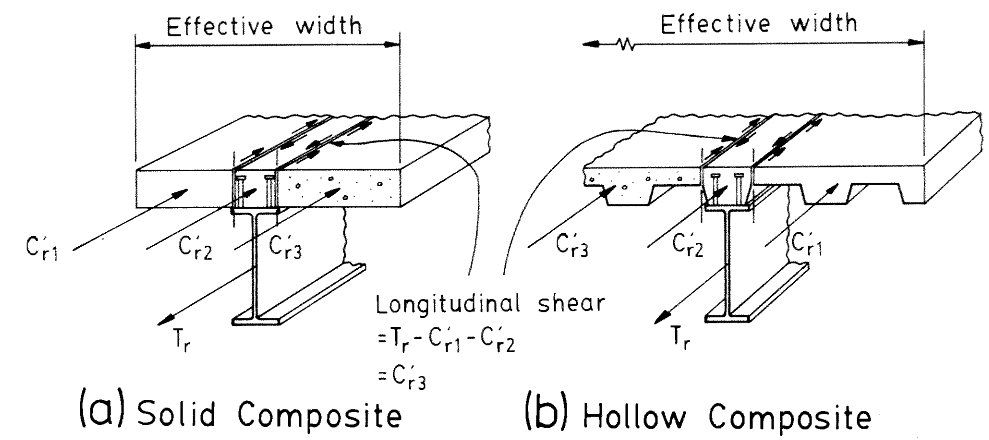


Figure 4.13
Longitudinal Shear due to Composite Action

If we examine the shear force transfer mechanism from steel girders to studs to deck-slabs, we will find a tendency for the concrete slab to shear adjacent to the girder, with the highest slab shear stresses found at the first deck flute because of the abrupt reduction in slab area (see Figs. 4.13a and 4.13b).

Researchers Johnson and Oehlery[4.25] found an occasional tendency for wedging action to split the concrete at stud locations, in lab specimens (Fig. 4.14). However, this phenomenon has not been observed under field conditions, possibly due to their conclusions that in order to be of significant influence, several of the following criteria must exist:

- a low ratio of span to width of slab,
- a high proportion of the total load applied as a single point load,
- a high intensity of longitudinal shear for which transverse reinforcement in excess of 1% of the area of the slab has to be provided,
- shear connection distributed over a narrow width of slab (e.g. a single row of studs).

The longitudinal shear strength of the deck-slab system is related to internal and external influences, and to transverse reinforcement. The following paragraphs will highlight the significant considerations, and differentiate between the "beam" application, with deck perpendicular, and "girder" applications, with deck parallel to the steel member.

Tests on scaled down composite beam specimens using solid slab top chords, by Davies[4.26], revealed that there was no gain in composite beam ultimate strength for amounts of transverse reinforcement in excess of one percent of slab area. His test data further illustrated that a reduction of only four percent of the ultimate beam strength was recorded when transverse reinforcement of 0.47 percent was used instead. From these results, one might conclude that the ultimate flexural strength of a composite beam is rather insensitive to a change in the amount of slab transverse

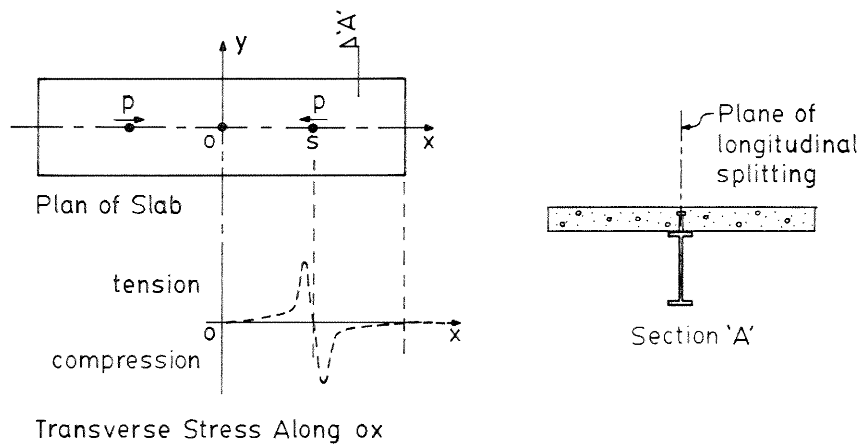


Figure 4.14
Shear Connector Induced
Longitudinal Splitting of Slabs

reinforcing. In the same work[4.26], Davies hypothesized that longitudinal cracks should develop initially in the lower half of the concrete slab, and that only transverse reinforcing close to the bottom of a slab might be credited as effective in resisting such crack action. Later research by Johnson[4.27] indicated that all transverse reinforcement contributes to longitudinal shear strength irrespective of its level in the slab.

The CISC Commentary to S16.1 indicates that longitudinal shear cracking of deck-slabs has not been observed in hollow composite beams, where deck flutes run perpendicular to beam span, possibly because the steel deck provides a measure of reinforcement.

Full-scale hollow composite beam specimens were tested by El-Ghazzi[4.28] and Azmi[4.29]. Although the researchers did not provide a solution to quantify the effects of the amount of transverse reinforcing on the shear resistance of the deck-slab, the test data did show an improvement of 6 percent in the ultimate flexural strength on a 9-metre composite beam resulting from an increase in the transverse reinforcement from 0.096 percent of slab area to 0.263 percent, an increase of 270 percent. In addition, El-Ghazzi suggested that due to the participation of the steel deck, cracks should not necessarily start at the lower part of the slab and then propagate gradually to the top surface, as had been reported[4.26] for solid slab composite beams.

While hollow composite "beams" are generally not critical in longitudinal shear resistance due to the orientation of deck flutes perpendicular to the "beam" spans, detailed design checks should be carried out for hollow composite "girders" where (due to deck flutes being parallel to the steel member) the contribution of the steel deck to the resistance of longitudinal cracking of the deck-slab becomes negligible, especially where steel decks are "cut and spread" (Fig. 1.5c) over a girder to accommodate a particular deck module, to permit fitting of beam flanges under the girder flange (Fig. 4.2), or to permit studs to be welded directly to the girder top flange.

It was noted earlier that in the girder application, external forces can create a more critical situation. Tensile forces in the slab caused by beam-end rotation due to beam deflection under load, deck and shear stud discontinuity causing shrinkage accumulations at the girder, no deck negative bending strength transverse to the girder, and the fact that deck-slab systems over girders will normally be more highly stressed than in the beam-deck-slab configuration, all contribute to this tendency to longitudinal cracking.

With the lack of research data on the girder-deck-slab configuration a need has existed for further information on the influence of slab cracking (from whatever cause), and the influence of transverse reinforcement on slab cracking and ultimate strength of the composite 'girder' section.

Recent tests of such composite girder assemblies by Robinson[4.30] were conducted to compare two levels of slab reinforcement. The assemblies were designed using LSD methods, assuming a 9 m × 9 m bay configuration. An assembly with a nominal 152×152 MW9.1×MW9.1 mesh reinforcement in a 65 mm cover slab on 76 mm wide-rib profile deck on a girder span of 9 m reached full specified loads prior to hair line cracking of the slab. Significant longitudinal slab cracking developed at the attainment of ultimate load. A second assembly, identical in geometry but incorporating additional reinforcing, as shown in Fig. 1.15, attained full ultimate load with better crack distribution and much smaller cracks. No significant variation in ultimate load capacity was found.

It should be noted that test apparatus limitations precluded the application of loads in such a manner that would cause negative transverse bending (other than that caused by the self weight of the structure) across the girder cross-section at the beam girder joints. Further testing incorporating this additional feature would be helpful. However, one might conclude from the two tests above that, even with more realistic transverse bending applied to the section, longitudinal cracking at service loads would be controlled to acceptable levels by the addition of reinforcing similar to the pattern used in the second test.

Study of longitudinal shear capacities of slabs by Buckner et al[4.31] has produced a proposed rational design method as follows:

a) The longitudinal shear strength should be sufficient to develop the capacity of the shear connectors, neglecting any transverse forces which might be created by membrane action of the slab.

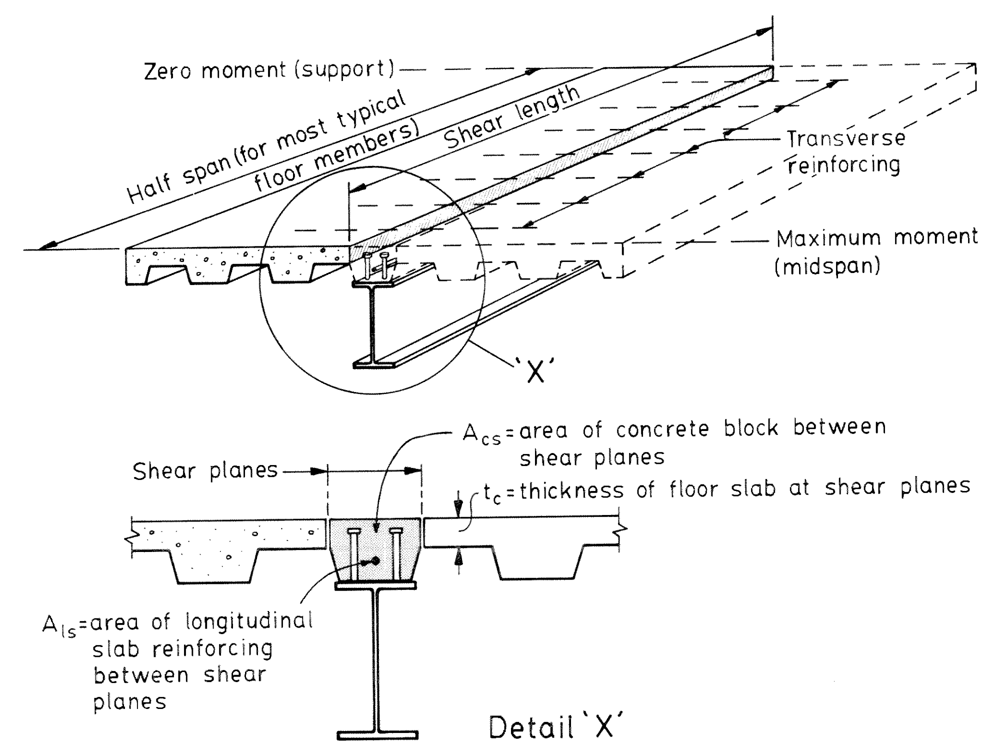


Figure 4.15
Longitudinal Shear Resistance of
Deck-Slabs in Hollow Composite Members

b) The critical planes for longitudinal shear are adjacent to the rows of connectors as indicated in Fig. 4.15. The ultimate longitudinal shear force, V_u, to be transferred across these planes can be approximated by the expression,

$$V_u = Q_u - 0.85 f'_c A_{cs} - A_{ls} f_y$$

in which Q_u = the ultimate horizontal shear capacity of the connectors,
f'_c = the specified compressive strength of concrete,
A_{cs} = the area of concrete block between shear planes,
A_{ls} = the area of longitudinal slab reinforcement between shear planes, and
f_y = the specified yield strength of the reinforcing steel.

The distribution of ultimate shear load in the connectors can be assumed to be uniform. Thus, for the case of a slab extending both sides of a member, the nominal ultimate longitudinal shear stress can be expressed as:

$$v_u = V_u/(2 l_{sh}\, t_c) \qquad 4.19$$

in which l_{sh} = the shear length under consideration, t_c = slab thickness in shear.

c) The shear stress computed by Eq. 4.19 should not exceed the limits proposed by Mattock, et al[4.32,4.33]. For normal density concrete, these limits are:

$$v_u \leq (0.8\, \rho\, f_y + 2.76)\ \text{MPa, and} \leq 0.3\, f'_c \qquad 4.20$$

The limits for semi-low density concrete are:

$$v_u \leq (0.8\, \rho\, f_y + 1.72)\ \text{MPa,} \leq 0.2\, f'_c\text{, and} \leq 6.9\ \text{MPa} \qquad 4.21$$

and for all low density concrete:

$$v_u \leq (0.8\, \rho\, f_y + 1.38)\ \text{MPa,} \leq 0.2\, f'_c\text{, and} \leq 5.52\ \text{MPa} \qquad 4.22$$

in which ρ = the ratio of transverse reinforcement; and f_y = the specified yield stress of the reinforcement.

d) At least half the reinforcement required by expressions 4.20 to 4.22 should be placed near the bottom of the slab, in the case of a solid slab.

e) The longitudinal shear stress should be assumed to vary linearly from its critical value, given by Eq. 4.19, to zero at the extreme edges of the effective width of slab. The area of transverse reinforcement can be reduced accordingly. In the absence of an established overall performance factor for longitudinal shear, ϕ_v = 0.60 is assumed in all example calculations in this publication.

$$\phi_v\, (V_u + 0.85\, f'_c A_{cs} + A_{ls} f_y) \geq Q_r \qquad 4.23$$

4.10 SHORED COMPOSITE BEAMS AND GIRDERS

Shored composite beam construction refers to the situation where the steel shapes are supported by shores during placing of the concrete cover slab. These shores remain in place until the concrete has attained about 75% of its 28 day strength. The advantages and disadvantages of shored construction of composite beams and girders incorporating rolled and welded H shapes were discussed and illustrated in an example by Ritchie and Chien[4.34]. These can be summarized as follows:

Advantages
- eliminates the need to select a larger or deeper section which will normally be required to support construction slab load and to control deflection
- eliminates need for cambering
- eliminates 'ponding' of concrete due to member deflection during slab pour thus controlling concrete quantities, avoiding potential deck overloading and making level screeding easier

Disadvantages
- defeats the traditionally desirable features of steel construction, (i.e. simplicity and immediate access by other building trades)
- requires the additional cost of shoring and shore removal, plus the fact that shores must stay in place several floors below
- more susceptible to creep deflection as the steel/concrete composite section must carry additional long-term dead load
- instantaneous deflection of beams at shore removal accentuates the negative bending at supports, amplifying the tendency to slab cracking
- requires greater accuracy of design calculations and construction quality control, including negative reinforcement at beam-girder joints
- an unshored design will normally have a greater overload capacity than a shored design

Traditionally, most Canadian construction is designed as unshored construction, with the exception being projects using "stub-girder" construction.

4.11 WEB OPENINGS IN COMPOSITE BEAMS

One important aspect of building design is the accommodation of mechanical ducts and electrical conduits within the "plenum" between the floor and the ceiling in an efficient manner. Structural floor systems using stub-girders, trusses and open web steel joists provide natural web openings for the passage of ducts and pipes. For beams and girders with solid webs, air ducts may pass below the beams and girders, or through web openings.

If economical ducts will not pass beneath the beams, beam depth may be reduced by providing a cover plate to the bottom flange but this approach usually results in an uneconomical design. Web openings are the more normal approach. Composite members with web holes are first selected in accordance with design requirements for composite members without web holes. Then, the effects due to the presence of the hole(s) must be evaluated and accounted for. If a web hole is favourably located to avoid regions of high shear, its effect on the strength and the behaviour of the member may be small. Depending on the size and the location of the hole, solutions in order of economic preference might be: an unreinforced hole; a lightly reinforced hole; a heavier or deeper section with an unreinforced hole; a heavier or deeper section with a lightly reinforced hole.

A guide for the design of composite beams with unreinforced web hole(s) is outlined by Redwood and Wong[4.35]. The capacity of bare steel members under construction loads may be evaluated in accordance with the procedures outlined in the CISC Handbook of Steel Construction or by Redwood and Shrivastava[4.36]. Large, heavily reinforced web holes can be very costly to fabricate and should be avoided if possible or at least restricted to a very few primary framing members. In any event, the total cost required to provide passage of service ducts through webs of floor members should be weighed against the additional cost of a storey height increase. Other systems such as stub-girders, trusses and open web steel joists may also be considered.

4.12 SPANDREL MEMBER DESIGN CONSIDERATIONS

Detailing of the spandrel framing/wall interface requires early and thorough attention to avoid

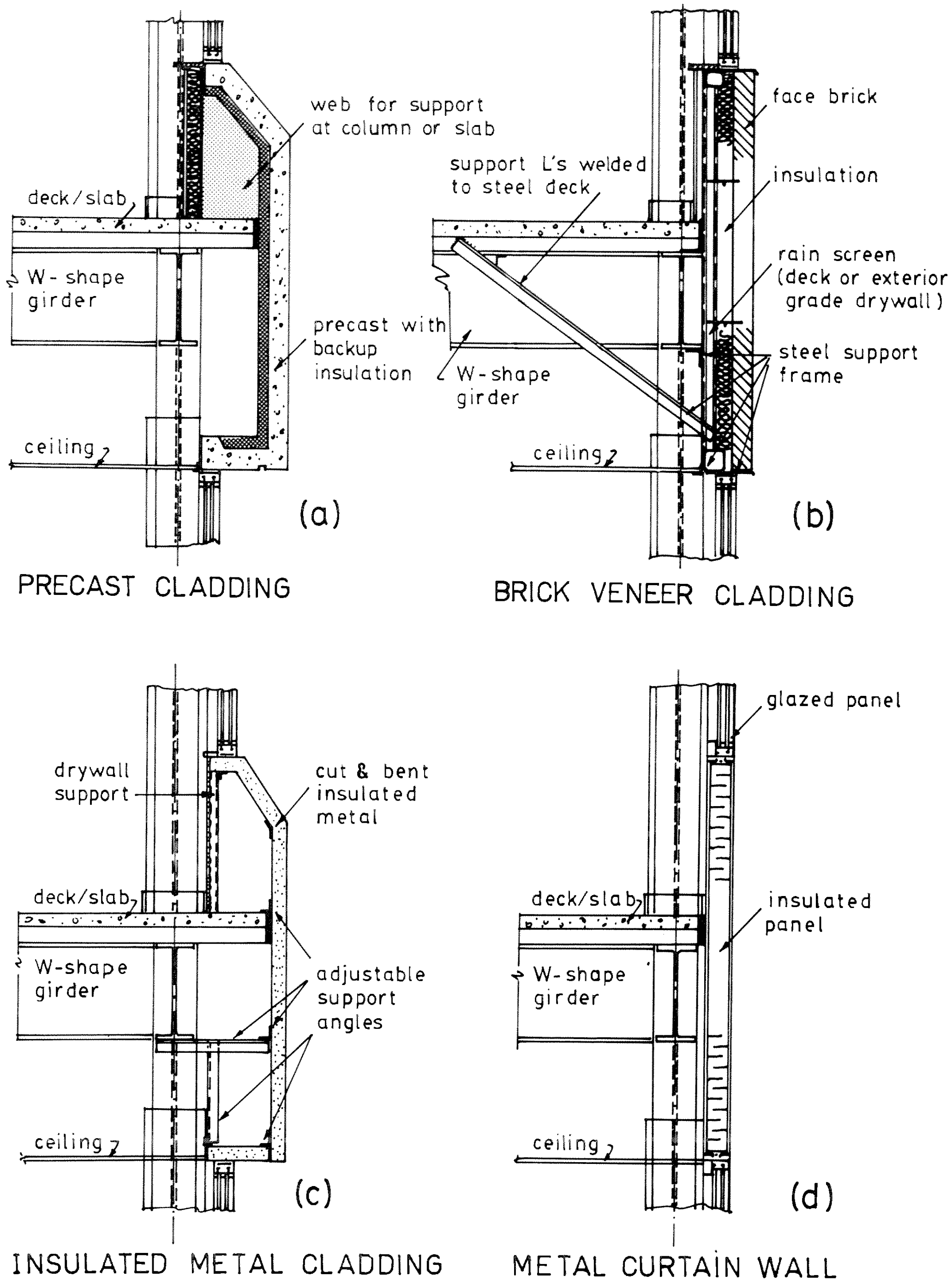


Figure 4.16
Exterior Wall Systems

cost penalties to both the steel frame and the cladding. To begin the design of a spandrel member, the designer must obtain detailed design information on:

- the mass and structural properties of the cladding, including backup material,
- its construction sequence,
- the location of the wall loading in relation to the spandrel member, and
- the position and details of acceptable wall-attachment frames at the spandrel.

Composite spandrel beams and girders incorporating rolled and welded H shapes can be designed relatively 'free' of torsional effects by using details of the type shown in Figures 4.16 (a) to (d). Composite spandrel members, so designed, are usually efficient both structurally and economically. Composite action, for stiffness only, may be considered.

Some practical considerations include:

- in the use of precast concrete cladding with horizontal accentuation, consideration should be given to supporting the precast either directly off column brackets, or off spandrel frame brackets very close to their column connections,
- the load-deflection behaviour of spandrel member at every stage of construction,
- the relative stiffness of adjacent wall-supporting members,
- the structural treatment of torsional load caused by the spandrel details,
- the adjustability of the attachment frames supported by the spandrel members, see Fig. 4.17.
- the treatment of slab overhangs at spandrel beams and girders, (see Figs. 4.18 and 4.19),
- the effects of wind or earthquake loading.

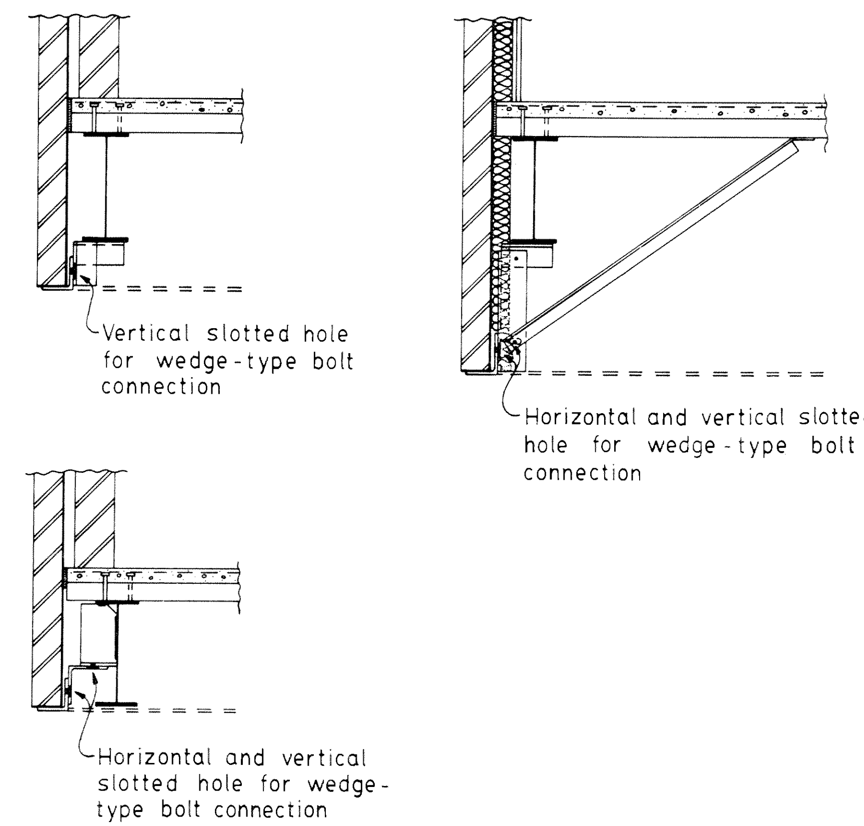
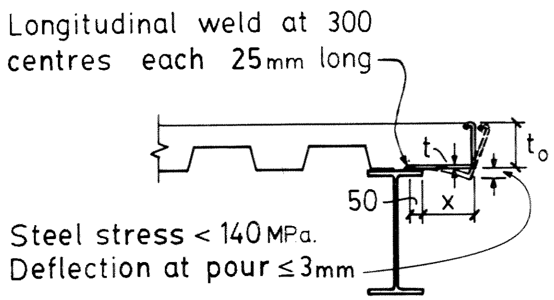


Figure 4.17
Adjustable Wall Supporting Framework Details



t_o (mm)	x (mm)	t (mm)
100	50	0.91
	95	1.22
	140	1.52
	180	1.91
	265	2.67
	340	3.43
140	35	1.22
	80	1.52
	120	1.91
	200	2.67
	285	3.43

Figure 4.18
Required Thickness of Cold-Formed Screed Flash at Spandrel Members

4.13 COMPOSITE BEAM TABLES

The Composite Beam and Girder Selection Tables found in this chapter provide composite and bare steel sectional properties and design data for composite members using W-shapes produced in Canada with nominal depths of 200 mm and deeper, including two WWF-shapes of 700 mm depth. The design data are computed based on CAN3-G40.21-M81 300W steel. Eight combinations of solid slab or cover slab thicknesses and deck depths for two specified concrete strengths are included in Tables 4.1 to 4.8:

Table **4.1** 130 mm solid slab with a specified concrete compressive strength, f'_c, of 20 MPa
Table **4.2** 38 mm steel deck and 65 mm cover slab with f'_c of 20 MPa
Table **4.3** 51 mm steel deck and 65 mm cover slab with f'_c of 20 MPa
Table **4.4** 76 mm steel deck and 65 mm cover slab with f'_c of 20 MPa
Table **4.5** 76 mm steel deck and 90 mm cover slab with f'_c of 20 MPa
Table **4.6** 76 mm steel deck and 75 mm cover slab with f'_c of 25 MPa
Table **4.7** 51 mm steel deck and 85 mm cover slab with f'_c of 25 MPa
Table **4.8** 76 mm steel deck and 85 mm cover slab with f'_c of 25 MPa

For each combination, composite sections are listed in descending order of nominal depth and mass of steel shape. The following section properties and design data are tabulated:

- b = flange width of steel shape, in millimetres.
- t = flange thickness of steel shape, in millimetres.
- d = overall depth of steel shape, in millimetres.
- b_1 = effective width of slab used in computing values of M_{rc}, $Q_{r100\%}$, I_t and S_t, in millimetres ($b_1 \leq b + 16t_o$).
- M_{rc} = factored moment resistance of composite section for percentages of shear connection of 50, 75 and 100, in kilonewton metres.
- $Q_{r100\%}$ = required sum of all the factored shear resistances of connectors between point of maximum moment and its adjacent point of zero moment, for 100% shear connection, in kilonewtons, $Q_{r100\%}$ = lesser of $\phi\, A_s\, F_y$ or $0.85\, \phi_c\, b_1\, t_c\, f'_c$.
- I_t = moment of inertia of the composite section, mathematically transformed into steel properties, computed by neglecting concrete in tension, using mass densities shown with each table, in units of 10^6 mm^4.
- S_t = section modulus of the composite section with respect to the extreme fibre of the steel bottom flange based on the value of I_t, in units of 10^3 mm^3.

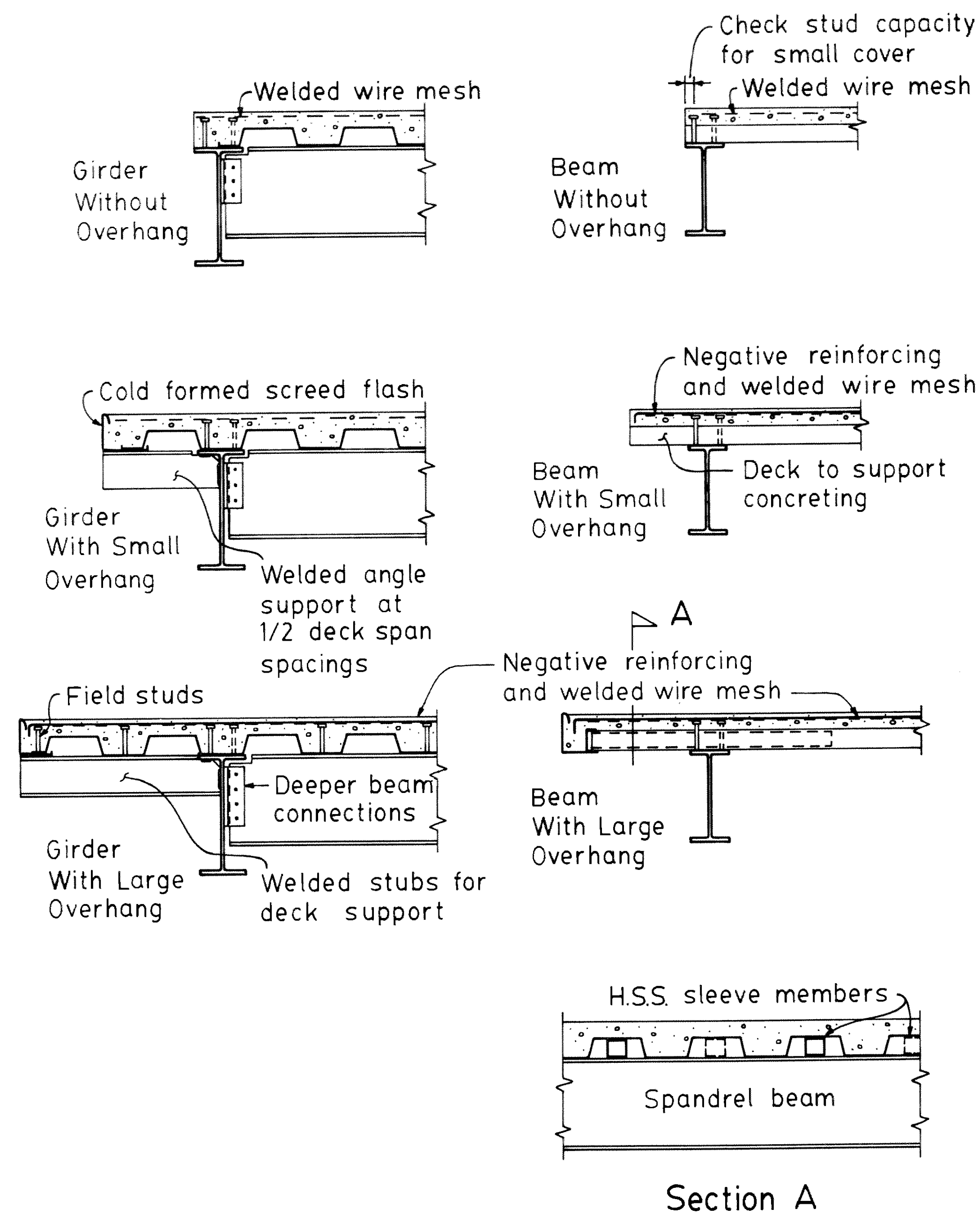


Figure 4.19
Typical Slab Overhang Arrangement for Spandrel Members

M_r	=	factored moment resistance of laterally supported bare steel section, in kilonewton metres.
V_r	=	factored shear resistance of steel section, computed in accordance with Clause 13.4.1 of S16.1 for the appropriate h/w ratio, in kilonewtons.
L_u	=	maximum unsupported length of compression flange of the bare steel member for which no reduction in M_r is required, in millimetres.
I_x	=	moment of inertia about the x-x axis of the bare steel section, in units of 10^6 mm^4.
S_x	=	elastic section modulus of bare steel section, in units of 10^3 mm^3.
M_r'	=	factored moment resistance of the bare steel member for an unsupported length of L′, in kilonewton metres.

4.14 FLOOR DESIGN EXAMPLE

The following example illustrates the design of some typical members in a hollow composite beam and girder floor framing system. One of the composite members will be selected using step-by-step hand calculations, to demonstrate the basic mechanics in the design of a composite member. The result will then be compared to a much quicker solution obtained by using the selection tables. All other members (and components) will be designed by fully utilizing the design aids provided in this publication.

Since all materials that are covered in Chapters 1 to 4 are required for the design of a composite floor, each pertinent area will be duly highlighted in the example.

The quarter floor plan of a typical floor in a multi-storey office building is shown in Fig. 4.E1. Select: interior beams B1, B2, spandrel beam SB, interior girder G and spandrel Girder SG, given specified loads and design requirements as follows:

Fire resistance rating of floor system: 2 hours

Storey height given:

floor to floor height	=	3 660 mm
floor to ceiling height	=	2 590 mm
plenum depth	=	1 070 mm
maximum longitudinal duct depth	=	350 mm

Type of construction: unshored construction with shop-cambered beams and girders

Specified loads:

a) live load = 2.4 kPa (25% sustained), $RF_2 = 0.3 + \sqrt{9.8/A}$

b) fresh-concrete condition dead load = deck + concrete on deflected deck + steel shape

c) superimposed dead loads:

mechanical and ceiling	=	0.5 kPa
fire protection and floor finishes	=	0.2 kPa
partitions	=	1.2 kPa (80% sustained)
spandrel wall	=	3.8 kN/m

d) construction loads: given in Table 3.2

Deflection limitations:

a) interior members:

deflection due to all superimposed specified loads after installation of ceilings ≤ span/300

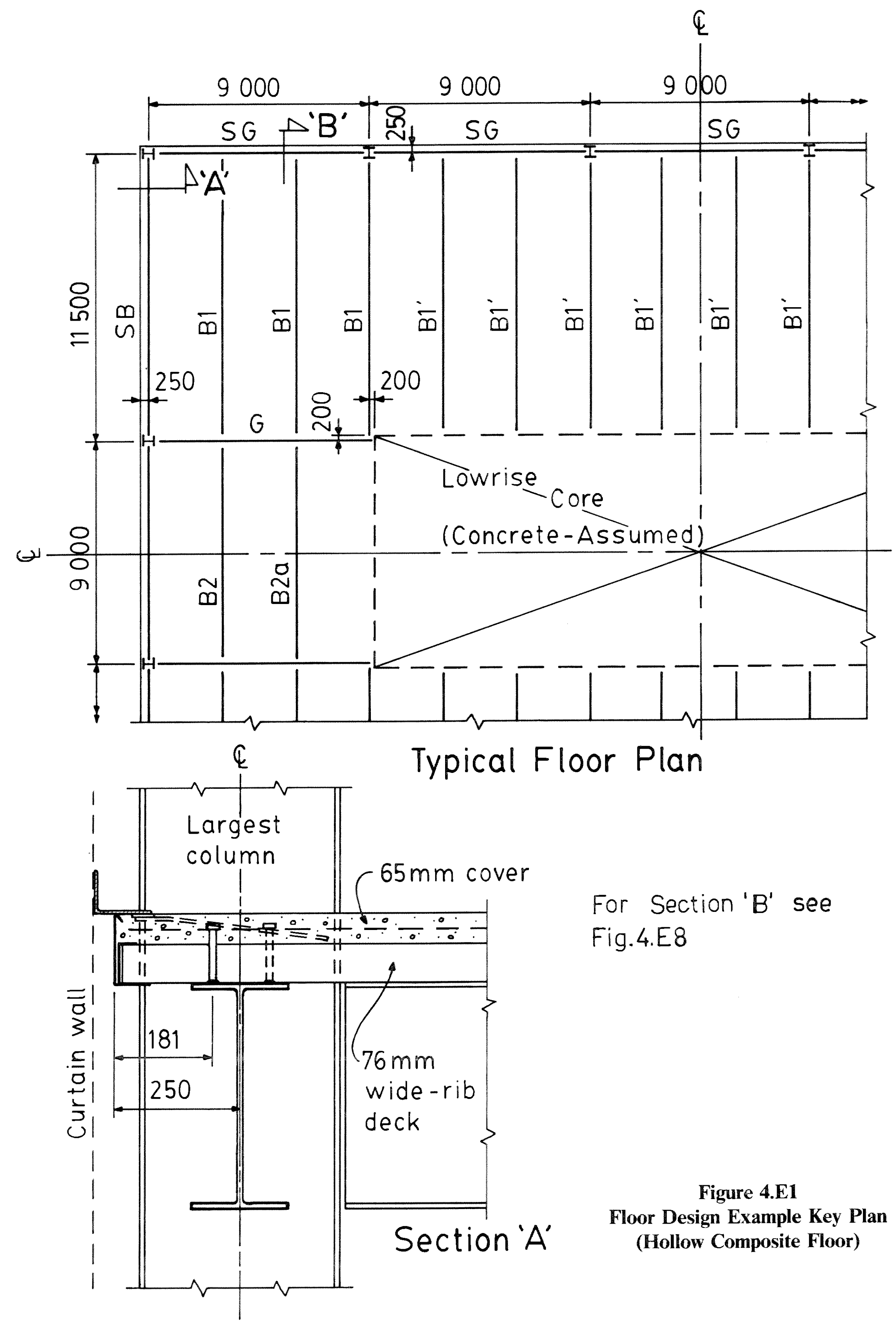


Figure 4.E1
Floor Design Example Key Plan
(Hollow Composite Floor)

b) spandrel members:

deflection due to all superimposed specified loads after concrete casting $\leq$ span/360,

instantaneous deflection due to cladding mass $\leq$ 12 mm (assumed as a chosen criterion for this project)

Materials:

structural steel	: G40.21-M 300W
steel deck	: CSSBI 101 M84, Grade A
shear studs	: 19 mm diameter
concrete	: f'_c = 20 MPa; w_c = 2 300 kg/m^3

Solution

Steel deck: 76 mm wide-rib composite steel deck is selected to span 3 metres.

Fire resistance rating requirement: A floor system consisting of a 65 mm normal density concrete cover slab on 76 mm steel deck with sprayed fire protection on steel beams, girders and deck underside in accordance with ULC Design No. F817 provides the required fire rating.

Steel deck nominal thickness: The steel deck serves as a form and as a construction platform before concrete hardens and subsequently becomes the positive moment reinforcing steel for the cured slab. Although design approaches that are adopted may vary from one deck manufacturer to another, the design procedures for deck-reinforced slabs usually include checking the following criteria:

a) steel deck as form and construction platform
- flexure of steel deck under fresh concrete plus construction loads (see Table 3.2),
- web stability (against crippling at supports) under fresh-concrete and construction loads (see Table 3.2),
- deck deflection due to the total mass of deck and concrete including the effect of concrete ponding ($\Delta \leq$ span/180 $\leq$ 20 mm).

b) deck-reinforced slab
- shear bond between steel deck and concrete,
- flexural tension in steel deck,
- flexural compression in concrete,
- deflection due to superimposed occupancy loads.

In this example, a wide-rib composite steel deck of 0.91 mm nominal thickness that satisfies all the criteria above is selected. Design parameters below are listed in the manufacturer's catalogue.

Dead load due to deck mass, q_d	= 0.10 kPa
Moment of inertia of deck , I_d	= 1.10×10^6 mm^4/m
Dead load due to deck-slab , q	= 2.40 kPa

Interior Beam, B1 – using detailed hand calculations

– live load:

Tributary area, A = 3.0(11.5) = 34.5 m^2

Reduction factor, $RF_2 = 0.3 + \sqrt{9.8/34.5} = 0.83$

Total live load per beam, W_L = 0.83 (2.4)(34.5) = 68.7 kN

– Fresh-concrete condition dead load including concrete ponding:

$w = (1 + 0.20\, w_c\, s^4/I_d)\, s\, q$ Table 3.1, triple span

$= [1 + 0.20(2\,300)(3)^4/1.10\times10^6]\, s\, q$

$= 1.034(3)(2.4)$

$= 7.44$ kN/m

Total fresh-concrete condition load per beam,

$W_c = (7.44 + 0.6)(11.5) = 92.5$ kN   (0.6 kN/m steel beam assumed)

– Superimposed dead loads:

Partition load per beam, W_p = 1.2(34.5) = 41.4 kN

Other dead loads per beam, W_{OD} = (0.5+0.2)(34.5) = 24.2 kN

– Factored loads, moment and shear:

$W_f = \alpha_L W_L + \alpha_D(W_c + W_p + W_{OD})$

$= 1.5(68.7) + 1.25(92.5+41.4+24.2) = 301$ kN

$M_f = W_f L/8 = 301(11.5)/8 = 433$ kN·m

$V_f = W_f/2 = 301/2 = 151$ kN

Trial section: In order to maintain a plenum depth of 1070 mm and to accommodate a 350 mm deep duct without providing web openings, the steel beam depth is restricted to 410 mm as shown in Figure 4.E2.

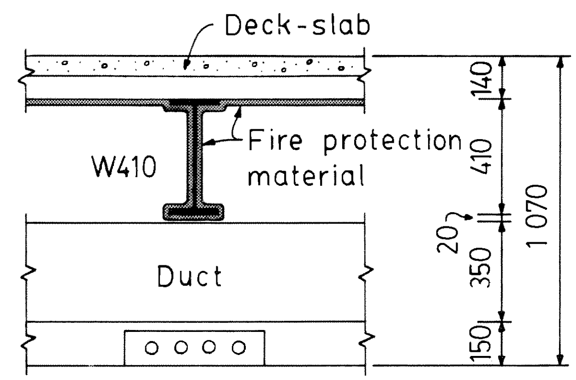


Figure 4.E2
Plenum Depth Computation

Try W410×60   L/d = 11 500/410 = 28 < 30 OK

From CISC Handbook of Steel Construction, Page 6-44,

b = 178 mm	t = 12.8 mm	d = 407 mm	w = 7.7mm
A_s = 7 580 mm^2	I_x = 216×10^6mm^4	S_x = 1.06×10^6mm^3	Z_x = 1.19×10^6mm^3
I_y = 12.0×10^6mm^4	J = 328×10^3mm^4	C_w = 468×10^9mm^6	

self dead load = 0.584 kN/m

– compute factored moment resistance of composite section, M_{rc}

L/4 = 11 500/4   = 2 875 mm

$16t_o + b = 16(141) + 178 = 2\,434$ mm (Governs)

beam spacing, s   = 3 000 mm

Therefore effective slab width, b_1 = 2 434 mm

$0.85\ \phi_c\ b_1\ t_c\ f'_c = 0.85(0.6)(2\ 434)(65)(20)\ 10^{-3} = 1\ 614$ kN
$\phi\ A_s\ F_y = 0.9(7\ 580)(300)\ 10^{-3} = 2\ 047\ \text{kN} > 1\ 614$ kN
Hence, total factored horizontal shear for 100% composite action,
$Q_{r100\%}$ = the lesser of $0.85\ \phi_c\ b_1\ t_c\ f'_c$ and $\phi\ A_s\ F_y$
= 1 614 kN

compute factored shear resistance per shear stud, q_r

$E_c = w_c^{1.5}(0.043)\ \sqrt{f'_c} = (2\ 300)^{1.5}(0.043)\ \sqrt{20} = 21\ 210$ MPa
F_u of shear stud = 415 MPa
$415\ \phi_{sc}A_{sc} = 415(0.8)(19^2\pi/4)\ 10^{-3} = 94.1$ kN (stud diameter $\simeq$ 19 mm, or = ¾ inch)
$0.5\ \phi_{sc}A_{sc}\ \sqrt{f'_c\ E_c} = 0.5(0.8)(19^2\pi/4)\ \sqrt{20(21\ 210)}\ 10^{-3} = 74.3$ kN

q_r = lesser of $0.5\ \phi_{sc}A_{sc}\ \sqrt{f'_c\ E_c}$ and $415\ \phi_{sc}A_{sc}$ (Eq. 2.3)
= 74.3 kN
$2.5t = 2.5(12.8) = 32\ \text{mm} > 19$ mm, and

$$\frac{W_{rib}}{t_d} \geq 2.0 \text{ (wide-rib)}$$

Therefore, there is no reduction in q_r, if not more than one stud per rib. i.e. $q_r = 74.3$ kN

Minimum number of shear studs per beam requird to provide 50% shear connection

$$= 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\ 614}{74.3}\right) = 21.7,$$ i.e provide at least 22 studs per beam
(use **24 studs** per beam, or 55% connection)

$Q_r = 24(74.3)/2 = 892$ kN

$$a = \frac{Q_r}{0.85\ \phi_c\ b_1\ f'_c} \qquad \text{(Eq. 4.10)}$$
$$= \frac{892}{0.85(0.6)(2\ 434)(20)} \times 10^3 = 35.9 \text{ mm}$$

Note: with partial shear connection, the plastic neutral axis, P.N.A. must lie in the steel section. See Fig. 4.E3

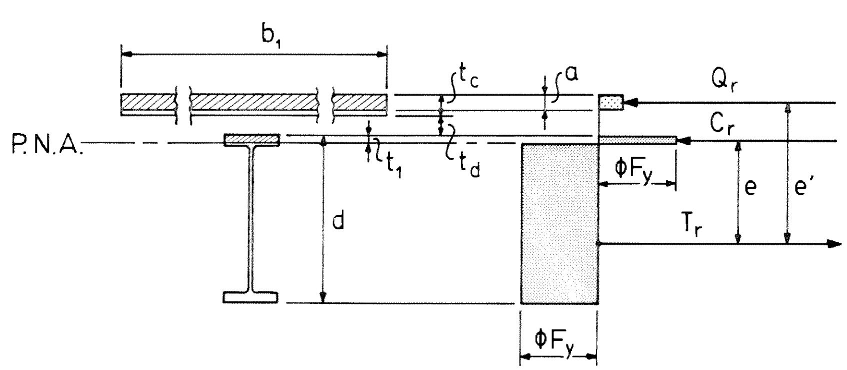


Figure 4.E3
Internal Forces of Beam 'B1'
(with Partial Shear Connection)

$$C_r = \frac{\phi\ A_s\ F_y - Q_r}{2} \qquad \text{(Eq. 4.11)}$$

$$= \frac{2\ 047 - 892}{2} = 578 \text{ kN}$$

Locate P.N.A. Factored axial resistance of steel top flange
$= \phi\ b\ t\ F_y$
$= 0.9(178)(12.8)(300)\ 10^{-3} = 615\ \text{kN} > 578$ kN
i.e. P.N.A. in steel top flange

Distance from top of steel section to P.N.A.,

$$t_1 = \frac{C_r}{\phi b\ F_y} = \frac{578}{0.9(178)(300)} \times 10^3 = 12.0 \text{ mm}$$

Lever arms,

$$e = \frac{(A_s d - b t_1^2)}{2(A_s - b t_1)} - \frac{t_1}{2} \qquad \text{(Eq. 4.7a)}$$

$$= \frac{7\ 580(407) - 178(12)^2}{2[7\ 580 - 178(12)]} - \frac{12}{2}$$

= 275 mm

$$e' = e + \frac{t_1}{2} + t_o - \frac{a}{2} \qquad \text{(Eq. 4.7b with } t_c \text{ replaced by a)}$$

$$= 275 + \frac{12.0}{2} + 141 - \frac{34.8}{2} = 405 \text{ mm}$$

Factored moment resistance,

$M_{rc} = C_r e + Q_r e'$ (Eq. 4.12)
$= [578(275) + 892(405)]\ 10^{-3} = 520$ kN·m
$M_f = 433\ \text{kN·m} < 520$ kN·m OK

– factored shear resistance

$V_r = \phi\ A_w\ F_s$ (Eq.4.13)
$= \phi\ d\ w\ (0.66F_y)$
$= 0.9(407)(7.7)(0.66)(300) \times 10^{-3} = 558$ kN
$V_f = 151\ \text{kN} < 558$ kN OK

– construction stage 1 – deck placement (no lateral support provided before deck welded to steel top flange)
$A = 34.5\ \text{m}^2 > 16\ \text{m}^2$ i.e. consider U.D.L. only
$7\ \text{m}^2 < A < 54\ \text{m}^2$.

Therefore,

$q_L' = 0.7 - A/135$ (see Table 3.2)

$= 0.7 - 34.5/135 = 0.44$ kPa

$W_f' = [1.25(0.10) + 1.5(0.44)](34.5) + 1.25(0.584)(11.5) = 35.5$ kN

$M_f' = 35.5(11.5)/8 = 51.0$ kN·m

$$M_u = \frac{\pi}{\omega L'}\sqrt{EI_yGJ + \left(\frac{E\pi}{L'}\right)^2 I_yC_w} \qquad \text{(Eq. 4.16)}$$

$$= \frac{\pi}{1.0(11.5)}\sqrt{\left[200(12.0)(77)(328) + \left(\frac{200\,\pi}{11.5}\right)^2(12.0)(468)\right] \times 10^{-3}}$$

$= 76.0$ kN·m

$\frac{2}{3} M_p = \frac{2}{3} Z_x F_y$ (W410×60 of 300W steel is Class 1 section in bending)

$= \frac{2}{3}(1.19)(300)$

$= 238$ kN·m > 76.0 kN·m. Therefore,

$M_r' = \phi M_u$ (Eq.4.14)

$= 0.9(76.0)$

$= 68.4$ kN·m > 51.0 kN·m OK

– construction stage 2 – concrete placement

$A > 8$ m^2 i.e. consider U.D.L. only

$q_L = 2\,q_L' = 0.88$ kPa (see Table 3.2)

$W_f = 1.5(0.88)(34.5) + 1.25(92.5) = 161$ kN

$M_f = 161(11.5)/8 = 231$ kN·m

$M_r = \phi Z_x F_y = 0.9(1.19)(300) = 321$ kN·m > 231 kN·m OK

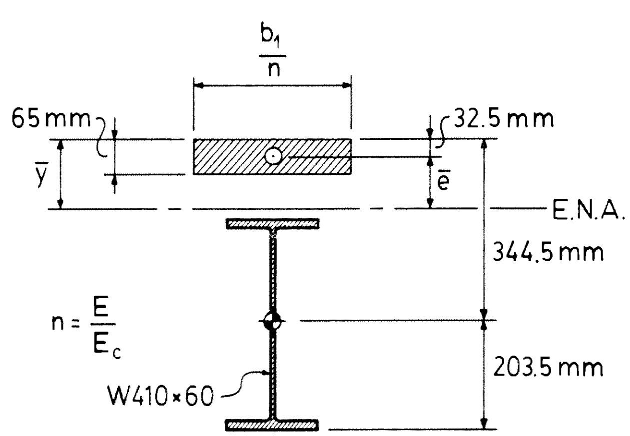


Figure 4.E4
Cross-Section of Composite Beam 'B1'
(Transformed into Elastic Steel Properties)

– Sectional properties of composite beam (see Fig. 4.E4)

a) Find moment of inertia of composite section, I_t (in steel unit)

$n = E/E_c = 200\,000/21\,210 = 9.43$

Section	Transformed Area, A (mm²) in steel unit	Distance from top of slab, y (mm)	Ay mm³	Ay² mm⁴	I_{local} mm⁴ in steel unit
Concrete	16 777	32.5	545 252	17.7×10⁶	5.91×10⁶
Steel	7 580	344.5	2 611 310	899.6×10⁶	216×10⁶
Total	24 357		3 156 562	917.3×10⁶	221.9×10⁶

$\bar{y} = \frac{3\,156\,562}{24\,357} = 129.6$ mm > 65 mm

i.e. all effective concrete is in compression

$I_t = \Sigma I_{local} + \Sigma A y^2 - \bar{y}^2 \Sigma A$

$= (221.9 + 917.3) \times 10^6 - (129.6)^2(24\,357) = 730 \times 10^6$ mm^4

$S_t = \frac{I_t}{d+t_o-\bar{y}} = \frac{730\times10^6}{407+141-129.6} = 1.745 \times 10^6$ mm^3

b) Find moment of inertia of composite section reduced to account for concrete creep, I_r. ($E_r = E_c/2.5$)

$n_r = E/E_r = 2.5(200\,000)/21\,210 = 23.6$

Section	Transformed Area, A (mm²) in steel unit	Distance from top of slab, y (mm)	Ay mm³	Ay² mm⁴	I_{local} mm⁴ in steel unit
Concrete	6 704	32.5	217 900	7.08×10⁶	2.4×10⁶
Steel	7 580	344.5	2 611 310	899.6×10⁶	216×10⁶
Total	14 284		2 829 210	906.7×10⁶	218.4×10⁶

$\bar{y} = \frac{2\,829\,210}{14\,284} = 198$ mm > 65 mm

$I_r = (218.4 + 906.7)\times10^6 - (198)^2(14\,284) = 565 \times 10^6$ mm^4

– Deflection estimate

a) Camber requirement:
Deflection of unshored beam under fresh-concrete condition load,

$$\Delta_c = \frac{5W_cL^3}{384EI_x} = \frac{5(92.5)(11.5)^3}{384(200)(216)} \times 10^3 \text{ mm} = 42 \text{ mm}$$

Therefore **camber 40 mm** at mid span.

b) Shrinkage deflection

$$\bar{e} = \bar{y} - \frac{t_c}{2} = 129.6 - \frac{65}{2} = 97.1 \text{ mm}$$

$$\Delta_{sh} = \frac{\bar{e}\,\epsilon\, t_c\, b_l\, L^2}{8\, n\, I_t} \quad \text{(from Figure 4.E5)}$$

$$= \frac{97.1(0.0002)(65)(2\,434)(11.5)^2}{8(9.43)(730)}$$

$$= 7.4 \text{ mm} \qquad \text{(Shrinkage strain, } \epsilon = 0.0002 \text{ assumed)}$$

c) Creep deflection
Total sustained load, W_s
$= 0.25W_L + 0.8W_p + W_{OD}$
$= 0.25(68.7) + 0.8(41.4) + 24.2 = 74.5$ kN

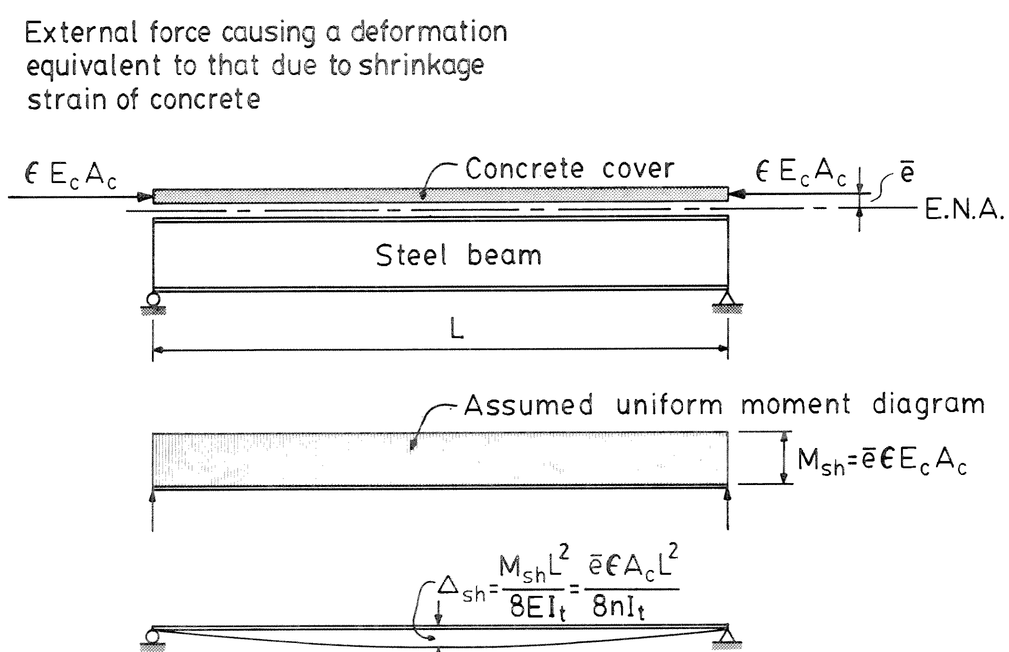


Figure 4.E5
Shrinkage Deflection of Composite Beams
(by analysing the structure as an eccentrically loaded column)

Δ_{creep} = (beam deflection with creep effect)
– (instantaneous deflection of the transformed section)

$$\Delta_{creep} = \frac{5W_sL^3}{384E}\left(\frac{1}{I_r} - \frac{1}{I_t}\right) = \frac{5(74.5)(11.5)^3}{384(200)}\left(\frac{1}{565} - \frac{1}{730}\right) \times 10^3 = 3.0 \text{ mm}$$

d) Deflection due to live load and partition including long-term effects

$$\Delta = \frac{5(W_L + W_p)L^3}{384\, E\, I_e} + \Delta_{creep} + \Delta_{sh}$$

$$= \frac{5(68.7 + 41.4)(11.5)^3\, 10^3}{384(200)(592)} + 3.0 + 7.4$$

$$= 28.8 \text{ mm} = L/399 < L/300 \qquad \text{OK}$$

where I_e = effective moment of inertia
$= I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$ (Eq. 4.17)
$= [216 + 0.85\,(0.55)^{0.25}\,(730 - 216)] \times 10^6 \text{ mm}^4$
$= 592 \times 10^6 \text{ mm}^4$
where $I_s = I_x = 216 \times 10^6 \text{ mm}^4$

– Unshored beam requirement

Moment due to specified fresh-concrete condition load acting on bare steel beam,

$$M_b = W_cL/8 = 92.5(11.5)/8 = 132.9 \text{ kN·m}$$

Moment due to all specified superimposed loads acting on composite beam (i.e. after concrete attains 75% f'_c),

$$M_t = (W_L + W_p + W_{OD})\,L/8 = (68.7 + 41.4 + 24.2)(11.5)/8 = 193.1 \text{ kN·m}$$

Combined stresses in bottom flange under specified loads become,

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{132.9}{1.060} + \frac{193.1}{1.745} = 236 \text{ MPa} < 0.9\,F_y \qquad \text{(Eq. 4.18)}$$

Therefore shoring is not required.

W410×60 with 55% shear connection is satisfactory.

The total amount of computation presented above can be substantially reduced by using the Composite Beam and Girder Selection Tables provided. The selection of beam B1 with the aid of design tables is illustrated below:

Interior beam B1 – using design tables provided in Section 4.13

From Table 4.4, for **W410×60** with b_l = 2 430 mm,

$M_{rc50\%}$ = 509 kN·m > 433 kN·m
V_r = 558 kN > 151 kN OK
$Q_{r100\%}$ = 1 610 kN

For 50% shear connection,

Minimum number of studs $= 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\,610}{74.3}\right)$, (from Table 2.1, $q_r = 74.3$ kN)
$= 21.7$ i.e a minimum of 22 studs per beam
(**24 studs** per beam – detailed)

Since deck module is 406 mm, these studs can easily be accommodated on a one stud per rib basis, justifying their use as single-stud per rib connections (see Fig. 4.E12)

– Construction stages

$M_r' = 68.6$ kN·m > 51.0 kN·m (by interpolation - Table 4.4, $L' = 11\,500$ mm) OK for deck placement stage.
$M_r = 321$ kN·m > 231 kN·m (Table 4.4) OK for concrete placement stage.

– Deflection estimate

From Table 4.4
$d = 407$ mm $I_x = 216 \times 10^6\ mm^4$
$I_t = 730 \times 10^6\ mm^4$ $S_t = 1.74 \times 10^6\ mm^3$

a) Camber requirement

$$\Delta_c = \frac{5(92.5)(11.5)^3}{384(200)(216)} \times 10^3 = 42 \text{ mm}$$

Therefore **camber 40** mm at mid span.

b) Shrinkage deflection

$$\bar{e} = d + t_d + \frac{t_c}{2} - \frac{I_t}{S_t} = 407 + 76 + \frac{65}{2} - \frac{730}{1.74} = 96.0 \text{ mm}$$

$$\Delta_{sh} = \frac{\bar{e}\,\epsilon\,t_c\,b_l\,L^2}{8\,n\,I_t} = \frac{96.0(0.0002)(65)(2\,430)(11.5)^2}{8(9.43)(730)} = 7.3 \text{ mm}$$

c) Deflection due to live load plus partitions including long term effects

$$\Delta = \frac{5(W_L + W_p)L^3}{384\,E\,I_e}(1.15 \text{ for creep}) + \Delta_{sh}$$

$$= \frac{5(68.7 + 41.4)(11.5)^3\,10^3}{384(200)(592)}(1.15) + 7.3$$

$= 28.5$ mm $= L/404 < L/300$ OK

Note: value of $I_e = 592$ (x $10^6\ mm^4$), as shown before.

– Unshored beam requirement

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{132.9}{1.060} + \frac{193.1}{1.74} = 236 \text{ MPa} < 0.9F_y$$

Therefore shoring is not required.

W410×60 with 50% shear connection is satisfactory.

Interior beam B2

– Live load

$A = 3.0(9.0) = 27\ m^2$
$RF_2 = 0.3 + \sqrt{9.8/27} = 0.90$
$W_L = 0.90(2.4)(27) = 58.3$ kN

– Dead loads

$W_c = (7.44 + 0.4)(9.0) = 70.6$ kN ($w = 7.44$ kN/m, 0.4 kN/m steel beam assumed)
$W_p = 1.2(27) = 32.4$ kN
$W_{OD} = 0.7(27) = 18.9$ kN

– Factored total load, moment and shear

$W_f = 1.5(58.3) + 1.25(70.6 + 32.4 + 18.9) = 240$ kN
$M_f = 240(9.0)/8 = 270$ kN·m
$V_f = 240/2 = 120$ kN

Try **W410×39** $L/d = 9000/410 = 22 < 30$ OK for a trial section.
$L/4 = 9000/4 = 2\,250$ mm (Governs)
Beam spacing $= 3\,000$ mm
$16t_o + b = 2\,400$ mm
For $b_l = 2\,250$ mm, $M_{rc50\%} = 349$ kN·m (Table 4.4)
> 270 kN·m OK
$Q_{r100\%} = 1\,330$ kN (Table 4.4)

$2.5\,t = 2.5\,(8.8) = 22$ mm > 19 mm.
Therefore no reduction in q_r, ($q_r = 74.3$ kN)

Minimum number of studs $= 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\,330}{74.3}\right)$
$= 17.9$ i.e. **18 studs** per beam

(See stud layout, Fig. 4.E11; all single-stud per rib connections, therefore q_r OK)

$V_r = 448$ kN (Table 4.4)
> 120 kN OK

– Construction stage 1 - Deck placement

$A = 27\ m^2 > 16\ m^2$ Therefore consider U.D.L. only.
$q_L' = 0.5$ kPa (see Table 3.2)
$W_f' = [1.5(0.5) + 1.25(0.10)](27) + 1.25(0.4)(9.0)$ (0.4 kN/m steel beam assumed)
$= 28.1$ kN
$M_f' = 28.1(9.0)/8 = 31.6$ kN·m

Since no lateral support can be assumed, $L' = 9\ 000$ mm.
Hence,

$M_r' = 31.3$ kN·m (Table 4.4)
$\simeq 31.6$ kN·m say OK. (under designed by 1%)

– Construction stage 2 - Concrete placement

$q_L = 1.0$ kPa (see Table 3.2)
$W_f = 1.5(1.0)(27) + 1.25(70.6) = 129$ kN
$M_f = 129(9.0)/8 = 145$ kN·m
$M_r = 197$ kN·m (Table 4.4)
> 145 kN·m OK

– Deflection estimate

From Table 4.4,
$d = 399$ mm $\quad I_x = 127 \times 10^6\ mm^4$
$I_t \simeq 490 \times 10^6\ mm^4$ (by interpolation)
$S_t \simeq 1.14 \times 10^6\ mm^3$ (by interpolation)

a) Camber requirement

$$\Delta_c = \frac{5(70.6)(9.0)^3}{384(200)(127)} \times 10^3 = 26 \text{ mm}$$

Therefore **camber 25** mm at mid span.

b) Shrinkage deflection

$$\bar{e} = 399 + 76 + \frac{65}{2} - \frac{490}{1.14} = 77.7 \text{ mm}$$

$$\Delta_{sh} = \frac{77.7(0.0002)(65)(2\ 250)(9)^2}{8(9.43)490} = 5.0 \text{ mm} \qquad \text{(see Fig. 4.E5)}$$

c) Deflection due to live load plus partition including long term effects

$$\Delta = \frac{5(58.3+32.4)(9)^3\ 10^3}{384(200)\ I_e}(1.15) + 5.0$$

$$= 17.8 \text{ mm} = L/506 < L/300 \qquad \text{OK}$$

where $I_e = I_s + 0.85\ (p)^{0.25}\ (I_t - I_s)$
$= [127 + 0.85\ (0.5)^{0.25}\ (490 - 127)] \times 10^6$
$= 386\ (\times 10^6\ mm^4)$

– Unshored beam requirement

$M_b = 70.6(9.0)/8 = 79.4$ kN·m
$M_t = (58.3 + 32.4 + 18.9)(9.0)/8 = 123$ kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{79.4}{0.634} + \frac{123}{1.14} \qquad \text{(from Table 4.4, } S_x = 0.634 \times 10^6\ mm^3\text{)}$$

$$= 233 \text{ MPa} < 0.9\ F_y$$

Therefore, shoring is not required and W410×39 with 50% shear connection is satisfactory.

Spandrel Beam SB

– Design loads

$L = 11\ 500$ mm $\quad$ slab overhang = 250 mm
$A = (1.5+0.250)(11.5) = 20.1\ m^2$
$RF_2 \simeq 1.0$
$W_L = 2.4(20.1)(1.0) = 48.2$ kN
$W_c = [1.034(2.4)(1.75)+0.7](11.5) = 58.0$ kN (0.7 kN/m steel beam assumed)
$W_w = 3.8(11.5) = 43.7$ kN (spandrel wall load)
$W_p + W_{OD} = (1.2 + 0.7)(1.75)(11.5) = 38.2$ kN

– Factored total loads, moment and shear

$W_f = 1.5(48.2) + 1.25(58.0 + 43.7 + 38.2) = 247.2$ kN
$M_f = 247.2\ (11.5)/8 = 355$ kN·m
$V_f = 247.2/2 = 124$ kN

Try **W460×67** $\quad L/d = 11\ 500/460 = 25 < 30$ $\quad$ OK as a trial section
$b = 190$ mm (Table 4.4)

$0.10(11.5) = 1\ 150$ mm
$6(141) = 846$ mm (Governs)
Half clear spacing $= 0.5(3\ 000 - 95 - 89) = 1\ 408$ mm

$b_l = 155 + 190 + 846 = 1\ 190$ mm (say) (See Fig. 4.E6)

$M_{rc50\%} = 519$ kN·m (by interpolation from Table 4.4)
> 355 kN·m OK

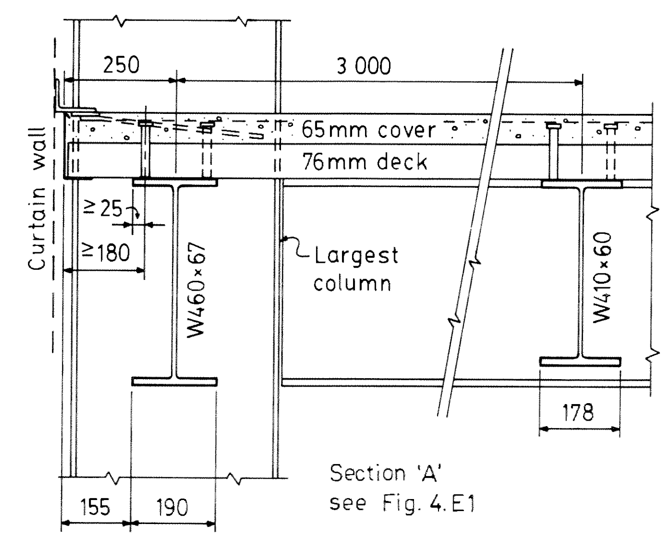


Figure 4.E6
Spandrel Beam Cross-Section and Slab Overhang Detail

$Q_{r100\%}$ = 789 kN (by interpolation from Table 4.4)

$2.5t = 2.5(12.7) = 31.8 \text{ mm} > 19 \text{ mm}$, and
edge distance $= 155+25 = 180 \text{ mm} > 141 \text{ mm}$.

Hence no reduction in stud capacity (from Fig. 2.4, $q_r = 74.3$ kN).

Minimum number of studs $= (2)\left(\frac{50\%}{100\%}\right)\left(\frac{789}{74.3}\right) = 10.6$ i.e. **12 studs** per beam (or 57% connection)

$V_r = 688$ kN (Table 4.4)
> 124 kN OK

– Construction stage 1 – Deck placement

$A = 20.1 \text{ m}^2 > 16 \text{ m}^2$ Therefore consider U.D.L. only.
$A = 20.1 \text{ m}^2 < 27 \text{ m}^2$
$q_L' = 0.5$ kPa (see table 3.2)
$W_f' = [1.5(0.5)+1.25(0.10)](20.1)+1.25(0.7)(11.5) = 27.7$ kN
$M_f' = 27.7\,(11.5)/8 = 39.8$ kN·m
No lateral support is assumed before deck is welded to beam, $L' = 11\,500$ mm
$M_r' = 87.2$ kN·m (by interpolation, Table 4.4)
> 39.8 kN·m OK

– Construction stage 2 – Concrete placement

$q_L = 1.0$ kPa
$W_f = 1.5(1.0)(20.1)+1.25(58.0) = 102.7$ kN
$M_f = 102.7(11.5)/8 = 148$ kN·m
$M_r = 405$ kN·m (Table 4.4)
> 148 kN·m OK

– Deflection estimate

From Table 4.4
$d = 454$ mm $I_x = 300 \times 10^6 \text{ mm}^4$
$I_t \simeq 776 \times 10^6 \text{ mm}^4$ (by interpolation)
$S_t \simeq 1.99 \times 10^6 \text{ mm}^3$ (by interpolation)

a) Camber requirement

$$\Delta_c = \frac{5(58.0)(11.5)^3}{384(200)(300)} \times 10^3 = 19 \text{ mm}$$

Camber 20 mm at mid span.

b) Instantaneous deflection due to cladding mass

$$\Delta_w = \frac{5(43.7)(11.5)^3}{384(200)(776)} \times 10^3 \,(1 + 15\% \text{ for deck profile} + 15\% \text{ for partial connection})$$

$= 7.2 \text{ mm} < 12 \text{ mm}$ OK

c) Shrinkage deflection

$$\bar{e} = 454 + 76 + \frac{65}{2} - \frac{776}{1.99} = 173 \text{ mm}$$

$$\Delta_{sh} = \frac{173(0.0002)(65)(1\,190)(11.5)^2}{8(9.43)(776)} = 6.0 \text{ mm} \quad \text{(see Fig. 4.E5)}$$

d) Deflection due to all superimposed loads including long term effects

$$\Delta = \frac{5(48.2+43.7+38.2)(11.5)^3\,10^3}{384(200)\,I_e}(1.15) + 6.0$$
$$= 28.7 \text{ mm} = L/400 < L/360 \quad \text{OK}$$

where $I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$
$= [300 + 0.85\,(0.57)^{0.25}\,(776 - 300)]\,10^6$
$= 652\,(\times 10^6 \text{ mm}^4)$

– Unshored beam requirement

$S_x = 1.32 \times 10^6 \text{ mm}^3$ (Table 4.4)
$M_b = 58.0(11.5)/8 = 83.4$ kN·m
$M_t = (48.2+43.7+38.2)(11.5)/8 = 187$ kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{83.4}{1.32} + \frac{187}{1.99} = 157 \text{ MPa} < 0.9\,F_y$$

Therefore shoring is not required.

W460×67 with 50% shear connection is satisfactory.

Interior Girder G

– Live Load

$A \simeq 2(3.0)(11.5+9.0)/2 = 61.5 \text{ m}^2$*
$RF_2 = 0.3 + \sqrt{9.8/61.5} = 0.70$
$P_{L1} = 0.70(2.4)(31.2) = 52.4$ kN
$P_{L2} = 0.70(2.4)(17.3) = 29.1$ kN

*For an asymmetrically loaded girder, the tributary area for live load reduction seems less well defined. A conservative approach is adopted here.

– Dead Loads

$P_{c1} \simeq (92.5 + 70.6)/2 + 1.034(2.4)(0.10)(4.5) = 82.7$ kN
$P_{c2} \simeq 92.5/2 = 46.2$ kN
$W_c = 0.8(9.0) = 7.2$ kN (0.8 kN/m steel girder assumed)
$P_{p1} \simeq (41.4 + 32.4)/2 + 1.2(0.10)(4.5) = 37.4$ kN
$P_{p2} = 41.4/2 = 20.7$ kN
$P_{OD1} \simeq (24.2 + 18.9)/2 + 0.7(0.10)(4.5) = 21.9$ kN
$P_{OD2} = 24.2/2 = 12.1$ kN

– Factored total loads, moment and shear

$$
\begin{aligned}
P_{f1} &= 1.5(52.4) + 1.25(82.7 + 37.4 + 21.9) = 256 \text{ kN}\\
P_{f2} &= 1.5(29.1) + 1.25(46.2 + 20.7 + 12.1) = 142 \text{ kN}\\
W_f &= 1.25(7.2) = 9.0 \text{ kN}\\
V_f &= [(9.0)P_{f2} + (3.0+6.0)P_{f1} + 4.6W_f]/9.2\\
&= [(9.0)(142) + (9.0)(256) + 4.6(9.0)]/9.2 = 394 \text{ kN}\\
M_f &= 3.2(394) - 3.0(142) - (3.2)^2(1.25)(0.8)/2 = 830 \text{ kN·m}
\end{aligned}
$$

Try W530×92

$16t_o + b$	= 2 470 mm (Table 4.4)
Average girder spacing	= 10 250 mm
$L/4 = 9\,200/4$	= 2 300 mm (Governs b_1)

From Table 4.4, for b_1 = 2 300 mm,

$M_{rc50\%}$ = 880 kN·m (by interpolation)
> 830 kN·m OK
V_r = 969 kN > 394 kN OK
$Q_{r100\%}$ = 1 530 kN (by interpolation)

2.5 t = 2.5(15.6) = 39 mm > 19 mm (19 mm studs used)
Therefore, no reduction in shear stud capacity

i.e. q_r = 74.3 kN (Table 2.1)

$$\text{Minimum number of studs} = 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\,530}{74.3}\right) = 20.6$$

Use no less than **22 studs** per girder or 11 studs (53% connection) on each side of point of maximum moment (see Fig. 4.E7). The studs required in shear span 2 (11 studs) can be uniformly distributed whereas the stud distribution between the point of maximum moment and the end support at the column depends on the number of studs required in shear span 1 (and other factors that will be dealt with later under steel deck and shear stud layouts).

Number of studs required in shear span 1,

$$
\begin{aligned}
n' &= n(M_{f1} - M_r)/(M_f - M_r) && \text{(Eq. 2.10)}\\
&= \frac{20.6}{2}\left(\frac{803 - 637}{830 - 637}\right) && (M_r = 637 \text{ kN·m, Table 4.4})\\
&= 8.9 \quad \text{i.e. 9 studs required}
\end{aligned}
$$

Therefore at least 2 studs are required between two point loads, P_{f1}.

– Construction stage

$$
\begin{aligned}
A &= 61.5 \text{ m}^2 > 54 \text{ m}^2 \qquad \text{i.e. } q_L = 0.6 \text{ kPa} \quad \text{(Table 3.2)}\\
P_{f1} &= 1.5(0.6)[3.0(5.75)+3.1(4.5)] + 1.25(82.7) = 131 \text{ kN}\\
P_{f2} &= 1.5(0.6)(3.0)(5.75) + 1.25(46.2) = 73.3 \text{ kN}
\end{aligned}
$$

$$M_f = [9.0(P_{f1} + P_{f2}) + 4.6\,W_f]\frac{3.2}{9.2} - 3.0\,P_{f2} - (3.2)^2\,(1.25)(0.8)/2$$

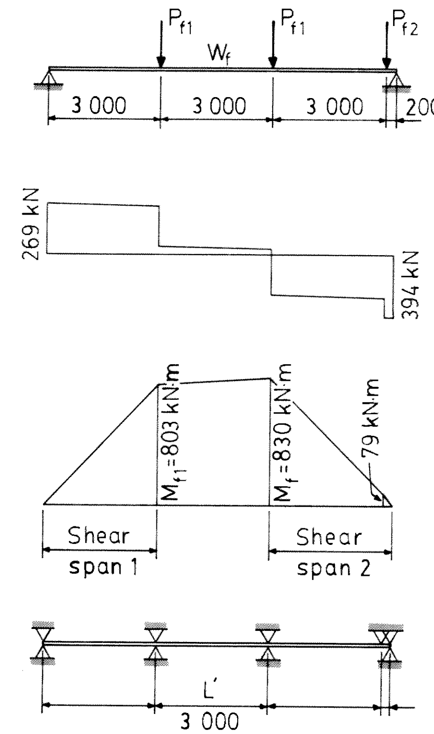


Figure 4.E7
Shear and Moment Diagrams of Floor Girder 'G'

$$= [9.0(131+73.3) + 4.6(9.0)]\,\frac{3.2}{9.2} - 3.0\,(73.3) - 5.12$$

$$= 429 \text{ kN·m}$$

For L' = 3 000 mm, M_r = 633 kN·m (Table 4.4)
> 429 kN·m OK

– Deflection estimate

From Table 4.4, d	= 533 mm	$I_x = 552 \times 10^6$ mm⁴
I_t	= $1\,510 \times 10^6$ mm⁴	(by interpolation)
S_t	= 3.12×10^6 mm⁴	(by interpolation)

a) Camber requirement

$$\Delta_c \simeq \frac{L^3}{48EI_x}\left\{\Sigma\left[P\left(\frac{3a}{L} - \frac{4a^3}{L^3}\right)\right] + \frac{5}{8}\,W_c\right\}$$

where a = distance from each load point to the nearest support

$$\Delta_c \simeq \frac{L^3}{48EI_x}\left\{P_{c1}\left[\frac{3(3.0)}{L} - \frac{4(3.0)^3}{L^3}\right] + P_{c1}\left[\frac{3(3.2)}{L} - \frac{4(3.2)^3}{L^3}\right]\right.$$

$$\left. + P_{c2}\left[\frac{3(0.2)}{L} - \frac{4(0.2)^3}{L^3}\right] + \frac{5}{8}W_c\right\}$$

$$= \frac{(9.2)^3 \times 10^3}{48(200)(552)}\left\{82.7(1.715) + 46.2(0.0652) + 0.625(7.2)\right\} = 22 \text{ mm}$$

Camber 20 mm at mid span.

b) Shrinkage deflection

$$\bar{e} = 533 + 76 + \frac{65}{2} - \frac{1\,510}{3.12} = 158 \text{ mm}$$

$$\Delta_{sh} = \frac{158(0.0002)(65)(2\,300)(9.2)^2}{8(9.43)(1\,510)} = 3.5 \text{ mm}$$

c) Deflection due to live load plus partition including long term effects

$$\Delta = \frac{L^3}{48EI_e}\left\{(P_{L1}+P_{p1})(1.715)+(P_{L2}+P_{p2})(0.0652)\right\}(1.15) + \Delta_{sh}$$

$$= \frac{(9.2)^3(10^3)}{48(200)\,I_e}\left\{(52.4+37.4)(1.715)+(29.1+20.7)(0.0652)\right\}(1.15)+3.5$$

$$= 15.3 \text{ mm} = L/601 < L/300 \qquad \text{OK}$$

where $I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$

$$= \left[552 + 0.85\,(0.53)^{0.25}\,(1\,510 - 552)\right] 10^6$$

$$= 1\,247\ (\times 10^6 \text{ mm}^4)$$

– Unshored girder requirement

$$M_b = \left[9.0(P_{c1}+P_{c2})+4.6W_c\right]\frac{3.2}{9.2} - 3.0\,P_{c2} - (3.2)^2(0.8)/2$$

$$= \left[9.0(82.7+46.2)+4.6(7.2)\right]\frac{3.2}{9.2} - 3.0(46.2) - 4.10 = 272 \text{ kN·m}$$

$$M_t = 9.0\,(P_{L1}+P_{L2}+P_{p1}+P_{p2}+P_{OD1}+P_{OD2})\,\frac{3.2}{9.2} - 3.0(P_{L2}+P_{p2}+P_{OD2})$$

$$= 9.0(52.4+29.1+37.4+20.7+21.9+12.1)\,\frac{3.2}{9.2} - 3.0(29.1+20.7+12.1)$$

$$= 358 \text{ kN·m}$$

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{272}{2.07} + \frac{358}{3.12} = 246 \text{ MPa} < 0.9\,F_y \quad (S_x = 2.07 \times 10^6 \text{ mm}^3\text{, Table 4.4})$$

Therefore shoring is not required.

Girder's longitudinal shear requirement will be dealt with after steel deck layout is planned.

Spandrel Girder SG

– Live loads

$A = 6.0(11.5)/2 + 9.0(0.25) = 36.8 \text{ m}^2$
$RF_2 = 0.3 + \sqrt{9.8/36.8} = 0.82$
$P_L = 0.82(2.4)(3.0)(11.5)/2 = 34.0 \text{ kN}$
$W_L = 0.82(2.4)(9.0)(0.25) = 4.43 \text{ kN}$

– Dead loads

Assuming 0.7 kN/m due to steel girder and 3.5 kPa due to slab overhang

$W_c = 3.5(9.0)(0.25)+0.7(9.0) = 14.2 \text{ kN}$
$P_c = 92.5/2 = 46.3 \text{ kN}$
$W_w = 3.8(9.0) = 34.2 \text{ kN}$
$W_{OD}+W_p = (1.2+0.7)(9.0)(0.25) = 4.28 \text{ kN}$
$P_{OD}+P_p = (41.4+24.2)/2 = 32.8 \text{ kN}$

– Factored total load, moment and shear

$P_f = 1.5(34.0)+1.25(46.3+32.8) = 150 \text{ kN}$
$W_f = 1.5(4.43)+1.25(14.2+34.2+4.28) = 72.5 \text{ kN}$
$M_f = 150(9.0)/3+72.5(9.0)/8 = 532 \text{ kN·m}$
$V_f = 150+72.5/2 = 186 \text{ kN}$

Try **W460×74**
$b = 190$ mm (Table 4.4)
$6(141) = 846$ mm (Governs slab projection to the interior floor from edge of beam flange)

$0.10(9\,000) = 900$ mm
Half clear spacing of girders > 846 mm (by inspection)
$b_l = 846 + 190/2 + 250 = 1\,190$ mm (250 mm = exterior slab overhang from centre of beam; see Fig. 4.E8)

From Table 4.4, $M_{rc50\%} = 565$ kN·m (by interpolation)
> 532 kN·m OK
$V_r = 733 \text{ kN} > 186 \text{ kN}$ OK
$Q_{r100\%} = 789$ kN (by interpolation)
$2.5t = 2.5(14.5) = 36 \text{ mm} > 19$ mm; edge distance $= 180 \text{ mm} > 141$ mm.
Therefore, no reduction in shear stud capacity, i.e. $q_r = 74.3$ kN (Table 2.1)

Minimum number of studs $= 2\left(\frac{50\%}{100\%}\right)\left(\frac{789}{74.3}\right) = 10.6$ i.e **12 studs** per girder 57% connection)

– Construction stage

$A = 36.8 \text{ m}^2 > 27 \text{ m}^2$
$q_L = 1.4 - A/67.5 = 0.85 \text{ kPa}$ (see Table 3.2)

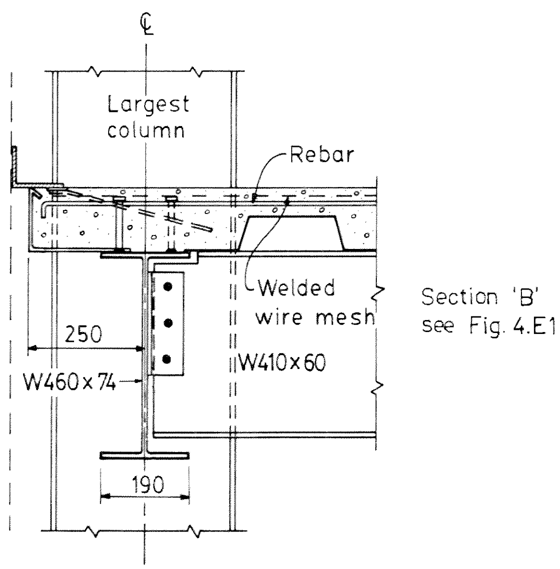


Figure 4.E8
Spandrel Girder 'SG' Cross-Section

$P_f = 1.5(0.85)(3.0)(11.5)/2 + 1.25(46.3) \quad = 79.9$ kN
$W_f = 1.5(0.85)(9.0)(0.25) + 1.25(14.2) \quad = 20.6$ kN
$M_f = 79.9(9.0)/3 + 20.6(9.0)/8 \quad = 263$ kN·m

For $L' = 3\,000$ mm, $M_r = 433$ kN·m (Table 4.4)
> 263 kN·m OK

– Deflection estimate

From Table 4.4
$d = 457$ mm $\quad I_x = 333 \times 10^6$ mm^4
$I_t = 833 \times 10^6$ mm^4 (by interpolation)
$S_t = 2.16 \times 10^6$ mm^3 (by interpolation)

a) Camber requirement

$$\Delta_c = \frac{23(46.3)(9.0)^3}{648(200)(333)} \times 10^3 + \frac{5(14.2)(9.0)^3}{384(200)(333)} \times 10^3 = 20 \text{ mm}$$

Camber 20 mm at mid span.

b) Instantaneous deflection due to cladding mass

$$\Delta_w = \frac{5(34.2)(9.0)^3}{384(200)(833)} \times 10^3 \; (1.0 + 15\% \text{ for partial connection})$$

$= 2.2$ mm < 12 mm OK

c) Shrinkage deflection

$$\bar{e} = 457 + 76 + \frac{65}{2} - \frac{833}{2.16} = 180 \text{ mm}$$

$$\Delta_{sh} = \frac{180(0.0002)(65)(1\,190)(9.0)^2}{8(9.43)(833)} = 3.6 \text{ mm}$$

d) Deflection due to all superimposed load including long term effects

$$\Delta = \left[\frac{23(34.0+32.8)(9.0)^3\;10^3}{648(200)\;I_e} + \frac{5(4.43+34.2+4.28)(9.0)^3\;10^3}{384(200)\;I_e}\right](1.15) + 3.6$$

$= 21.1$ mm $= L/426 < L/360$ OK

where $I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$

$= \left[333 + 0.85\,(0.57)^{0.25}\,(833 - 333)\right] 10^6$ mm^4

$= 702\ (\times 10^6$ mm$^4)$

– Unshored girder requirement

$M_b = 46.3(9.0)/3 + 14.2(9.0)/8 = 155$ kN·m
$M_t = (34.0 + 32.8)(9.0)/3 + (4.43+34.2+4.28)(9.0)/8 = 249$ kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{155}{1.46} + \frac{249}{2.16} \qquad (S_x = 1.46 \times 10^6 \text{ mm}^4, \quad \text{Table 4.4})$$

$= 221$ MPa $< 0.9\,F_y$

Therefore shoring is not required.

W460×74 with 50% shear connection is satisfactory.

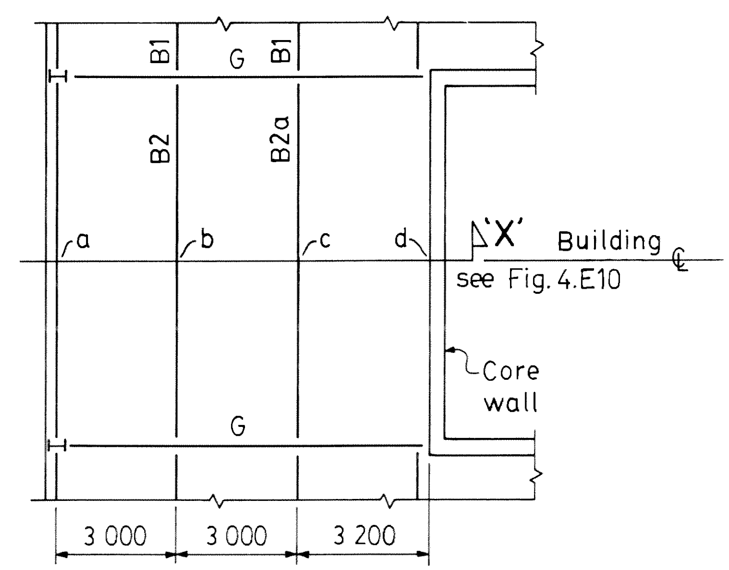


Figure 4.E9
Layout of Floor Bay at End of Service Core

Slab overhang

Select cold formed screed flash (bent plate) from data given in Fig. 4.18. For length of slab overhang, $x = 155$ mm and $t_o = 140$ mm, screed flash of 2.67 mm nominal thickness will limit tip deflection to less than 3 mm (see Fig. 4.18 for welding requirement). Negative reinforcing bars are required for cantilever action in the direction transverse to girder (see Fig. 4.E8).

Beam B2a (floor topography)

Since beam B2a is parallel and adjacent to a core wall, its mid-span deflection with respect to the floor elevation at the core wall, a rigid support, must be investigated. In this case deflection limitation may govern the design of beam B2a including the amount of camber required. The final elevation of the beam's mid-span (identified as 'c' in Fig. 4.E9) is a function of the simply supported beam deflection plus the girder deflection at the beam supports whereas the floor elevation at the core wall (identified as 'd' in Fig. 4.E10) remains virtually unchanged. The variation of support stiffnesses at 'c' and 'd' illustrates the fact that the true tributary area on beam B2a is less than the 'tributary' area assumed (based on half the deck span at either side of beam B2a); thus, actual total deflection at 'c' (beam B2a) should be less than that shown in Fig. 4.E10.

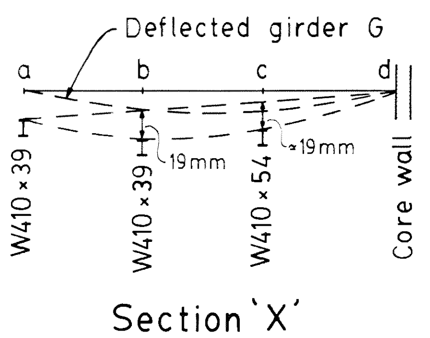


Figure 4.E10
Final Elevation of Deck-Slab with respect to Support Elevation at Core Wall

Steel deck layout and shear stud distribution (wide-rib deck)

Steel deck layout is an important part of hollow-composite construction. A proper layout reduces wastage, cutting and erection time. The actual shear stud quantity also depends on steel deck layout. The minimum number of shear studs required for each composite member that was determined previously was computed based on the assumption that no more than one stud per deck flute is allowed. Quite often some flutes must accommodate more than one shear stud due to a limited number of wide flutes available per beam span. A steel deck layout and shear stud distribution plan is shown in Figs. 4.E11 and 4.E12. This layout allows one stud per flute for all beams except for **beam B2a**. In the case of beam B2a,

$$Q_{r50\%} = \frac{1\,490}{2} = 745 \text{ kN}$$

$$\text{Minimum number of single studs} = 2\left(\frac{745}{74.3}\right) = 20.1$$

i.e. 11 single studs per half span, but number of flutes available per half span = 10. Try 9 single groups plus 1 double stud group per half span.

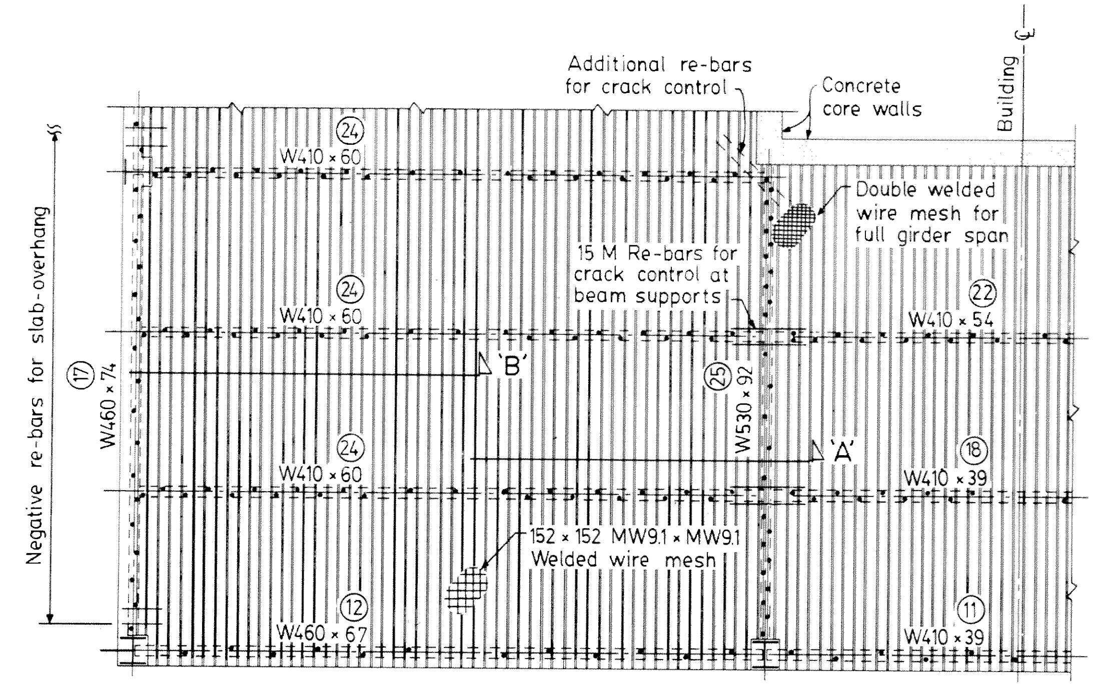


Figure 4.E11
Steel Deck and Shear Stud Layout (Non-Cellular Configuration)

$$q_{rN} = \frac{0.85}{\sqrt{N}}\left(\frac{H - t_d}{t_d}\right)\left(\frac{W_{rib}}{t_d}\right) q_r \qquad \text{(Eq. 2.10)}$$

$$q_{r2} = \frac{0.85}{\sqrt{2}}\left(\frac{120-76}{76}\right)\left(\frac{180}{76}\right)(74.3) = 61.2 \text{ kN per stud}$$

$$\begin{aligned} Q_r &= \Sigma\, q_r = 9(74.3) + (61.2)(2) \\ &= 791 \text{ kN} > 745 \text{ kN} \qquad \text{OK} \end{aligned}$$

Alternatively B2a may be designed as non-composite member for strength consideration.
$M_r = 283$ kN·m $> M_f = 270$ kN·m
For deflection, no less than 25% connection is used to create composite action (use 12 studs per beam)

Vertical separation and mechanical ties

Shear stud distribution and weld pattern along beam B1 are illustrated in Fig. 4.E12. The deck is welded to each steel member before shear studs are installed. These arc spot welds also serve to prevent vertical separation between the deck and the steel shape after concrete is cast while the composite steel deck holds the concrete by means of its embossment. The weld spacing must also satisfy the fire-resistance rating requirements of the floor assembly.

When a deck flute is not aligned over a girder, the deck is cut out to admit shear studs (see details in Figures 4.E8 and 4.E13). In the absence of steel deck additional shear studs are provided near the girder mid-span to serve as mechanical ties. Average spacing of mechanical ties should not exceed 600 mm in accordance with Clause 17.3.7 of S16.1.

Owing to the presence of either a stiffener (embossment) or a side-lap joint in the middle of each flute, each stud must be installed asymmetrically to the flute and should be placed on the side closer to the nearest beam support whenever possible (see Section 2.4).

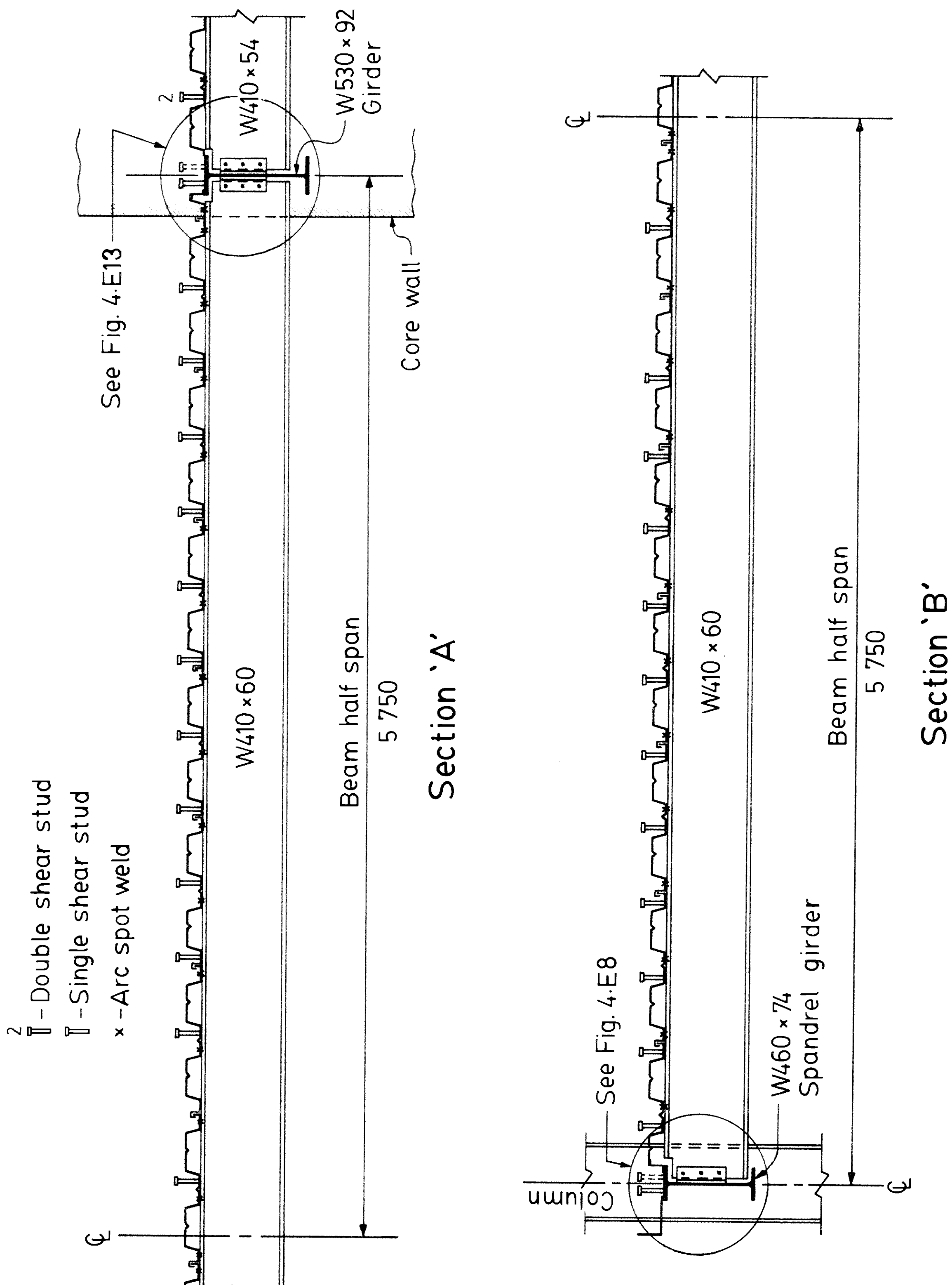


Figure 4.E12
Detailed Sections of Deck and Shear Stud Layout (Non-Cellular Configuration)

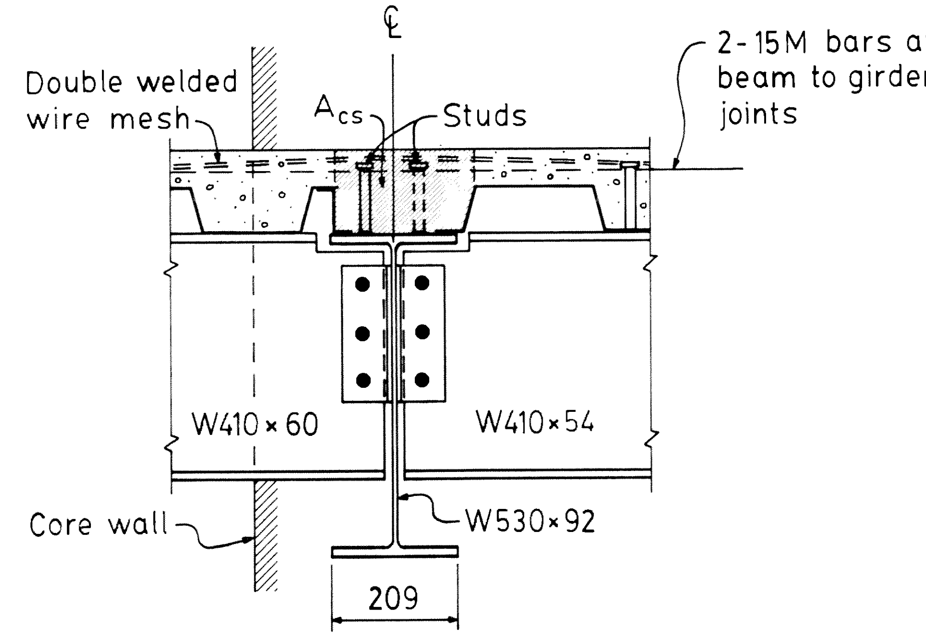


Figure 4.E13
Interior Girder 'G' Cross-Section

Longitudinal shear in composite girders

a) Interior girder G: Check if **two** layers of 152×152 MW9.1×MW9.1 welded wire mesh satisfy the design criterion outlined in Section 4.9. Since shear studs are more closely spaced in shear span 2 (l_{sh} = 3 200 mm), shear span 2 governs.

$$Q_r = 1\,530/2 = 765 \text{ kN}$$

$$\rho = \frac{(9.0)}{152(65)} = 0.00091 \text{ (credit 1 layer of mesh for longitudinal shear resistance)}$$

$$\begin{aligned} v_u &= 0.8\,\rho\,f_y + 2.76 && \text{(Eq. 4.20)} \\ &= 0.8\,(0.00091)(400) + 2.76 && (f_y = 400 \text{ MPa}) \\ &= 3.05 \text{ MPa} < 0.3\,f_c' && (0.3\,f_c' = 6.0 \text{ MPa}) \\ V_u &= 2\,l_{sh}\,t_c\,v_u && \text{(Eq. 4.19)} \\ &= 2(3\,200)(65)(3.05) \times 10^{-3} = 1\,270 \text{ kN} \\ A_{cs} &= 209(140) = 29\,300 \text{ mm}^2 && \text{(see Fig. 4.E15)} \end{aligned}$$

$$\begin{aligned} \phi_v(V_u + 0.85\,f_c'\,A_{cs}) &= 0.60[1\,270 + 0.85(0.020)(29\,300)] \quad \text{(Eq. 4.23)} \\ &= 1\,060 \text{ kN} > 765 \text{ kN} \quad \text{OK} \end{aligned}$$

b) Spandrel girder SG: Resistance to longitudinal shear is found adequate.

Cellular Floor

Figures 4.E14 and 15 illustrate a layout of a cellular floor system. Typical cross-sectional and pictorial views of cellular steel deck and trench header duct are also shown in Fig. 1.4. The presence of these underfloor distribution features have affected the structural design in several major areas:

a) The presence of cellular decks may reduce the number of wide-ribs per beam, depending on the deck manufacturer, and hence may result in double stud application on some interior beams.

b) The trench header duct truncates the effective slab width of the adjacent beam. As a result, a heavier steel beam section may become necessary, but requiring fewer shear studs.

c) The trench duct also crosses over the main girder and reduces the girder's composite shear span. This necessitates closer spacing of shear studs and additional longitudinal shear reinforcement. The proximity of the trench duct to the end of the girder normally permits the girder to resist bending and shear forces as a non-composite section.

d) Since the trench duct displaces concrete over the steel deck, the steel deck alone must resist the total bending moment and shear at the trench duct location as a non-composite section. A thicker deck may have to be provided locally (see Figs. 4.E14 and 15).

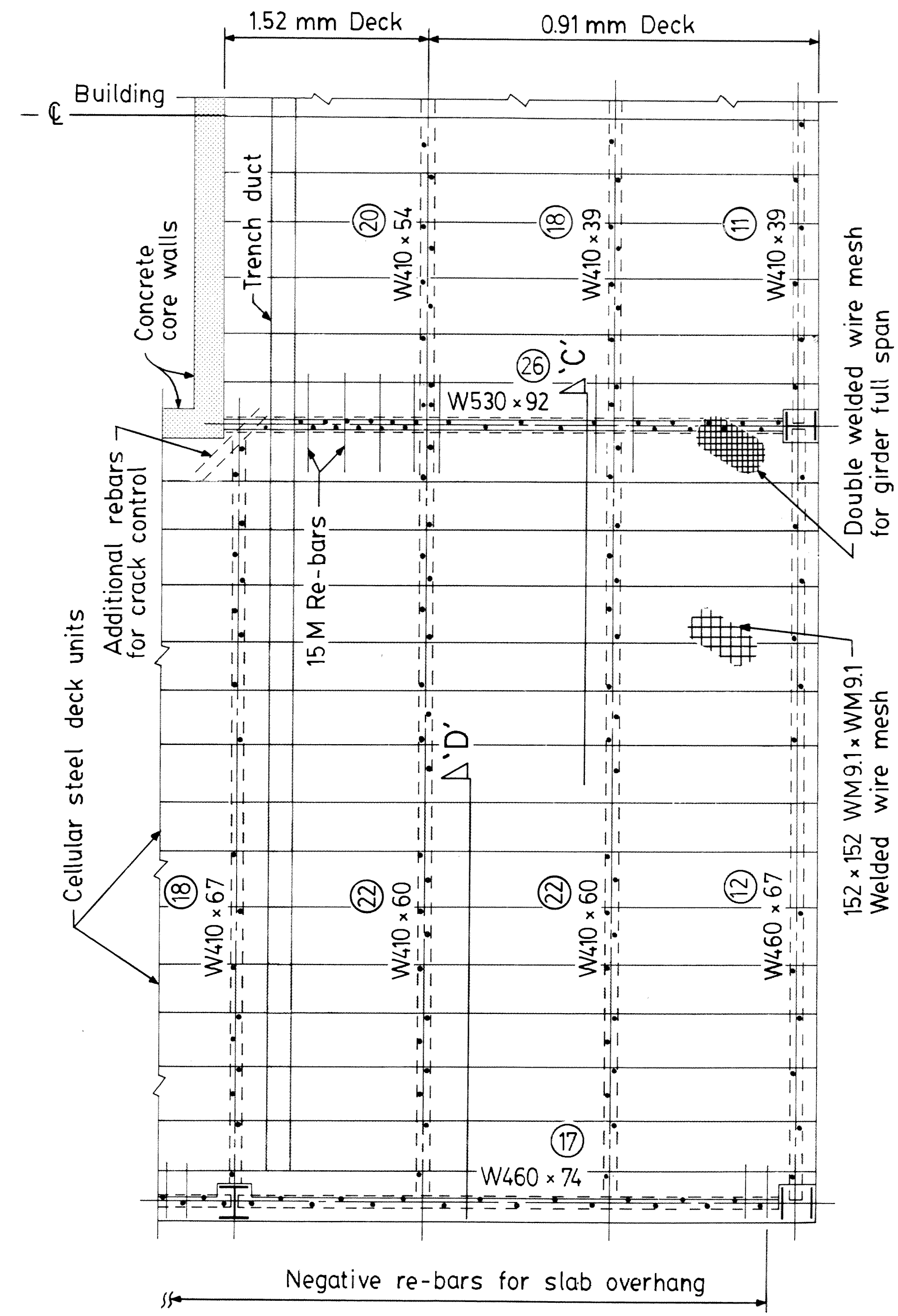


Figure 4.E14
Steel Deck and Shear Stud Layout
(Cellular Configuration)

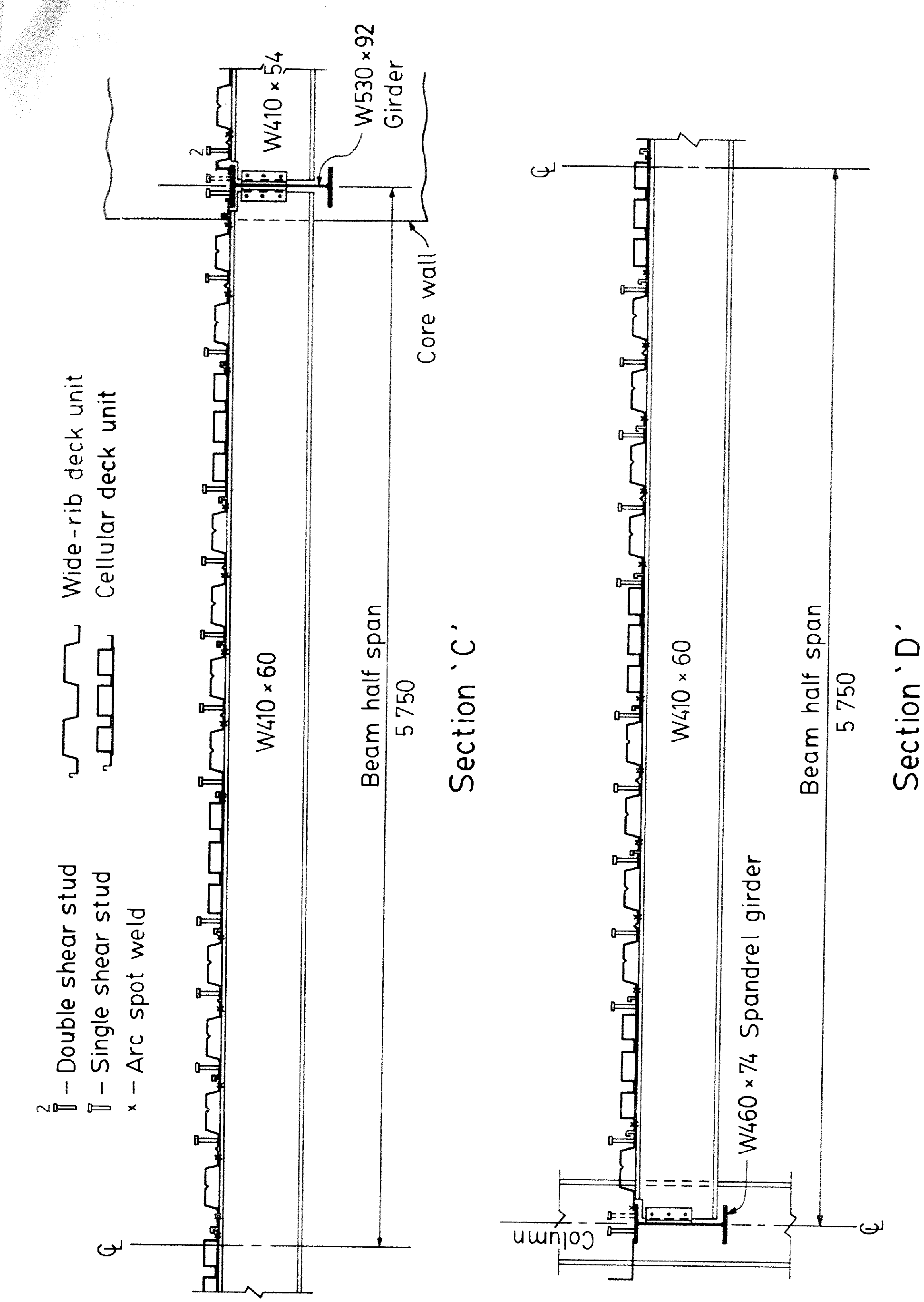


Figure 4.E15
Detailed Sections of Deck and Shear Stud Layout (Cellular Configuration)

REFERENCES

4.1 MacKay, H.M., Gillespie, P., and Leluau, C., "Report on the Strength of Steel I-Beams Haunched with Concrete", the Engineering Journal, Engineering Institute of Canada, Vol. 6, No. 8 1923.

4.2 Viest, I.M., "Review of Research on Composite Steel-Concrete Beams", Journal of Structural Division, ASCE, June 1960.

4.3 David, R., and Meyerhof, G.G., "Composite Construction of Bridges using Steel and Concrete", The Engineering Journal, Engineering Institute of Canada, May 1958.

4.4 Viest, I.M., Fountain, R.S., Singleton, R.C., "Composite Construction in Steel and Concrete", McGraw-Hill, 1958.

4.5 AASHO, "Standard Specifications for Highway Bridges", Washington, D.C., 1944.

4.6 "Inland Hi-Bond Composite Beam Design – Design Manual for Building Construction", 1964.

4.7 "Cofar Composite Design", Granco Steel Products Co., March 1965.

4.8 "Composite Floor Systems", Fenestra Inc., 1966.

4.9 "Composite Beam Manual", Canadian Sheet Steel Building Institute, 1968.

4.10 Timoshenko, S.P., and Goodier, J.N., "Theory of Elasticity", McGraw-Hill, New York, 1951.

4.11 Heins, C.P., Fan, H.M., "Effective Composite Beam Width at Ultimate Load", Journal of the Structural Division, ASCE, Nov. 1976.

4.12 Fisher, J.W., "Design of Composite Beams with Formed Metal Deck", AISC Engineering Journal, July 1970.

4.13 "National Building Code of Canada 1985", NRCC No. 17303.

4.14 "Steel Structures for Buildings – Limit States Design", CSA CAN3-S16.1-M84.

4.15 Robinson, H., and Wallace, I.W., "Composite Beams with 1½ Inch Metal Deck and Partial and Full Shear Connection", Transactions, CSCE, Sept. 1973.

4.16 "Standard for Steel Floor Deck", Canadian Sheet Steel Building Institute, October 1976.

4.17 SABNIS, G.M., et al, "Handbook of Composite Construction Engineering", Van Nostrand Reinhold Company, 1979.

4.18 Tall, L., et al, "Structural Steel Design", Roland Press Company, 1964.

4.19 "Design and Control of Concrete Mixtures", Eleventh Edition, Portland Cement Association, July 1968.

4.20 Park, P., and Paulay, T., "Reinforced Concrete Structures", John Wiley and Son, New York, 1975.

4.21 Johnson, R.P., and Allison, R.W., "Shrinkage and Tension Stiffening in Negative Moment Regions of Composite Beams", The Structural Engineer, March 1981.

4.22 Montgomery, C.J., Kulak, G.L., and Shwartsburd, G., "Deflection of a Composite Floor System", Canadian Journal of Civil Engineering Vol.10, No. 2, June 1983.

4.23 Deflection measurements of hollow composite beam under drying shrinkage, (Full-scale beam tests carried out in 1983), Dr. H. Robinson, McMaster University, Hamilton.

4.24 "Tentative Recommendations for the Design and Construction of Composite Beams and Girders for Buildings", ASCE-ACI Committee on Composite Construction, Journal of the Structural Division, December 1960.

4.25 Johnson, R.P., and Oehlery, D.J., "Analysis and Design for Longitudinal Shear in Composite T-Beams", Institution of Civil Engineers, Proceedings, December 1981.

4.26 Davies, C., "Tests on Half-Scale Steel-Concrete Composite Beams with Welded Stud Connectors", The Structural Engineer, January 1969.

4.27 Johnson, R.P., "Longitudinal Shear Strength of Composite Beams", ACI Journal, June 1970.

4.28 El-Ghazzi, M.N., "Longitudinal Shear Capacity of the Slabs of Composite Beams", Dept. of Civil Engineering and Engineering Mechanics, McMaster University, Nov. 1972.

4.29 Azmi, H.H., "Strength of Composite Beams Incorporating 3 Inch Cellular Metal Deck", Dept. of Civil Engineering and Engineering Mechanics, McMaster University, June 1975.

4.30 Robinson, H., "Resistance to Longitudinal Cracking in Composite Girders", McMaster University, Report No. 81-1.

4.31 Buckner, C.D., Deville, D.J., McKee, D.C., "Shear Strength of Slabs in Stub-Girders", Journal of the Structural Division, ASCE, February 1981.

4.32 Mattock, A.H., "Shear Transfer in Concrete having Reinforcement at an Angle to the Shear Plane", Special Publication 42, Shear in Reinforced Concrete, American Concrete Institute, 1974.

4.33 Mattock, A.H., Li, W.K., and Wang, T.C., "Shear Transfer in Lightweight Reinforced Concrete", PCI Journal, Vol. 21, No.1.

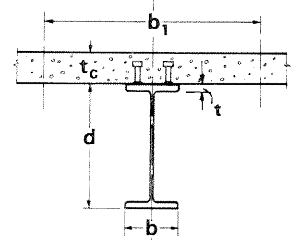
4.34 Ritchie, J.K., Chien, E.Y.L., "Composite Structural Systems – Design, Construction and Cost Considerations", Canadian Structural Engineering Conference Proceedings, 1980.

4.35 Redwood, R.G., and Wong, P.K., "Web Holes in Composite Beams with Steel Deck", Canadian Structural Engineering Conference Proceedings, 1982.

4.36 Redwood, R.G., and Shrivastava, S.C., "Design Recommendations for Steel Beams with Web Holes", Canadian Journal of Civil Engineering, Dec. 1980.

4.37 McGuire, R.K., Cornell, C.A., "Live Load Effects in Office Buildings", Journal of the Structural Division, ASCE, July 1974.

NOTES



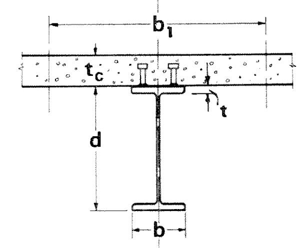
300W
20 MPa

130 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	b_1 (mm)	Composite Beam*: Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
WWF700X151	2 380	2 010	1 980	1 930	3 160	3 870	6 330	M_r 1 480	6 000	1 330	14 000	485
	2 040	1 970	1 950	1 890	2 710	3 740	6 280	V_r 846	9 000	942	16 000	404
b=300	1 690	1 930	1 910	1 850	2 240	3 590	6 210	L_u 4 500	10 000	797	18 000	347
t=22.0	1 350	1 900	1 870	1 790	1 790	3 390	6 120	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 850	1 800	1 720	1 330	3 140	6 000	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 380	1 910	1 880	1 830	3 160	3 680	5 950	M_r 1 380	6 000	1 220	14 000	420
	2 040	1 870	1 850	1 800	2 710	3 560	5 900	V_r 846	9 000	831	16 000	348
b=300	1 690	1 830	1 820	1 750	2 240	3 410	5 840	L_u 4 420	10 000	700	18 000	297
t=20.0	1 350	1 800	1 770	1 690	1 790	3 230	5 760	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 750	1 700	1 620	1 330	2 990	5 640	S_x 4 620	12 000	527	22 000	230
W610X155	2 400	1 820	1 790	1 730	3 180	3 030	5 640	M_r 1 280	6 000	1 180	13 000	516
W24X104	2 030	1 780	1 760	1 680	2 690	2 920	5 580	V_r 1 380	9 000	866	14 000	465
b=324	1 650	1 740	1 710	1 620	2 190	2 770	5 500	L_u 4 740	10 000	762	16 000	388
t=19.0	1 280	1 690	1 650	1 560	1 700	2 590	5 390	I_x 1 290	11 000	659	18 000	333
d=611	900	1 610	1 560	1 480	1 190	2 350	5 240	S_x 4 220	12 000	579	20 000	291
W610X140	2 310	1 680	1 650	1 580	3 060	2 760	5 050	M_r 1 120	5 000	946	11 000	373
W24X94	1 960	1 640	1 610	1 530	2 600	2 660	4 990	V_r 1 440	6 000	829	13 000	303
b=230	1 610	1 600	1 560	1 470	2 130	2 530	4 920	L_u 3 320	7 000	695	15 000	255
t=22.2	1 250	1 550	1 490	1 400	1 660	2 370	4 820	I_x 1 120	8 000	573	17 000	221
d=617	900	1 460	1 410	1 330	1 190	2 160	4 680	S_x 3 630	9 000	486	19 000	195
W610X125	2 310	1 510	1 480	1 420	3 060	2 490	4 500	M_r 991	5 000	821	11 000	301
W24X84	1 960	1 470	1 450	1 380	2 600	2 400	4 450	V_r 1 300	6 000	708	13 000	243
b=229	1 600	1 430	1 410	1 330	2 120	2 290	4 390	L_u 3 250	7 000	575	15 000	204
t=19.6	1 250	1 390	1 350	1 270	1 660	2 150	4 300	I_x 985	8 000	470	17 000	176
d=612	900	1 320	1 270	1 200	1 190	1 960	4 190	S_x 3 220	9 000	396	19 000	155
W610X113	2 310	1 380	1 360	1 300	3 060	2 270	4 070	M_r 888	5 000	719	11 000	247
W24X76	1 960	1 340	1 330	1 260	2 600	2 190	4 020	V_r 1 210	6 000	610	13 000	198
b=228	1 600	1 310	1 290	1 210	2 120	2 090	3 970	L_u 3 180	7 000	481	15 000	166
t=17.3	1 250	1 270	1 230	1 160	1 660	1 970	3 900	I_x 875	8 000	391	17 000	142
d=608	900	1 200	1 160	1 090	1 190	1 800	3 790	S_x 2 880	9 000	328	19 000	125
W610X101	2 310	1 260	1 240	1 190	3 060	2 050	3 650	M_r 783	5 000	619	11 000	199
W24X68	1 960	1 220	1 210	1 150	2 600	1 980	3 610	V_r 1 130	6 000	512	13 000	158
b=228	1 600	1 190	1 170	1 110	2 120	1 890	3 560	L_u 3 110	7 000	396	15 000	132
t=14.9	1 250	1 150	1 120	1 050	1 660	1 780	3 490	I_x 764	8 000	320	17 000	113
d=603	900	1 090	1 050	984	1 190	1 630	3 400	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 290	1 340	1 320	1 260	3 040	2 000	4 020	M_r 867	4 000	794	9 000	361
W21X83	1 920	1 310	1 280	1 220	2 550	1 920	3 970	V_r 1 270	5 000	706	11 000	281
b=212	1 550	1 270	1 240	1 170	2 060	1 820	3 900	L_u 3 100	6 000	613	13 000	230
t=21.2	1 170	1 220	1 180	1 110	1 550	1 690	3 810	I_x 761	7 000	505	15 000	195
d=544	800	1 140	1 100	1 040	1 060	1 510	3 680	S_x 2 800	8 000	421	17 000	170

Note: *20 MPa, 2300 kg/m³ Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



300W
20 MPa

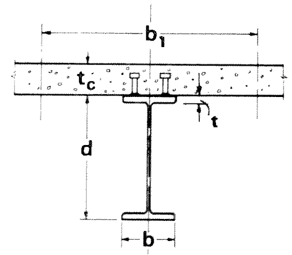
130 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	b_1 (mm)	Composite Beam*: Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W530X109	2 290	1 210	1 190	1 140	3 040	1 790	3 570	M_r 764	4 000	692	9 000	291
W21X73	1 920	1 170	1 150	1 100	2 550	1 720	3 530	V_r 1 110	5 000	608	11 000	225
b=211	1 550	1 130	1 110	1 050	2 060	1 640	3 470	L_u 3 040	6 000	517	13 000	183
t=18.8	1 170	1 090	1 060	996	1 550	1 530	3 400	I_x 667	7 000	413	15 000	155
d=539	800	1 030	988	930	1 060	1 370	3 280	S_x 2 480	8 000	342	17 000	134
W530X101	2 290	1 130	1 110	1 070	3 040	1 680	3 330	M_r 707	4 000	635	9 000	255
W21X68	1 920	1 100	1 080	1 030	2 550	1 620	3 290	V_r 1 040	5 000	553	11 000	196
b=210	1 550	1 060	1 040	990	2 060	1 540	3 240	L_u 2 990	6 000	462	13 000	159
t=17.4	1 170	1 020	996	935	1 550	1 440	3 170	I_x 617	7 000	365	15 000	134
d=537	800	962	927	871	1 060	1 290	3 060	S_x 2 300	8 000	301	17 000	116
W530X92	2 290	1 050	1 030	985	3 040	1 540	3 030	M_r 637	3 000	633	8 000	253
W21X62	1 920	1 010	997	955	2 550	1 490	3 000	V_r 969	4 000	565	10 000	185
b=209	1 540	976	961	914	2 040	1 420	2 950	L_u 2 930	5 000	486	12 000	146
t=15.6	1 170	938	920	863	1 550	1 320	2 890	I_x 552	6 000	393	14 000	120
d=533	800	887	854	800	1 060	1 190	2 800	S_x 2 070	7 000	309	16 000	103
W530X82	2 290	945	926	881	2 830	1 380	2 690	M_r 559	3 000	551	8 000	204
W21X55	1 920	914	900	861	2 550	1 330	2 660	V_r 894	4 000	487	10 000	148
b=209	1 540	878	865	824	2 040	1 270	2 620	L_u 2 860	5 000	412	12 000	115
t=13.3	1 170	842	828	776	1 550	1 190	2 570	I_x 479	6 000	321	14 000	94.8
d=528	800	797	768	717	1 060	1 080	2 490	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 270	1 050	1 030	980	3 010	1 380	3 110	M_r 645	3 000	640	8 000	308
W18X71	1 880	1 010	992	945	2 490	1 320	3 060	V_r 1 050	4 000	579	9 000	266
b=194	1 490	970	953	900	1 980	1 250	3 000	L_u 2 910	5 000	512	11 000	210
t=20.6	1 090	925	900	844	1 450	1 150	2 920	I_x 488	6 000	444	13 000	174
d=469	700	858	827	780	928	1 010	2 800	S_x 2 080	7 000	366	15 000	148
W460X97	2 270	969	952	905	3 010	1 270	2 850	M_r 589	3 000	581	8 000	264
W18X65	1 880	933	916	874	2 490	1 220	2 810	V_r 947	4 000	522	9 000	227
b=193	1 490	894	878	834	1 980	1 160	2 760	L_u 2 870	5 000	457	11 000	178
t=19.0	1 090	851	833	783	1 450	1 070	2 690	I_x 445	6 000	389	13 000	147
d=466	700	794	766	721	928	939	2 580	S_x 1 910	7 000	314	15 000	125
W460X89	2 270	908	893	847	3 010	1 190	2 650	M_r 543	3 000	534	8 000	231
W18X60	1 880	873	857	818	2 490	1 140	2 610	V_r 866	4 000	477	9 000	198
b=192	1 490	835	821	782	1 980	1 080	2 570	L_u 2 830	5 000	414	11 000	155
t=17.7	1 090	794	779	734	1 450	1 000	2 500	I_x 410	6 000	343	13 000	127
d=463	700	742	717	674	928	883	2 400	S_x 1 770	7 000	276	15 000	108
W460X82	2 270	841	821	776	2 810	1 090	2 420	M_r 494	3 000	482	8 000	195
W18X55	1 880	807	793	756	2 490	1 050	2 390	V_r 812	4 000	427	9 000	167
b=191	1 490	771	758	724	1 980	1 000	2 350	L_u 2 770	5 000	365	11 000	129
t=16.0	1 090	731	719	678	1 450	926	2 290	I_x 370	6 000	292	13 000	106
d=460	700	684	662	622	928	819	2 200	S_x 1 610	7 000	234	15 000	90.0

Note: *20 MPa, 2300 kg/m³ Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.1



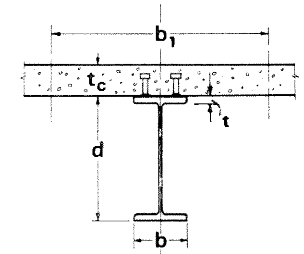
300W
20 MPa

130 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b₁ (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W460X74	2 270	774	751	706	2 550	1 000	2 210	M_r 445	3 000	433	8 000	164
W18X50	1 880	745	733	696	2 490	966	2 180	V_r 733	4 000	380	9 000	140
b=190	1 490	710	698	667	1 980	919	2 140	L_u 2 730	5 000	320	10 000	122
t=14.5	1 090	671	661	626	1 450	854	2 090	I_x 333	6 000	249	12 000	96.9
d=457	700	629	610	572	928	758	2 010	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 270	718	692	648	2 340	923	2 030	M_r 405	3 000	390	8 000	140
W18X46	1 880	693	678	642	2 340	891	2 000	V_r 688	4 000	339	9 000	119
b=190	1 490	660	649	619	1 980	849	1 960	L_u 2 660	5 000	281	10 000	103
t=12.7	1 090	622	612	579	1 450	790	1 920	I_x 300	6 000	214	12 000	82.0
d=454	700	581	563	525	928	703	1 850	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 270	649	621	579	2 100	826	1 810	M_r 354	3 000	336	8 000	111
W18X41	1 880	629	610	574	2 100	798	1 780	V_r 650	4 000	288	9 000	93.9
b=189	1 480	599	589	561	1 960	761	1 750	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 090	563	554	525	1 450	711	1 710	I_x 259	6 000	172	12 000	64.1
d=450	700	525	509	473	928	634	1 640	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 260	803	786	741	2 920	959	2 330	M_r 467	3 000	455	8 000	205
W16X57	1 850	766	752	713	2 450	918	2 290	V_r 810	4 000	406	9 000	178
b=181	1 430	726	712	678	1 900	865	2 240	L_u 2 730	5 000	354	10 000	157
t=18.2	1 020	683	671	631	1 350	792	2 180	I_x 315	6 000	297	11 000	141
d=417	600	627	606	571	796	677	2 070	S_x 1 510	7 000	243	12 000	127
W410X74	2 260	723	701	655	2 580	858	2 070	M_r 408	3 000	394	8 000	163
W16X50	1 850	692	679	643	2 450	823	2 040	V_r 714	4 000	348	9 000	140
b=180	1 430	653	641	611	1 900	777	2 000	L_u 2 670	5 000	297	10 000	124
t=16.0	1 020	613	602	569	1 350	714	1 940	I_x 275	6 000	239	11 000	110
d=413	600	563	544	511	796	613	1 840	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 260	661	635	591	2 320	781	1 870	M_r 367	3 000	352	8 000	135
W16X45	1 840	634	620	584	2 320	750	1 840	V_r 643	4 000	307	9 000	116
b=179	1 430	598	587	559	1 900	710	1 810	L_u 2 610	5 000	258	10 000	102
t=14.4	1 010	558	549	520	1 340	653	1 760	I_x 246	6 000	201	11 000	90.5
d=410	600	514	498	467	796	565	1 670	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 260	592	564	521	2 050	699	1 660	M_r 321	3 000	306	8 000	109
W16X40	1 840	571	552	516	2 050	672	1 640	V_r 558	4 000	264	9 000	93.2
b=178	1 430	540	530	503	1 900	638	1 610	L_u 2 580	5 000	217	10 000	81.4
t=12.8	1 010	501	492	469	1 340	589	1 560	I_x 216	6 000	165	11 000	72.2
d=407	600	460	448	421	796	513	1 490	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 260	536	507	467	1 840	626	1 490	M_r 283	3 000	266	8 000	86.1
W16X36	1 840	519	498	463	1 840	603	1 460	V_r 539	4 000	225	9 000	73.2
b=177	1 430	494	483	457	1 840	573	1 430	L_u 2 480	5 000	176	10 000	63.6
t=10.9	1 010	456	448	427	1 340	530	1 390	I_x 186	6 000	132	11 000	56.3
d=403	600	417	406	380	796	463	1 330	S_x 924	7 000	104	12 000	50.5

Note: *20 MPa, 2300 kg/m³ Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



300W
20 MPa

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.1

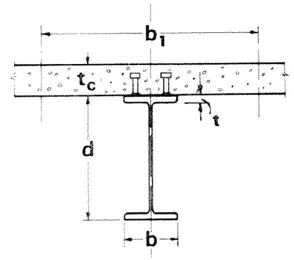
130 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b₁ (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W410X46	2 220	471	443	406	1 590	549	1 290	M_r 239	2 000	236	7 000	61.7
W16X31	1 820	459	436	403	1 590	530	1 270	V_r 503	3 000	195	8 000	51.8
b=140	1 410	439	425	398	1 590	504	1 240	L_u 1 930	4 000	142	9 000	44.6
t=11.2	1 010	407	400	379	1 340	470	1 210	I_x 156	5 000	99.9	10 000	39.2
d=403	600	368	360	335	796	412	1 150	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 220	404	377	343	1 350	468	1 090	M_r 197	2 000	193	6 000	55.2
W16X26	1 820	395	372	341	1 350	453	1 070	V_r 448	3 000	155	7 000	44.1
b=140	1 410	381	364	338	1 350	432	1 050	L_u 1 860	4 000	105	8 000	36.6
t=8.8	1 010	356	350	331	1 340	404	1 020	I_x 127	4 500	86.7	9 000	31.3
d=399	600	318	313	290	796	356	978	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 290	678	657	612	2 730	720	2 000	M_r 386	4 000	364	8 000	225
W14X53	1 850	642	629	592	2 450	687	1 960	V_r 593	5 000	331	8 500	209
b=205	1 420	602	590	560	1 880	645	1 920	L_u 3 270	6 000	298	9 000	195
t=16.8	980	558	548	522	1 300	585	1 850	I_x 227	7 000	264	10 000	172
d=354	550	510	496	469	729	491	1 750	S_x 1 280	7 500	244	11 000	154
W360X72	2 280	620	595	551	2 460	652	1 800	M_r 346	4 000	322	8 000	186
W14X48	1 850	590	578	543	2 450	624	1 770	V_r 536	5 000	290	8 500	172
b=204	1 420	551	540	512	1 880	587	1 730	L_u 3 190	6 000	257	9 000	161
t=15.1	980	509	500	477	1 300	533	1 680	I_x 201	7 000	222	10 000	141
d=350	550	464	452	427	729	450	1 580	S_x 1 150	7 500	203	11 000	126
W360X64	2 280	563	536	493	2 200	590	1 620	M_r 308	4 000	283	8 000	153
W14X43	1 850	539	522	487	2 200	565	1 590	V_r 476	5 000	252	8 500	141
b=203	1 420	503	493	466	1 880	533	1 560	L_u 3 110	6 000	220	9 000	131
t=13.5	980	462	454	433	1 300	486	1 510	I_x 178	7 000	183	10 000	115
d=347	550	419	410	387	729	412	1 430	S_x 1 030	7 500	167	11 000	102
W360X57	2 250	520	491	450	1 950	553	1 460	M_r 273	3 000	259	7 000	119
W14X38	1 830	501	480	445	1 950	531	1 440	V_r 504	4 000	225	7 500	109
b=172	1 400	470	460	434	1 860	501	1 410	L_u 2 550	5 000	189	8 000	99.8
t=13.1	980	431	423	402	1 300	460	1 360	I_x 161	6 000	147	9 000	86.0
d=358	550	388	379	356	729	392	1 290	S_x 897	6 500	132	10 000	75.7
W360X51	2 250	469	441	402	1 740	498	1 310	M_r 241	3 000	227	7 000	97.0
W14X34	1 830	454	432	398	1 740	478	1 290	V_r 455	4 000	195	7 500	88.3
b=171	1 400	429	418	392	1 740	452	1 260	L_u 2 500	5 000	159	8 000	81.0
t=11.6	980	393	385	366	1 300	417	1 220	I_x 141	6 000	121	9 000	69.5
d=355	550	351	344	323	729	357	1 160	S_x 796	6 500	108	10 000	60.9
W360X45	2 250	421	393	357	1 550	445	1 160	M_r 210	3 000	195	7 000	76.5
W14X30	1 830	409	386	354	1 550	428	1 150	V_r 433	4 000	165	7 500	69.4
b=171	1 400	390	375	349	1 550	406	1 120	L_u 2 430	5 000	128	8 000	63.4
t=9.8	980	356	350	331	1 300	375	1 090	I_x 122	6 000	96.1	9 000	54.1
d=352	550	316	310	290	729	322	1 030	S_x 691	6 500	85.3	10 000	47.2

Note: *20 MPa, 2300 kg/m³ Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.1



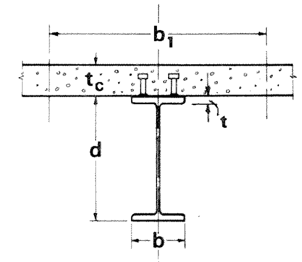
300W
20 MPa

130 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for	10^6mm^4	10^3mm^3		L'	M_r'	L'	M_r'
		100%	75%	50%	100%				mm	kN·m	mm	kN·m
W360X39	2 210	372	345	311	1 340	392	1 010	M_r 179	2 000	173	5 500	61.1
W14X26	1 790	363	340	309	1 340	377	994	V_r 409	3 000	139	6 000	54.2
b=128	1 380	348	331	305	1 340	359	973	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	960	320	314	296	1 270	332	942	I_x 102	4 500	81.3	8 000	37.5
d=353	550	282	277	258	729	288	894	S_x 580	5 000	69.8	9 000	32.5
W360X33	2 210	315	290	260	1 130	331	847	M_r 146	2 000	139	5 500	43.2
W14X22	1 790	308	286	259	1 130	320	835	V_r 361	3 000	108	6 000	38.1
b=127	1 380	298	280	256	1 130	305	819	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	960	278	269	251	1 130	283	794	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	243	238	223	729	248	753	S_x 474	5 000	49.7		
W310X129	2 390	909	888	838	3 170	906	2 940	M_r 583	6 000	573	8 500	515
W12X87	1 920	863	844	802	2 550	855	2 880	V_r 742	6 500	562	9 000	504
b=308	1 440	813	797	764	1 910	788	2 800	L_u 5 580	7 000	550	9 500	492
t=20.6	970	762	750	717	1 290	697	2 690	I_x 308	7 500	539	10 000	481
d=318	500	699	683	657	663	562	2 500	S_x 1 940	8 000	527		
W310X118	2 390	840	821	772	3 170	829	2 680	M_r 526	6 000	513	8 500	455
W12X79	1 920	795	777	737	2 550	784	2 630	V_r 666	6 500	501	9 000	444
b=307	1 440	746	732	700	1 910	724	2 550	L_u 5 390	7 000	490	9 500	432
t=18.7	970	696	686	657	1 290	642	2 450	I_x 275	7 500	478	10 000	421
d=314	500	638	623	598	663	517	2 280	S_x 1 750	8 000	467		
W310X107	2 390	776	759	712	3 170	762	2 450	M_r 478	6 000	461	8 500	404
W12X72	1 910	732	716	677	2 530	721	2 400	V_r 604	6 500	450	9 000	393
b=306	1 440	686	672	642	1 910	668	2 330	L_u 5 220	7 000	438	9 500	381
t=17.0	970	637	627	602	1 290	595	2 250	I_x 248	7 500	427	10 000	370
d=311	500	583	570	546	663	480	2 090	S_x 1 590	8 000	415		
W310X86	2 330	661	644	599	2 970	641	2 010	M_r 383	5 000	367	7 500	309
W12X58	1 880	622	608	571	2 490	609	1 970	V_r 503	5 500	355	8 000	297
b=254	1 420	578	566	537	1 880	567	1 920	L_u 4 250	6 000	344	8 500	285
t=16.3	960	533	523	502	1 270	508	1 850	I_x 199	6 500	332	9 000	273
d=310	500	484	473	451	663	414	1 730	S_x 1 280	7 000	320	9 500	262
W310X79	2 330	615	593	549	2 730	587	1 840	M_r 346	5 000	327	7 500	270
W12X53	1 880	579	566	531	2 490	559	1 800	V_r 480	5 500	316	8 000	258
b=254	1 420	537	526	497	1 880	521	1 760	L_u 4 140	6 000	305	8 500	247
t=14.6	960	492	483	463	1 270	468	1 690	I_x 177	6 500	293	9 000	235
d=306	500	444	434	413	663	381	1 580	S_x 1 160	7 000	282	9 500	221
W310X74	2 290	590	566	521	2 560	562	1 730	M_r 321	4 000	307	7 500	221
W12X50	1 840	556	543	508	2 440	535	1 700	V_r 519	5 000	282	8 000	206
b=205	1 390	515	503	475	1 840	499	1 650	L_u 3 380	6 000	258	8 500	192
t=16.3	950	471	462	440	1 260	450	1 590	I_x 165	6 500	245	9 000	179
d=310	500	423	412	390	663	368	1 490	S_x 1 060	7 000	233	9 500	168

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.1

300W
20 MPa

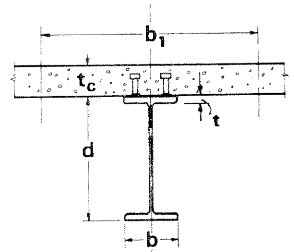
130 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for	10^6mm^4	10^3mm^3		L'	M_r'	L'	M_r'
		100%	75%	50%	100%				mm	kN·m	mm	kN·m
W310X67	2 280	537	510	467	2 300	507	1 560	M_r 286	4 000	270	7 500	184
W12X45	1 840	510	495	460	2 300	483	1 530	V_r 463	5 000	246	8 000	169
b=204	1 390	470	460	433	1 840	452	1 490	L_u 3 280	6 000	222	8 500	157
t=14.6	950	429	420	400	1 260	409	1 430	I_x 145	6 500	210	9 000	147
d=306	500	382	373	353	663	336	1 340	S_x 949	7 000	198	9 500	138
W310X60	2 280	487	458	416	2 050	459	1 400	M_r 254	4 000	237	7 500	151
W12X40	1 840	465	446	411	2 050	438	1 370	V_r 405	5 000	214	8 000	139
b=203	1 390	430	421	395	1 840	411	1 340	L_u 3 200	6 000	191	8 500	129
t=13.1	950	390	382	363	1 260	373	1 290	I_x 129	6 500	179	9 000	120
d=303	500	345	338	320	663	308	1 210	S_x 849	7 000	166	9 500	112
W310X52	2 250	449	420	380	1 800	434	1 260	M_r 226	3 000	216	6 500	116
W12X35	1 810	432	410	376	1 800	416	1 240	V_r 429	4 000	189	7 000	106
b=167	1 370	404	395	369	1 800	391	1 210	L_u 2 570	5 000	162	7 500	96.8
t=13.2	940	365	357	339	1 250	356	1 160	I_x 118	5 500	146	8 000	89.4
d=317	500	321	315	297	663	297	1 090	S_x 747	6 000	130	8 500	83.0
W310X45	2 250	389	360	324	1 540	376	1 080	M_r 191	3 000	180	6 500	87.8
W12X30	1 810	376	353	321	1 540	361	1 060	V_r 368	4 000	155	7 000	79.3
b=166	1 370	356	342	316	1 540	340	1 040	L_u 2 490	5 000	128	7 500	72.4
t=11.2	940	321	314	297	1 250	311	1 000	I_x 99.2	5 500	111	8 000	66.5
d=313	500	279	275	259	663	262	942	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 250	341	314	281	1 330	331	945	M_r 165	3 000	153	6 500	69.1
W12X26	1 810	332	309	279	1 330	318	930	V_r 320	4 000	130	7 000	62.2
b=165	1 370	316	300	275	1 330	301	910	L_u 2 440	5 000	103	7 500	56.5
t=9.7	940	288	282	265	1 250	276	880	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	247	243	230	663	234	827	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 340	661	644	598	3 100	567	2 080	M_r 378	5 000	377	7 500	338
W10X68	1 880	618	602	563	2 490	535	2 030	V_r 560	5 500	369	8 000	330
b=257	1 420	573	559	528	1 880	493	1 970	L_u 4 950	6 000	361	8 500	323
t=19.6	960	525	514	491	1 270	436	1 880	I_x 164	6 500	354		
d=264	500	473	463	442	663	348	1 740	S_x 1 240	7 000	346		
W250X89	2 340	602	586	542	3 080	508	1 850	M_r 332	5 000	327	7 500	289
W10X60	1 880	561	547	510	2 490	481	1 810	V_r 496	5 500	320	8 000	281
b=256	1 420	517	505	475	1 880	445	1 760	L_u 4 690	6 000	312	8 500	273
t=17.3	960	471	461	440	1 270	395	1 680	I_x 143	6 500	304		
d=260	500	422	413	395	663	316	1 550	S_x 1 100	7 000	296		
W250X80	2 340	552	530	485	2 750	460	1 670	M_r 294	5 000	287	7 500	249
W10X54	1 880	514	502	466	2 490	435	1 630	V_r 429	5 500	280	8 000	242
b=255	1 420	472	461	432	1 880	403	1 580	L_u 4 520	6 000	272	8 500	234
t=15.6	960	427	418	398	1 270	359	1 520	I_x 126	6 500	265		
d=256	500	380	373	356	663	289	1 400	S_x 982	7 000	257		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.1



300W
20 MPa

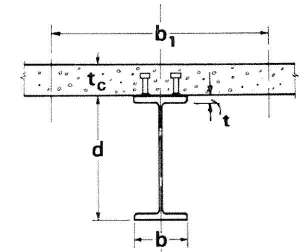
130 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for				L′	M_r′	L′	M_r′
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		mm	kN·m	mm	kN·m
W250X73	2 330	511	486	441	2 510	421	1 520	M_r 266	5 000	257	7 500	220
W10X49	1 880	479	467	433	2 490	400	1 490	V_r 388	5 500	250	8 000	212
b=254	1 420	438	427	400	1 880	371	1 450	L_u 4 390	6 000	242	8 500	205
t=14.2	960	394	386	366	1 270	332	1 390	I_x 113	6 500	235		
d=253	500	348	342	327	663	268	1 290	S_x 891	7 000	227		
W250X67	2 280	482	456	412	2 310	399	1 410	M_r 243	4 000	237	6 500	197
W10X45	1 840	455	440	405	2 310	379	1 380	V_r 408	4 500	229	7 000	189
b=204	1 390	415	405	378	1 840	352	1 340	L_u 3 570	5 000	221	7 500	181
t=15.7	950	374	365	345	1 260	316	1 290	I_x 104	5 500	213	8 000	174
d=257	500	327	321	305	663	257	1 190	S_x 806	6 000	205		
W250X58	2 280	427	398	356	2 000	348	1 230	M_r 208	4 000	199	6 500	160
W10X39	1 840	406	386	351	2 000	332	1 200	V_r 359	4 500	191	7 000	153
b=203	1 390	372	363	337	1 840	309	1 170	L_u 3 410	5 000	184	7 500	145
t=13.5	950	332	324	305	1 260	279	1 120	I_x 87.3	5 500	176	8 000	137
d=252	500	287	282	268	663	227	1 040	S_x 693	6 000	168		
W250X49	2 280	367	338	300	1 690	296	1 040	M_r 171	4 000	160	6 500	123
W10X33	1 840	352	329	296	1 690	283	1 020	V_r 326	4 500	153	7 000	115
b=202	1 390	327	316	290	1 690	265	992	L_u 3 240	5 000	146	7 500	106
t=11.0	950	289	283	265	1 260	239	953	I_x 70.6	5 500	138	8 000	97.2
d=247	500	247	242	229	663	197	885	S_x 572	6 000	130		
W250X45	2 230	354	326	289	1 540	300	983	M_r 163	3 000	151	5 500	101
W10X30	1 800	341	319	286	1 540	287	966	V_r 360	3 500	142	6 000	90.6
b=148	1 360	320	307	281	1 540	269	941	L_u 2 360	4 000	132	6 500	82.2
t=13.0	930	285	278	260	1 230	244	905	I_x 71.1	4 500	122	7 000	75.2
d=266	500	244	239	225	663	203	843	S_x 534	5 000	112	7 500	69.3
W250X39	2 230	308	281	248	1 330	261	851	M_r 139	3 000	126	5 500	77.5
W10X26	1 800	299	276	246	1 330	250	837	V_r 308	3 500	117	6 000	69.2
b=147	1 360	283	267	242	1 330	235	816	L_u 2 280	4 000	108	6 500	62.5
t=11.2	930	254	248	231	1 230	215	787	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	214	210	199	663	180	734	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 230	264	239	209	1 130	223	723	M_r 114	3 000	102	5 500	55.6
W10X22	1 790	257	235	208	1 130	214	711	V_r 280	3 500	93.1	6 000	49.4
b=146	1 360	246	229	205	1 130	202	695	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	930	225	217	200	1 130	185	670	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	187	183	173	663	156	626	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 290	530	514	469	3 000	393	1 620	M_r 265	5 000	261	7 500	235
W8X58	1 840	490	475	438	2 440	370	1 580	V_r 514	5 500	256	8 000	230
b=209	1 390	447	433	403	1 840	339	1 530	L_u 4 620	6 000	251		
t=20.6	950	401	391	369	1 260	299	1 450	I_x 94.7	6 500	245		
d=222	500	352	343	326	663	236	1 320	S_x 853	7 000	240		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.1



300W
20 MPa

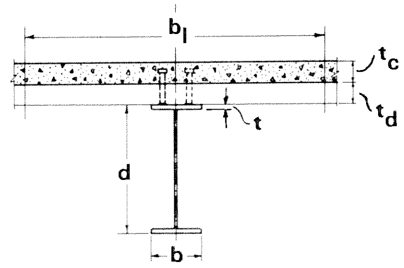
130 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for				L′	M_r′	L′	M_r′
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		mm	kN·m	mm	kN·m
W200X71	2 290	456	431	386	2 460	331	1 350	M_r 217	5 000	208	7 500	183
W8X48	1 840	424	413	378	2 440	312	1 320	V_r 393	5 500	203	8 000	178
b=206	1 390	384	373	345	1 840	288	1 280	L_u 4 150	6 000	198		
t=17.4	950	341	332	312	1 260	256	1 220	I_x 76.6	6 500	193		
d=216	500	294	289	274	663	204	1 120	S_x 709	7 000	188		
W200X59	2 290	390	362	320	2 040	279	1 130	M_r 176	4 000	174	6 500	149
W8X40	1 840	369	350	315	2 040	264	1 110	V_r 341	4 500	169	7 000	144
b=205	1 390	334	325	299	1 840	245	1 070	L_u 3 780	5 000	164	7 500	139
t=14.2	950	293	286	267	1 260	218	1 020	I_x 61.1	5 500	159		
d=210	500	249	244	232	663	175	941	S_x 582	6 000	154		
W200X52	2 280	349	321	281	1 800	248	1 000	M_r 154	4 000	150	6 500	126
W8X35	1 840	333	311	277	1 800	235	982	V_r 290	4 500	145	7 000	121
b=204	1 390	305	295	270	1 800	219	954	L_u 3 620	5 000	140	7 500	116
t=12.6	950	266	259	240	1 260	196	912	I_x 52.7	5 500	135		
d=206	500	222	218	207	663	158	839	S_x 512	6 000	131		
W200X46	2 280	312	284	247	1 580	221	888	M_r 134	4 000	129	6 500	105
W8X31	1 840	300	277	244	1 580	210	871	V_r 260	4 500	124	7 000	99.8
b=203	1 390	278	264	239	1 580	196	847	L_u 3 460	5 000	119	7 500	94.9
t=11.0	950	242	236	218	1 260	176	811	I_x 45.5	5 500	114		
d=203	500	200	196	186	663	143	747	S_x 448	6 000	109		
W200X42	2 250	289	261	226	1 430	205	810	M_r 120	3 000	119	5 500	93.5
W8X28	1 810	278	255	223	1 430	195	794	V_r 263	3 500	114	6 000	88.4
b=166	1 370	260	245	219	1 430	183	773	L_u 2 850	4 000	109	6 500	83.4
t=11.8	940	228	222	204	1 250	165	741	I_x 40.9	4 500	104	7 000	77.7
d=205	500	187	182	172	663	135	684	S_x 399	5 000	98.6		
W200X36	2 250	252	226	194	1 240	179	703	M_r 103	3 000	100	5 500	75.5
W8X24	1 810	244	221	192	1 240	171	690	V_r 222	3 500	95.3	6 000	70.6
b=165	1 370	230	214	189	1 240	160	672	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	940	205	199	183	1 240	145	646	I_x 34.4	4 500	85.5		
d=201	500	166	162	152	663	119	598	S_x 342	5 000	80.5		
W200X31	2 210	228	204	175	1 080	167	627	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	1 790	222	200	174	1 080	160	617	V_r 240	3 500	75.3	6 000	45.6
b=134	1 360	212	195	171	1 080	151	602	L_u 2 150	4 000	69.5		
t=10.2	930	192	184	166	1 080	137	579	I_x 31.4	4 500	63.6		
d=210	500	155	151	142	663	114	537	S_x 299	5 000	57.0		
W200X27	2 210	195	173	148	915	143	534	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	1 780	191	171	147	915	138	526	V_r 214	3 500	59.7	6 000	32.6
b=133	1 360	184	167	145	915	130	514	L_u 2 050	4 000	54.0		
t=8.4	930	170	159	142	915	119	496	I_x 25.8	4 500	47.5		
d=207	500	137	134	125	663	100	462	S_x 249	5 000	41.2		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2



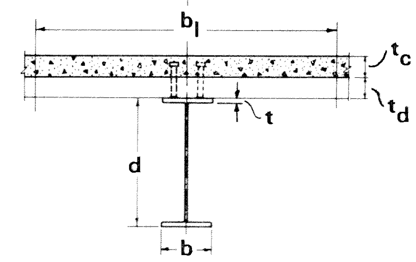
300W
20 MPa

38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for				L'	M_r'	L'	M_r'
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		mm	kN·m	mm	kN·m
WWF700X151	1 950	1 850	1 790	1 710	1 290	3 140	6 010	M_r 1 480	6 000	1 330	14 000	485
	1 690	1 820	1 760	1 680	1 120	3 030	5 950	V_r 846	9 000	942	16 000	404
b=300	1 430	1 780	1 730	1 660	948	2 900	5 880	L_u 4 500	10 000	797	18 000	347
t=22.0	1 180	1 740	1 690	1 630	782	2 750	5 790	I_x 1 740	11 000	688	20 000	303
d=700	920	1 690	1 650	1 590	610	2 590	5 690	S_x 4 980	12 000	605	22 000	270
WWF700X141	1 950	1 750	1 690	1 610	1 290	2 990	5 650	M_r 1 380	6 000	1 220	14 000	420
	1 690	1 720	1 660	1 580	1 120	2 880	5 590	V_r 846	9 000	831	16 000	348
b=300	1 430	1 680	1 630	1 550	948	2 750	5 520	L_u 4 420	10 000	700	18 000	297
t=20.0	1 180	1 640	1 590	1 520	782	2 620	5 440	I_x 1 620	11 000	602	20 000	259
d=700	920	1 590	1 540	1 490	610	2 450	5 340	S_x 4 620	12 000	527	22 000	230
W610X155	1 970	1 640	1 580	1 490	1 310	2 430	5 300	M_r 1 280	6 000	1 180	13 000	516
W24X104	1 700	1 600	1 540	1 470	1 130	2 330	5 230	V_r 1 380	9 000	886	14 000	465
b=324	1 440	1 560	1 510	1 440	955	2 230	5 160	L_u 4 740	10 000	762	16 000	388
t=19.0	1 170	1 520	1 470	1 410	776	2 100	5 070	I_x 1 290	11 000	659	18 000	333
d=611	900	1 470	1 420	1 380	597	1 960	4 950	S_x 4 220	12 000	579	20 000	291
W610X140	1 880	1 480	1 420	1 330	1 250	2 200	4 710	M_r 1 120	5 000	946	11 000	373
W24X94	1 620	1 440	1 380	1 310	1 070	2 110	4 650	V_r 1 440	6 000	829	13 000	303
b=230	1 360	1 400	1 340	1 280	902	2 010	4 570	L_u 3 320	7 000	695	15 000	255
t=22.2	1 110	1 360	1 310	1 250	736	1 890	4 490	I_x 1 120	8 000	573	17 000	221
d=617	850	1 300	1 260	1 220	564	1 760	4 370	S_x 3 630	9 000	486	19 000	195
W610X125	1 880	1 340	1 280	1 200	1 250	2 000	4 210	M_r 991	5 000	821	11 000	301
W24X84	1 620	1 300	1 240	1 170	1 070	1 920	4 160	V_r 1 300	6 000	708	13 000	243
b=229	1 360	1 260	1 210	1 140	902	1 820	4 090	L_u 3 250	7 000	575	15 000	204
t=19.6	1 100	1 220	1 170	1 120	729	1 720	4 010	I_x 985	8 000	470	17 000	176
d=612	850	1 170	1 130	1 090	564	1 590	3 910	S_x 3 220	9 000	396	19 000	155
W610X113	1 880	1 220	1 170	1 090	1 250	1 840	3 810	M_r 888	5 000	719	11 000	247
W24X76	1 620	1 190	1 140	1 070	1 070	1 760	3 770	V_r 1 210	6 000	610	13 000	198
b=228	1 360	1 150	1 100	1 040	902	1 670	3 710	L_u 3 180	7 000	481	15 000	166
t=17.3	1 100	1 110	1 060	1 010	729	1 570	3 630	I_x 875	8 000	391	17 000	142
d=608	850	1 060	1 030	982	564	1 460	3 540	S_x 2 880	9 000	328	19 000	125
W610X101	1 880	1 110	1 060	985	1 250	1 670	3 420	M_r 783	5 000	619	11 000	199
W24X68	1 620	1 080	1 030	960	1 070	1 600	3 380	V_r 1 130	6 000	512	13 000	158
b=228	1 360	1 040	995	933	902	1 520	3 330	L_u 3 110	7 000	396	15 000	132
t=14.9	1 100	1 000	958	905	729	1 430	3 260	I_x 764	8 000	320	17 000	113
d=603	850	959	921	877	564	1 330	3 180	S_x 2 530	9 000	267	19 000	98.4
W530X123	1 860	1 180	1 130	1 050	1 230	1 590	3 740	M_r 867	4 000	794	9 000	361
W21X83	1 600	1 150	1 100	1 030	1 060	1 520	3 690	V_r 1 270	5 000	706	11 000	281
b=212	1 330	1 110	1 060	1 000	882	1 440	3 620	L_u 3 100	6 000	613	13 000	230
t=21.2	1 070	1 070	1 030	977	709	1 350	3 540	I_x 761	7 000	505	15 000	195
d=544	800	1 020	987	948	530	1 240	3 440	S_x 2 800	8 000	421	17 000	170

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm t–mm d–mm M_r–kN·m V_r–kN L_u–mm I_x–10^6mm^4 S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2

300W
20 MPa

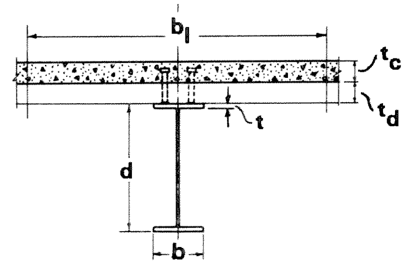
38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for				L'	M_r'	L'	M_r'
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		mm	kN·m	mm	kN·m
W530X109	1 860	1 060	1 010	947	1 230	1 440	3 330	M_r 764	4 000	692	9 000	291
W21X73	1 590	1 030	984	923	1 050	1 380	3 290	V_r 1 110	5 000	608	11 000	225
b=211	1 330	997	953	898	882	1 310	3 230	L_u 3 040	6 000	517	13 000	183
t=18.8	1 060	958	919	871	703	1 220	3 160	I_x 667	7 000	413	15 000	155
d=539	800	915	882	844	530	1 130	3 070	S_x 2 480	8 000	342	17 000	134
W530X101	1 860	994	951	888	1 230	1 360	3 110	M_r 707	4 000	635	9 000	255
W21X68	1 590	967	924	865	1 050	1 300	3 070	V_r 1 040	5 000	553	11 000	196
b=210	1 330	936	894	841	882	1 240	3 020	L_u 2 990	6 000	462	13 000	159
t=17.4	1 060	898	860	814	703	1 160	2 950	I_x 617	7 000	365	15 000	134
d=537	800	857	825	788	530	1 060	2 870	S_x 2 300	8 000	301	17 000	116
W530X92	1 860	916	878	817	1 230	1 250	2 840	M_r 637	3 000	633	8 000	253
W21X62	1 590	891	851	794	1 050	1 200	2 800	V_r 969	4 000	565	10 000	185
b=209	1 330	862	822	770	882	1 140	2 760	L_u 2 930	5 000	486	12 000	146
t=15.6	1 060	826	790	744	703	1 070	2 700	I_x 552	6 000	393	14 000	120
d=533	800	786	755	718	530	981	2 620	S_x 2 070	7 000	309	16 000	103
W530X82	1 860	823	789	732	1 230	1 130	2 520	M_r 559	3 000	551	8 000	204
W21X55	1 590	801	765	710	1 050	1 080	2 490	V_r 894	4 000	487	10 000	148
b=209	1 330	775	737	687	882	1 030	2 450	L_u 2 860	5 000	412	12 000	115
t=13.3	1 060	741	706	662	703	964	2 400	I_x 479	6 000	321	14 000	94.8
d=528	800	702	672	636	530	886	2 340	S_x 1 810	7 000	251	16 000	80.5
W460X106	1 840	908	869	811	1 220	1 100	2 880	M_r 645	3 000	640	8 000	308
W18X71	1 560	881	842	788	1 030	1 050	2 840	V_r 1 050	4 000	579	9 000	266
b=194	1 270	848	811	764	842	985	2 780	L_u 2 910	5 000	512	11 000	210
t=20.6	990	811	778	738	656	912	2 710	I_x 488	6 000	444	13 000	174
d=469	700	767	741	711	464	820	2 610	S_x 2 080	7 000	366	15 000	148
W460X97	1 840	837	804	751	1 220	1 020	2 650	M_r 589	3 000	581	8 000	264
W18X65	1 560	814	779	729	1 030	977	2 610	V_r 947	4 000	522	9 000	227
b=193	1 270	785	750	705	842	919	2 560	L_u 2 870	5 000	457	11 000	178
t=19.0	990	750	719	681	656	852	2 500	I_x 445	6 000	389	13 000	147
d=466	700	709	683	654	464	766	2 400	S_x 1 910	7 000	314	15 000	125
W460X89	1 840	782	753	703	1 220	962	2 460	M_r 543	3 000	534	8 000	231
W18X60	1 560	762	730	682	1 030	918	2 430	V_r 866	4 000	477	9 000	198
b=192	1 270	734	702	659	842	865	2 380	L_u 2 830	5 000	414	11 000	155
t=17.7	990	702	672	635	656	802	2 330	I_x 410	6 000	343	13 000	127
d=463	700	662	637	608	464	721	2 240	S_x 1 770	7 000	276	15 000	108
W460X82	1 840	720	695	649	1 220	890	2 250	M_r 494	3 000	482	8 000	195
W18X55	1 550	701	673	628	1 030	850	2 220	V_r 812	4 000	427	9 000	167
b=191	1 270	678	648	607	842	802	2 180	L_u 2 770	5 000	365	11 000	129
t=16.0	980	646	618	583	650	742	2 130	I_x 370	6 000	292	13 000	106
d=460	700	609	586	558	464	669	2 060	S_x 1 610	7 000	234	15 000	90.0

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm t–mm d–mm M_r–kN·m V_r–kN L_u–mm I_x–10^6mm^4 S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2



300W
20 MPa

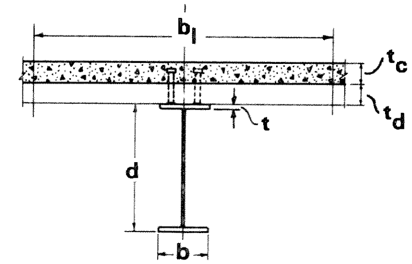
38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition: L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W460X74	1 840	660	640	597	1 220	822	2 050	M_r 445	3 000	433	8 000	164
W18X50	1 550	644	620	578	1 030	785	2 030	V_r 733	4 000	380	9 000	140
b=190	1 270	623	596	557	842	742	1 990	L_u 2 730	5 000	320	10 000	122
t=14.5	980	594	568	534	650	688	1 950	I_x 333	6 000	249	12 000	96.9
d=457	700	559	537	509	464	620	1 880	S_x 1 460	7 000	198	14 000	80.6
W460X67	1 840	612	592	551	1 220	761	1 880	M_r 405	3 000	390	8 000	140
W18X46	1 550	596	573	531	1 030	728	1 860	V_r 688	4 000	339	9 000	119
b=190	1 270	576	550	511	842	689	1 830	L_u 2 660	5 000	281	10 000	103
t=12.7	980	548	522	487	650	639	1 790	I_x 300	6 000	214	12 000	82.0
d=454	700	513	490	462	464	576	1 730	S_x 1 320	7 000	169	14 000	68.0
W460X61	1 840	554	536	497	1 220	684	1 670	M_r 354	3 000	336	8 000	111
W18X41	1 550	538	518	479	1 030	655	1 650	V_r 650	4 000	288	9 000	93.9
b=189	1 270	521	496	458	842	621	1 630	L_u 2 580	5 000	231	10 000	81.3
t=10.8	980	494	469	435	650	577	1 590	I_x 259	6 000	172	12 000	64.1
d=450	700	461	438	411	464	521	1 540	S_x 1 150	7 000	135	14 000	53.0
W410X85	1 830	679	655	611	1 210	772	2 150	M_r 467	3 000	455	8 000	205
W16X57	1 520	659	632	590	1 010	733	2 120	V_r 810	4 000	406	9 000	178
b=181	1 210	633	605	567	802	684	2 070	L_u 2 730	5 000	354	10 000	157
t=18.2	910	600	575	543	603	626	2 010	I_x 315	6 000	297	11 000	141
d=417	600	560	540	517	398	549	1 920	S_x 1 510	7 000	243	12 000	127
W410X74	1 830	608	589	549	1 210	696	1 910	M_r 408	3 000	394	8 000	163
W16X50	1 520	591	568	530	1 010	662	1 880	V_r 714	4 000	348	9 000	140
b=180	1 210	568	544	508	802	620	1 850	L_u 2 670	5 000	297	10 000	124
t=16.0	910	539	515	485	603	568	1 790	I_x 275	6 000	239	11 000	110
d=413	600	501	482	459	398	498	1 720	S_x 1 330	7 000	194	12 000	99.8
W410X67	1 830	555	537	503	1 210	639	1 730	M_r 367	3 000	352	8 000	135
W16X45	1 520	538	520	484	1 010	608	1 710	V_r 643	4 000	307	9 000	116
b=179	1 210	519	497	464	802	570	1 670	L_u 2 610	5 000	258	10 000	102
t=14.4	910	493	471	442	603	524	1 630	I_x 246	6 000	201	11 000	90.5
d=410	600	457	438	416	398	459	1 560	S_x 1 200	7 000	161	12 000	81.7
W410X60	1 830	498	481	453	1 210	576	1 540	M_r 321	3 000	306	8 000	109
W16X40	1 520	482	467	437	1 010	550	1 520	V_r 558	4 000	264	9 000	93.2
b=178	1 210	465	448	417	802	517	1 490	L_u 2 580	5 000	217	10 000	81.4
t=12.8	910	443	424	396	603	476	1 450	I_x 216	6 000	165	11 000	72.2
d=407	600	410	393	372	398	418	1 390	S_x 1 060	7 000	131	12 000	65.0
W410X54	1 830	454	438	411	1 210	518	1 370	M_r 283	3 000	266	8 000	86.1
W16X36	1 520	438	424	395	1 010	495	1 350	V_r 539	4 000	225	9 000	73.2
b=177	1 210	421	405	376	802	466	1 330	L_u 2 480	5 000	176	10 000	63.6
t=10.9	910	401	382	355	603	430	1 290	I_x 186	6 000	132	11 000	56.3
d=403	600	369	352	331	398	378	1 240	S_x 924	7 000	104	12 000	50.5

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2



300W
20 MPa

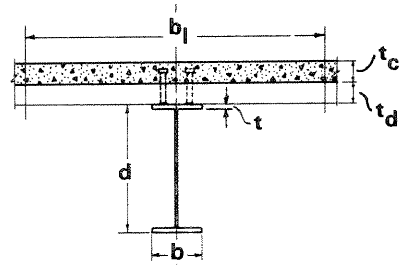
38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition: L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W410X46	1 790	403	387	363	1 190	455	1 180	M_r 239	2 000	236	7 000	61.7
W16X31	1 490	388	374	348	988	437	1 170	V_r 503	3 000	195	8 000	51.8
b=140	1 190	372	357	330	789	413	1 150	L_u 1 930	4 000	142	9 000	44.6
t=11.2	900	353	336	310	597	382	1 120	I_x 156	5 000	99.9	10 000	39.2
d=403	600	324	308	288	398	337	1 070	S_x 773	6 000	76.4	11 000	35.0
W410X39	1 790	352	337	315	1 190	390	996	M_r 197	2 000	193	6 000	55.2
W16X26	1 490	338	325	302	988	375	985	V_r 448	3 000	155	7 000	44.1
b=140	1 190	322	310	286	789	356	969	L_u 1 860	4 000	105	8 000	36.6
t=8.8	900	306	291	267	597	331	947	I_x 127	4 500	86.7	9 000	31.3
d=399	600	280	265	245	398	293	911	S_x 634	5 000	73.1	10 000	27.4
W360X79	1 850	559	540	510	1 230	577	1 830	M_r 386	4 000	364	8 000	225
W14X53	1 530	541	524	492	1 010	547	1 800	V_r 593	5 000	331	8 500	209
b=205	1 200	521	502	472	796	508	1 760	L_u 3 270	6 000	298	9 000	195
t=16.8	880	495	476	450	583	461	1 710	I_x 227	7 000	264	10 000	172
d=354	550	459	444	425	365	397	1 620	S_x 1 280	7 500	244	11 000	154
W360X72	1 850	510	492	465	1 230	526	1 650	M_r 346	4 000	322	8 000	186
W14X48	1 530	492	477	449	1 010	499	1 630	V_r 536	5 000	290	8 500	172
b=204	1 200	474	458	429	796	465	1 590	L_u 3 190	6 000	257	9 000	161
t=15.1	880	451	433	408	583	423	1 550	I_x 201	7 000	222	10 000	141
d=350	550	417	402	384	365	364	1 470	S_x 1 150	7 500	203	11 000	126
W360X64	1 850	463	446	422	1 230	478	1 480	M_r 308	4 000	283	8 000	153
W14X43	1 530	446	432	407	1 010	455	1 460	V_r 476	5 000	252	8 500	141
b=203	1 200	428	415	389	796	425	1 430	L_u 3 110	6 000	220	9 000	131
t=13.5	880	408	392	369	583	387	1 390	I_x 178	7 000	183	10 000	115
d=347	550	377	363	345	365	333	1 330	S_x 1 030	7 500	167	11 000	102
W360X57	1 820	431	414	390	1 210	451	1 340	M_r 273	3 000	259	7 000	119
W14X38	1 500	414	400	375	994	429	1 320	V_r 504	4 000	225	7 500	109
b=172	1 190	397	383	357	789	403	1 290	L_u 2 550	5 000	189	8 000	99.8
t=13.1	870	377	361	337	577	367	1 260	I_x 161	6 000	147	9 000	86.0
d=358	550	346	332	314	365	317	1 200	S_x 897	6 500	132	10 000	75.7
W360X51	1 820	393	376	354	1 210	407	1 190	M_r 241	3 000	227	7 000	97.0
W14X34	1 500	376	362	340	994	389	1 180	V_r 455	4 000	195	7 500	88.3
b=171	1 180	359	347	324	782	365	1 160	L_u 2 500	5 000	159	8 000	81.0
t=11.6	870	341	327	305	577	334	1 130	I_x 141	6 000	121	9 000	69.5
d=355	550	313	300	282	365	289	1 070	S_x 796	6 500	108	10 000	60.9
W360X45	1 820	357	341	320	1 210	365	1 060	M_r 210	3 000	195	7 000	76.5
W14X30	1 500	341	327	307	994	350	1 050	V_r 433	4 000	165	7 500	69.4
b=171	1 180	324	313	291	782	329	1 030	L_u 2 430	5 000	128	8 000	63.4
t=9.8	870	307	294	273	577	302	1 000	I_x 122	6 000	96.1	9 000	54.1
d=352	550	281	268	251	365	262	956	S_x 691	6 500	85.3	10 000	47.2

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2



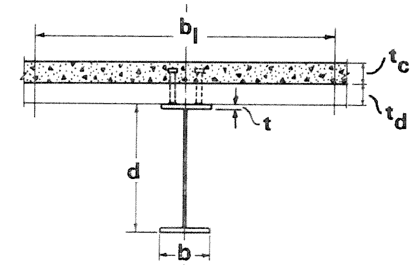
300W
20 MPa

38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W360X39	1 780	320	305	284	1 180	322	916	M_r 179	2 000	173	5 500	61.1
W14X26	1 470	305	292	272	975	309	904	V_r 409	3 000	139	6 000	54.2
b=128	1 160	289	278	257	769	292	888	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	860	273	261	240	570	269	866	I_x 102	4 500	81.3	8 000	37.5
d=353	550	248	236	219	365	235	828	S_x 580	5 000	69.8	9 000	32.5
W360X33	1 780	278	263	243	1 130	274	766	M_r 146	2 000	139	5 500	43.2
W14X22	1 470	265	253	235	975	264	757	V_r 361	3 000	108	6 000	38.1
b=127	1 160	250	240	222	769	250	745	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	860	234	225	206	570	232	728	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	213	202	186	365	203	698	S_x 474	5 000	49.7		
W310X129	1 960	770	748	711	1 300	704	2 680	M_r 583	6 000	573	8 500	515
W12X87	1 590	748	726	691	1 050	659	2 630	V_r 742	6 500	562	9 000	504
b=308	1 230	723	700	668	815	606	2 560	L_u 5 580	7 000	550	9 500	492
t=20.6	860	689	669	643	570	540	2 460	I_x 308	7 500	539	10 000	481
d=318	500	648	634	617	331	459	2 310	S_x 1 940	8 000	527		
W310X118	1 960	705	684	651	1 300	648	2 450	M_r 526	6 000	513	8 500	455
W12X79	1 590	683	664	631	1 050	607	2 400	V_r 666	6 500	501	9 000	444
b=307	1 230	660	640	609	815	558	2 330	L_u 5 390	7 000	490	9 500	432
t=18.7	860	629	609	585	570	497	2 240	I_x 275	7 500	478	10 000	421
d=314	500	589	575	559	331	421	2 110	S_x 1 750	8 000	467		
W310X107	1 950	645	625	596	1 290	598	2 230	M_r 478	6 000	461	8 500	404
W12X72	1 590	625	608	578	1 050	562	2 190	V_r 604	6 500	450	9 000	393
b=306	1 230	604	586	557	815	518	2 140	L_u 5 220	7 000	438	9 500	381
t=17.0	860	575	557	534	570	461	2 060	I_x 248	7 500	427	10 000	370
d=311	500	537	524	508	331	390	1 940	S_x 1 590	8 000	415		
W310X86	1 900	538	520	494	1 260	508	1 830	M_r 383	5 000	367	7 500	309
W12X58	1 550	519	503	478	1 030	478	1 800	V_r 503	5 500	355	8 000	297
b=254	1 200	499	485	460	796	442	1 760	L_u 4 250	6 000	344	8 500	285
t=16.3	850	476	460	438	564	395	1 690	I_x 199	6 500	332	9 000	273
d=310	500	442	430	414	331	334	1 600	S_x 1 280	7 000	320	9 500	262
W310X79	1 900	498	480	455	1 260	466	1 670	M_r 346	5 000	327	7 500	270
W12X53	1 550	479	464	439	1 030	440	1 640	V_r 480	5 500	316	8 000	258
b=254	1 200	460	446	421	796	407	1 600	L_u 4 140	6 000	305	8 500	247
t=14.6	850	437	422	400	564	364	1 550	I_x 177	6 500	293	9 000	235
d=306	500	404	391	376	331	307	1 460	S_x 1 160	7 000	282	9 500	221
W310X74	1 850	476	457	431	1 230	446	1 570	M_r 321	4 000	307	7 500	221
W12X50	1 510	457	441	415	1 000	421	1 540	V_r 519	5 000	282	8 000	206
b=205	1 180	438	423	397	782	391	1 510	L_u 3 380	6 000	258	8 500	192
t=16.3	840	414	398	376	557	350	1 460	I_x 165	6 500	245	9 000	179
d=310	500	381	368	353	331	295	1 370	S_x 1 060	7 000	233	9 500	168

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21−M 300W Steel

Units: b−mm, t−mm; d−mm; M_r−kN·m, V_r−kN; L_u−mm; I_x−10^6mm^4, S_x−10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2

300W
20 MPa

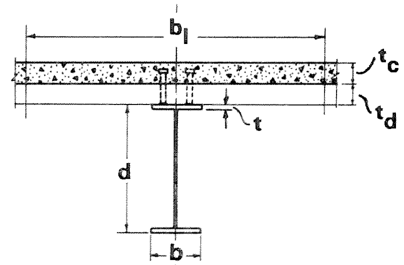
38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W310X67	1 850	433	415	391	1 230	404	1 410	M_r 286	4 000	270	7 500	184
W12X45	1 510	414	400	377	1 000	383	1 390	V_r 463	5 000	246	8 000	169
b=204	1 180	396	383	360	782	356	1 360	L_u 3 280	6 000	222	8 500	157
t=14.6	840	375	361	340	557	319	1 310	I_x 145	6 500	210	9 000	147
d=306	500	344	332	317	331	269	1 240	S_x 949	7 000	198	9 500	138
W310X60	1 850	394	377	354	1 230	368	1 270	M_r 254	4 000	237	7 500	151
W12X40	1 510	376	362	341	1 000	349	1 250	V_r 405	5 000	214	8 000	139
b=203	1 180	358	347	326	782	325	1 220	L_u 3 200	6 000	191	8 500	129
t=13.1	840	339	327	307	557	293	1 180	I_x 129	6 500	179	9 000	120
d=303	500	311	299	285	331	247	1 120	S_x 849	7 000	166	9 500	112
W310X52	1 820	369	352	330	1 210	351	1 140	M_r 226	3 000	216	6 500	116
W12X35	1 490	351	337	317	988	334	1 120	V_r 429	4 000	189	7 000	106
b=167	1 160	334	322	302	769	312	1 100	L_u 2 570	5 000	162	7 500	96.8
t=13.2	830	315	303	283	550	282	1 070	I_x 118	5 500	146	8 000	89.4
d=317	500	288	276	262	331	238	1 010	S_x 747	6 000	130	8 500	83.0
W310X45	1 810	324	309	288	1 200	305	976	M_r 191	3 000	180	6 500	87.8
W12X30	1 490	308	295	277	988	291	963	V_r 368	4 000	155	7 000	79.3
b=166	1 160	291	281	263	769	273	945	L_u 2 490	5 000	128	7 500	72.4
t=11.2	830	274	264	247	550	248	918	I_x 99.2	5 500	111	8 000	66.5
d=313	500	250	240	226	331	211	872	S_x 634	6 000	98.2	8 500	61.6
W310X39	1 810	291	276	256	1 200	269	851	M_r 165	3 000	153	6 500	69.1
W12X26	1 480	275	263	245	981	258	840	V_r 320	4 000	130	7 000	62.2
b=165	1 160	259	249	234	769	243	826	L_u 2 440	5 000	103	7 500	56.5
t=9.7	830	242	234	218	550	222	804	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	222	212	199	331	190	765	S_x 549	6 000	77.7	8 500	47.8
W250X101	1 910	531	511	484	1 270	436	1 860	M_r 378	5 000	377	7 500	338
W10X68	1 550	510	494	468	1 030	407	1 820	V_r 560	5 500	369	8 000	330
b=257	1 200	490	475	450	796	374	1 770	L_u 4 950	6 000	361	8 500	323
t=19.6	850	466	451	430	564	331	1 700	I_x 164	6 500	354		
d=264	500	434	422	408	331	277	1 580	S_x 1 240	7 000	346		
W250X89	1 900	477	458	433	1 260	393	1 660	M_r 332	5 000	327	7 500	289
W10X60	1 550	457	441	418	1 030	368	1 620	V_r 496	5 500	320	8 000	281
b=256	1 200	437	424	402	796	339	1 580	L_u 4 690	6 000	312	8 500	273
t=17.3	850	416	402	383	564	301	1 520	I_x 143	6 500	304		
d=260	500	386	375	361	331	251	1 420	S_x 1 100	7 000	296		
W250X80	1 900	433	415	391	1 260	357	1 490	M_r 294	5 000	287	7 500	249
W10X54	1 550	414	399	378	1 030	335	1 460	V_r 429	5 500	280	8 000	242
b=255	1 200	395	383	363	796	309	1 420	L_u 4 520	6 000	272	8 500	234
t=15.6	850	375	363	345	564	275	1 370	I_x 126	6 500	265		
d=256	500	347	337	324	331	229	1 280	S_x 982	7 000	257		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21−M 300W Steel

Units: b−mm, t−mm; d−mm; M_r−kN·m, V_r−kN; L_u−mm; I_x−10^6mm^4, S_x−10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2



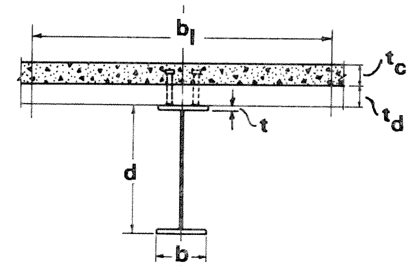
300W
20 MPa

38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W250X73	1 900	400	382	359	1 260	328	1 360	M_r 266	5 000	257	7 500	220
W10X49	1 550	381	367	347	1 030	310	1 330	V_r 388	5 500	250	8 000	212
b=254	1 200	362	351	333	796	286	1 300	L_u 4 390	6 000	242	8 500	205
t=14.2	850	343	332	315	564	254	1 260	I_x 113	6 500	235		
d=253	500	318	308	295	331	212	1 180	S_x 891	7 000	227		
W250X67	1 850	378	360	337	1 230	311	1 260	M_r 243	4 000	237	6 500	197
W10X45	1 510	359	345	325	1 000	294	1 230	V_r 408	4 500	229	7 000	189
b=204	1 180	341	329	310	782	272	1 210	L_u 3 570	5 000	221	7 500	181
t=15.7	840	322	311	293	557	243	1 160	I_x 104	5 500	213	8 000	174
d=257	500	296	286	273	331	202	1 090	S_x 806	6 000	205		
W250X58	1 850	336	319	297	1 230	273	1 090	M_r 208	4 000	199	6 500	160
W10X39	1 510	318	304	285	1 000	258	1 070	V_r 359	4 500	191	7 000	153
b=203	1 180	301	289	273	782	240	1 050	L_u 3 410	5 000	184	7 500	145
t=13.5	840	282	273	256	557	215	1 010	I_x 87.3	5 500	176	8 000	137
d=252	500	259	250	237	331	179	953	S_x 693	6 000	168		
W250X49	1 850	294	278	256	1 230	233	918	M_r 171	4 000	160	6 500	123
W10X33	1 510	277	263	245	1 000	221	904	V_r 326	4 500	153	7 000	115
b=202	1 180	260	249	234	782	206	885	L_u 3 240	5 000	146	7 500	106
t=11.0	840	242	233	218	557	186	857	I_x 70.6	5 500	138	8 000	97.2
d=247	500	220	212	200	331	155	808	S_x 572	6 000	130		
W250X45	1 800	289	273	252	1 190	238	873	M_r 163	3 000	151	5 500	101
W10X30	1 470	272	259	241	975	226	859	V_r 360	3 500	142	6 000	90.6
b=148	1 150	255	244	228	762	211	842	L_u 2 360	4 000	132	6 500	82.2
t=13.0	820	237	228	213	544	191	815	I_x 71.1	4 500	122	7 000	75.2
d=266	500	216	207	195	331	161	770	S_x 534	5 000	112	7 500	69.3
W250X39	1 800	258	243	222	1 190	208	754	M_r 139	3 000	126	5 500	77.5
W10X26	1 470	242	229	212	975	198	743	V_r 308	3 500	117	6 000	69.2
b=147	1 150	226	215	201	762	186	729	L_u 2 280	4 000	108	6 500	62.5
t=11.2	820	208	201	187	544	169	707	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	190	181	170	331	144	671	S_x 459	5 000	88.0	7 500	52.4
W250X33	1 790	226	212	193	1 130	177	637	M_r 114	3 000	102	5 500	55.6
W10X22	1 470	214	202	185	975	170	629	V_r 280	3 500	93.1	6 000	49.4
b=146	1 150	198	188	175	762	160	618	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	820	181	174	161	544	146	600	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	164	156	145	331	125	571	S_x 379	5 000	63.6	7 500	36.9
W200X86	1 860	406	386	361	1 230	295	1 420	M_r 265	5 000	261	7 500	235
W8X58	1 520	386	370	347	1 010	276	1 390	V_r 514	5 500	256	8 000	230
b=209	1 180	366	353	331	782	252	1 340	L_u 4 620	6 000	251		
t=20.6	840	345	332	314	557	222	1 280	I_x 94.7	6 500	245		
d=222	500	317	307	294	331	183	1 190	S_x 853	7 000	240		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.2

300W
20 MPa

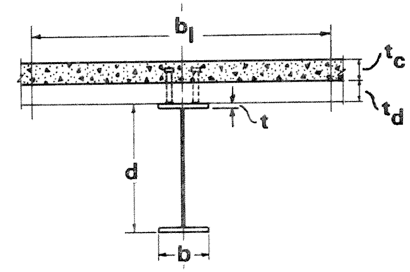
38 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W200X71	1 850	345	327	304	1 230	250	1 180	M_r 217	5 000	208	7 500	183
W8X48	1 520	327	312	292	1 010	236	1 160	V_r 393	5 500	203	8 000	178
b=206	1 180	308	296	279	782	216	1 130	L_u 4 150	6 000	198		
t=17.4	840	289	279	263	557	191	1 080	I_x 76.6	6 500	193		
d=216	500	265	257	245	331	157	1 000	S_x 709	7 000	188		
W200X59	1 850	298	281	258	1 230	212	984	M_r 176	4 000	174	6 500	149
W8X40	1 510	280	266	247	1 000	200	966	V_r 341	4 500	169	7 000	144
b=205	1 180	262	251	235	782	185	942	L_u 3 780	5 000	164	7 500	139
t=14.2	840	244	235	221	557	164	906	I_x 61.1	5 500	159		
d=210	500	223	215	204	331	135	845	S_x 582	6 000	154		
W200X52	1 850	270	254	232	1 230	189	871	M_r 154	4 000	150	6 500	126
W8X35	1 510	253	239	221	1 000	179	855	V_r 290	4 500	145	7 000	121
b=204	1 180	236	225	210	782	166	836	L_u 3 620	5 000	140	7 500	116
t=12.6	840	217	209	197	557	148	806	I_x 52.7	5 500	135		
d=206	500	198	191	181	331	122	754	S_x 512	6 000	131		
W200X46	1 850	246	231	210	1 230	169	769	M_r 134	4 000	129	6 500	105
W8X31	1 510	230	216	199	1 000	160	757	V_r 260	4 500	124	7 000	99.8
b=203	1 180	213	202	188	782	149	740	L_u 3 460	5 000	119	7 500	94.9
t=11.0	840	195	187	176	557	134	715	I_x 45.5	5 500	114		
d=203	500	177	170	160	331	111	671	S_x 448	6 000	109		
W200X42	1 810	231	216	195	1 200	157	700	M_r 120	3 000	119	5 500	93.5
W8X28	1 490	215	203	185	988	149	689	V_r 263	3 500	114	6 000	88.4
b=166	1 160	199	188	174	769	139	674	L_u 2 850	4 000	109	6 500	83.4
t=11.8	830	181	174	162	550	125	652	I_x 40.9	4 500	104	7 000	77.7
d=205	500	163	157	147	331	105	614	S_x 399	5 000	98.6		
W200X36	1 810	209	194	174	1 200	137	606	M_r 103	3 000	100	5 500	75.5
W8X24	1 480	193	181	164	981	130	597	V_r 222	3 500	95.3	6 000	70.6
b=165	1 160	177	167	154	769	122	585	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	830	160	153	143	550	111	567	I_x 34.4	4 500	85.5		
d=201	500	143	137	128	331	93.1	535	S_x 342	5 000	80.5		
W200X31	1 780	193	178	159	1 080	129	542	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	1 460	182	170	153	968	123	534	V_r 240	3 500	75.3	6 000	45.6
b=134	1 140	166	156	143	756	116	523	L_u 2 150	4 000	69.5		
t=10.2	820	150	142	132	544	106	508	I_x 31.4	4 500	63.6		
d=210	500	133	127	118	331	89.6	481	S_x 299	5 000	57.0		
W200X27	1 780	166	152	135	915	111	461	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	1 460	161	149	133	915	106	454	V_r 214	3 500	59.7	6 000	32.6
b=133	1 140	148	138	126	756	100	446	L_u 2 050	4 000	54.0		
t=8.4	820	132	125	115	544	91.9	433	I_x 25.8	4 500	47.5		
d=207	500	116	111	102	331	78.8	412	S_x 249	5 000	41.2		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3



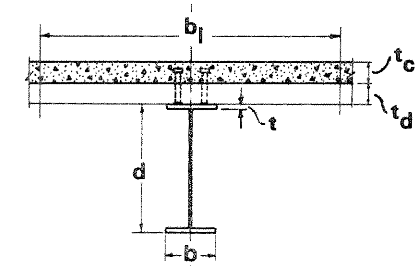
300W
20 MPa

51 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b_1 (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
WWF700X151	2 160	1 890	1 830	1 740	1 430	3 320	6 160	M_r 1 480	6 000	1 330	14 000	485
	1 870	1 860	1 800	1 710	1 240	3 190	6 090	V_r 846	9 000	942	16 000	404
b=300	1 580	1 820	1 760	1 680	1 050	3 050	6 020	L_u 4 500	10 000	797	18 000	347
t=22.0	1 290	1 770	1 710	1 650	855	2 890	5 920	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 720	1 670	1 610	663	2 700	5 800	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 160	1 790	1 730	1 640	1 430	3 160	5 790	M_r 1 380	6 000	1 220	14 000	420
	1 870	1 760	1 700	1 610	1 240	3 040	5 730	V_r 846	9 000	831	16 000	348
b=300	1 580	1 720	1 660	1 580	1 050	2 900	5 660	L_u 4 420	10 000	700	18 000	297
t=20.0	1 290	1 670	1 610	1 540	855	2 740	5 570	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 610	1 560	1 510	663	2 560	5 450	S_x 4 620	12 000	527	22 000	230
W610X155	2 180	1 680	1 610	1 520	1 450	2 590	5 460	M_r 1 280	6 000	1 180	13 000	516
W24X104	1 860	1 640	1 570	1 490	1 230	2 470	5 380	V_r 1 380	9 000	886	14 000	465
b=324	1 540	1 590	1 530	1 460	1 020	2 340	5 290	L_u 4 740	10 000	762	16 000	388
t=19.0	1 220	1 540	1 480	1 420	809	2 180	5 180	I_x 1 290	11 000	659	18 000	333
d=611	900	1 470	1 430	1 380	597	2 010	5 030	S_x 4 220	12 000	579	20 000	291
W610X140	2 090	1 530	1 460	1 360	1 390	2 350	4 870	M_r 1 120	5 000	946	11 000	373
W24X94	1 790	1 480	1 420	1 330	1 190	2 250	4 800	V_r 1 440	6 000	829	13 000	303
b=230	1 490	1 440	1 370	1 300	988	2 130	4 710	L_u 3 320	7 000	695	15 000	255
t=22.2	1 200	1 380	1 330	1 260	796	1 990	4 610	I_x 1 120	8 000	573	17 000	221
d=617	900	1 320	1 280	1 230	597	1 830	4 470	S_x 3 630	9 000	486	19 000	195
W610X125	2 090	1 380	1 320	1 230	1 390	2 140	4 350	M_r 991	5 000	821	11 000	301
W24X84	1 790	1 340	1 280	1 200	1 190	2 040	4 290	V_r 1 300	6 000	708	13 000	243
b=229	1 490	1 290	1 240	1 170	988	1 940	4 220	L_u 3 250	7 000	575	15 000	204
t=19.6	1 200	1 250	1 190	1 130	796	1 810	4 130	I_x 985	8 000	470	17 000	176
d=612	900	1 190	1 150	1 100	597	1 660	4 010	S_x 3 220	9 000	396	19 000	155
W610X113	2 080	1 260	1 200	1 120	1 380	1 960	3 940	M_r 888	5 000	719	11 000	247
W24X76	1 790	1 230	1 170	1 090	1 190	1 880	3 890	V_r 1 210	6 000	610	13 000	198
b=228	1 490	1 180	1 130	1 060	988	1 780	3 820	L_u 3 180	7 000	481	15 000	166
t=17.3	1 200	1 140	1 090	1 030	796	1 670	3 740	I_x 875	8 000	391	17 000	142
d=608	900	1 080	1 040	992	597	1 530	3 630	S_x 2 880	9 000	328	19 000	125
W610X101	2 080	1 150	1 090	1 010	1 380	1 780	3 540	M_r 783	5 000	619	11 000	199
W24X68	1 790	1 120	1 060	984	1 190	1 710	3 490	V_r 1 130	6 000	512	13 000	158
b=228	1 490	1 080	1 020	953	988	1 620	3 430	L_u 3 110	7 000	396	15 000	132
t=14.9	1 200	1 030	981	921	796	1 520	3 360	I_x 764	8 000	320	17 000	113
d=603	900	976	934	887	597	1 390	3 260	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 070	1 220	1 160	1 080	1 370	1 710	3 880	M_r 867	4 000	794	9 000	361
W21X83	1 750	1 180	1 120	1 050	1 160	1 630	3 820	V_r 1 270	5 000	706	11 000	281
b=212	1 430	1 140	1 080	1 020	948	1 530	3 740	L_u 3 100	6 000	613	13 000	230
t=21.2	1 120	1 090	1 040	987	743	1 420	3 640	I_x 761	7 000	505	15 000	195
d=544	800	1 030	992	952	530	1 280	3 510	S_x 2 800	8 000	421	17 000	170

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3

300W
20 MPa

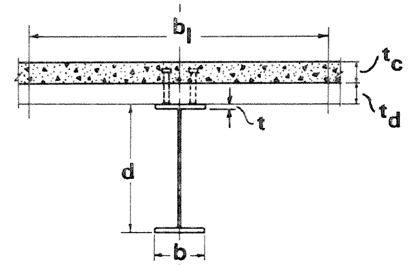
51 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b_1 (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W530X109	2 070	1 100	1 050	974	1 370	1 550	3 460	M_r 764	4 000	692	9 000	291
W21X73	1 750	1 060	1 010	945	1 160	1 480	3 400	V_r 1 110	5 000	608	11 000	225
b=211	1 430	1 020	975	914	948	1 390	3 340	L_u 3 040	6 000	517	13 000	183
t=18.8	1 120	977	934	882	743	1 290	3 260	I_x 667	7 000	413	15 000	155
d=539	800	922	887	848	530	1 160	3 140	S_x 2 480	8 000	342	17 000	134
W530X101	2 070	1 030	984	915	1 370	1 460	3 220	M_r 707	4 000	635	9 000	255
W21X68	1 750	998	952	886	1 160	1 390	3 180	V_r 1 040	5 000	553	11 000	196
b=210	1 430	961	915	856	948	1 310	3 120	L_u 2 990	6 000	462	13 000	159
t=17.4	1 120	917	875	825	743	1 220	3 040	I_x 617	7 000	365	15 000	134
d=537	800	864	830	791	530	1 100	2 940	S_x 2 300	8 000	301	17 000	116
W530X92	2 070	949	909	843	1 370	1 350	2 940	M_r 637	3 000	633	8 000	253
W21X62	1 750	922	878	815	1 160	1 290	2 900	V_r 969	4 000	565	10 000	185
b=209	1 430	886	843	786	948	1 210	2 850	L_u 2 930	5 000	486	12 000	146
t=15.6	1 120	844	804	755	743	1 130	2 780	I_x 552	6 000	393	14 000	120
d=533	800	793	760	722	530	1 010	2 680	S_x 2 070	7 000	309	16 000	103
W530X82	2 070	854	820	757	1 370	1 210	2 620	M_r 559	3 000	551	8 000	204
W21X55	1 750	830	791	731	1 160	1 160	2 580	V_r 894	4 000	487	10 000	148
b=209	1 430	798	758	702	948	1 100	2 540	L_u 2 860	5 000	412	12 000	115
t=13.3	1 120	759	720	672	743	1 020	2 480	I_x 479	6 000	321	14 000	94.8
d=528	800	709	677	639	530	918	2 390	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 050	942	900	836	1 360	1 190	3 000	M_r 645	3 000	640	8 000	308
W18X71	1 710	911	868	808	1 130	1 130	2 950	V_r 1 050	4 000	579	9 000	266
b=194	1 380	873	832	779	915	1 060	2 880	L_u 2 910	5 000	512	11 000	210
t=20.6	1 040	827	791	748	690	964	2 800	I_x 488	6 000	444	13 000	174
d=469	700	773	746	714	464	849	2 670	S_x 2 080	7 000	366	15 000	148
W460X97	2 050	869	834	775	1 360	1 110	2 760	M_r 589	3 000	581	8 000	264
W18X65	1 710	842	804	748	1 130	1 050	2 710	V_r 947	4 000	522	9 000	227
b=193	1 370	807	770	720	908	983	2 650	L_u 2 870	5 000	457	11 000	178
t=19.0	1 040	766	732	690	690	901	2 580	I_x 445	6 000	389	13 000	147
d=466	700	715	688	657	464	794	2 460	S_x 1 910	7 000	314	15 000	125
W460X89	2 050	812	781	726	1 360	1 040	2 570	M_r 543	3 000	534	8 000	231
W18X60	1 710	788	754	701	1 130	989	2 530	V_r 866	4 000	477	9 000	198
b=192	1 370	756	721	673	908	926	2 470	L_u 2 830	5 000	414	11 000	155
t=17.7	1 040	717	684	644	690	849	2 410	I_x 410	6 000	343	13 000	127
d=463	700	668	642	611	464	748	2 300	S_x 1 770	7 000	276	15 000	108
W460X82	2 050	749	722	671	1 360	964	2 350	M_r 494	3 000	482	8 000	195
W18X55	1 710	727	697	647	1 130	917	2 310	V_r 812	4 000	427	9 000	167
b=191	1 370	699	666	620	908	860	2 270	L_u 2 770	5 000	365	11 000	129
t=16.0	1 040	662	631	592	690	790	2 210	I_x 370	6 000	292	13 000	106
d=460	700	615	590	561	464	696	2 110	S_x 1 610	7 000	234	15 000	90.0

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3



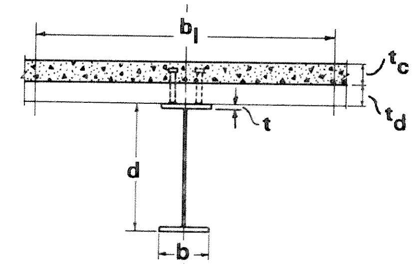
300W
20 MPa

51 mm Deck with 65 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W460X74	2 050	690	665	619	1 360	889	2 140	M_r 445	3 000	433	8 000	164
W18X50	1 710	668	642	596	1 130	848	2 110	V_r 733	4 000	380	9 000	140
b=190	1 370	643	614	570	908	796	2 070	L_u 2 730	5 000	320	10 000	122
t=14.5	1 040	610	581	543	690	732	2 020	I_x 333	6 000	249	12 000	96.9
d=457	700	565	541	512	464	646	1 930	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 050	641	617	573	1 360	823	1 960	M_r 405	3 000	390	8 000	140
W18X46	1 710	620	595	550	1 130	786	1 940	V_r 688	4 000	339	9 000	119
b=190	1 370	596	568	524	908	739	1 900	L_u 2 660	5 000	281	10 000	103
t=12.7	1 040	563	535	497	690	681	1 850	I_x 300	6 000	214	12 000	82.0
d=454	700	519	495	465	464	601	1 780	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 050	582	559	519	1 360	741	1 750	M_r 354	3 000	336	8 000	111
W18X41	1 710	562	540	497	1 130	709	1 720	V_r 650	4 000	288	9 000	93.9
b=189	1 370	540	513	472	908	668	1 690	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 040	509	482	445	690	616	1 650	I_x 259	6 000	172	12 000	64.1
d=450	700	467	443	414	464	544	1 580	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 040	708	681	633	1 350	841	2 250	M_r 467	3 000	455	8 000	205
W16X57	1 680	684	655	608	1 110	796	2 210	V_r 810	4 000	406	9 000	178
b=181	1 320	654	624	581	875	739	2 160	L_u 2 730	5 000	354	10 000	157
t=18.2	960	615	587	552	636	667	2 090	I_x 315	6 000	297	11 000	141
d=417	600	565	544	520	398	571	1 970	S_x 1 510	7 000	243	12 000	127
W410X74	2 040	638	613	570	1 350	758	2 000	M_r 408	3 000	394	8 000	163
W16X50	1 680	614	590	547	1 110	719	1 970	V_r 714	4 000	348	9 000	140
b=180	1 320	589	561	522	875	670	1 930	L_u 2 670	5 000	297	10 000	124
t=16.0	960	553	527	493	636	606	1 860	I_x 275	6 000	239	11 000	110
d=413	600	506	486	462	398	519	1 770	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 040	584	560	523	1 350	695	1 810	M_r 367	3 000	352	8 000	135
W16X45	1 680	561	540	502	1 110	661	1 780	V_r 643	4 000	307	9 000	116
b=179	1 320	538	514	477	875	617	1 750	L_u 2 610	5 000	258	10 000	102
t=14.4	960	506	482	450	636	560	1 690	I_x 246	6 000	201	11 000	90.5
d=410	600	462	442	419	398	480	1 610	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 030	526	503	471	1 350	626	1 610	M_r 321	3 000	306	8 000	109
W16X40	1 680	505	485	453	1 110	597	1 580	V_r 558	4 000	264	9 000	93.2
b=178	1 320	482	464	430	875	560	1 550	L_u 2 580	5 000	217	10 000	81.4
t=12.8	960	456	434	404	636	509	1 510	I_x 216	6 000	165	11 000	72.2
d=407	600	415	397	375	398	438	1 440	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 030	482	459	429	1 350	563	1 430	M_r 283	3 000	266	8 000	86.1
W16X36	1 670	460	441	410	1 110	538	1 410	V_r 539	4 000	225	9 000	73.2
b=177	1 320	439	421	389	875	506	1 390	L_u 2 480	5 000	176	10 000	63.6
t=10.9	960	413	392	363	636	461	1 350	I_x 186	6 000	132	11 000	56.3
d=403	600	374	356	334	398	396	1 280	S_x 924	7 000	104	12 000	50.5

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3

300W
20 MPa

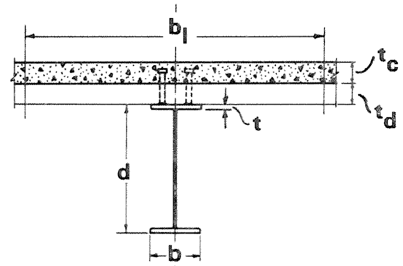
51 mm Deck with 65 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410X46	2 000	431	409	380	1 330	496	1 240	M_r 239	2 000	236	7 000	61.7
W16X31	1 650	410	392	363	1 090	475	1 220	V_r 503	3 000	195	8 000	51.8
b=140	1 300	389	373	342	862	448	1 200	L_u 1 930	4 000	142	9 000	44.6
t=11.2	950	365	346	318	630	410	1 170	I_x 156	5 000	99.9	10 000	39.2
d=403	600	329	312	290	398	355	1 110	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 000	380	359	332	1 330	425	1 040	M_r 197	2 000	193	6 000	55.2
W16X26	1 650	360	342	317	1 090	409	1 030	V_r 448	3 000	155	7 000	44.1
b=140	1 300	339	325	298	862	387	1 010	L_u 1 860	4 000	105	8 000	36.6
t=8.8	950	318	301	275	630	356	989	I_x 127	4 500	86.7	9 000	31.3
d=399	600	285	269	248	398	310	945	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 060	588	563	529	1 370	633	1 920	M_r 386	4 000	364	8 000	225
W14X53	1 680	564	543	508	1 110	597	1 890	V_r 593	5 000	331	8 500	209
b=205	1 310	540	519	485	869	554	1 850	L_u 3 270	6 000	298	9 000	195
t=16.8	930	508	486	458	617	495	1 780	I_x 227	7 000	264	10 000	172
d=354	550	464	447	428	365	415	1 670	S_x 1 280	7 500	244	11 000	154
W360X72	2 060	539	515	483	1 370	577	1 740	M_r 346	4 000	322	8 000	186
W14X48	1 680	515	495	464	1 110	546	1 710	V_r 536	5 000	290	8 500	172
b=204	1 310	491	473	442	869	507	1 670	L_u 3 190	6 000	257	9 000	161
t=15.1	930	463	443	416	617	455	1 610	I_x 201	7 000	222	10 000	141
d=350	550	422	406	386	365	381	1 520	S_x 1 150	7 500	203	11 000	126
W360X64	2 060	492	469	439	1 370	525	1 560	M_r 308	4 000	283	8 000	153
W14X43	1 680	469	449	422	1 110	498	1 530	V_r 476	5 000	252	8 500	141
b=203	1 300	445	429	401	862	463	1 500	L_u 3 110	6 000	220	9 000	131
t=13.5	930	420	402	376	617	417	1 450	I_x 178	7 000	183	10 000	115
d=347	550	382	367	348	365	350	1 370	S_x 1 030	7 500	167	11 000	102
W360X57	2 030	459	437	407	1 350	494	1 400	M_r 273	3 000	259	7 000	119
W14X38	1 660	437	418	390	1 100	470	1 380	V_r 504	4 000	225	7 500	109
b=172	1 290	414	398	369	855	439	1 350	L_u 2 550	5 000	189	8 000	99.8
t=13.1	920	389	370	344	610	396	1 310	I_x 161	6 000	147	9 000	86.0
d=358	550	351	335	316	365	334	1 240	S_x 897	6 500	132	10 000	75.7
W360X51	2 030	421	399	370	1 350	447	1 250	M_r 241	3 000	227	7 000	97.0
W14X34	1 660	399	380	355	1 100	426	1 240	V_r 455	4 000	195	7 500	88.3
b=171	1 290	376	361	335	855	399	1 210	L_u 2 500	5 000	159	8 000	81.0
t=11.6	920	353	337	312	610	361	1 180	I_x 141	6 000	121	9 000	69.5
d=355	550	318	303	285	365	305	1 110	S_x 796	6 500	108	10 000	60.9
W360X45	2 030	384	363	335	1 350	401	1 110	M_r 210	3 000	195	7 000	76.5
W14X30	1 660	363	345	321	1 100	383	1 100	V_r 433	4 000	165	7 500	69.4
b=171	1 290	341	327	302	855	360	1 080	L_u 2 430	5 000	128	8 000	63.4
t=9.8	920	318	303	280	610	327	1 050	I_x 122	6 000	96.1	9 000	54.1
d=352	550	285	271	253	365	277	992	S_x 691	6 500	85.3	10 000	47.2

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3



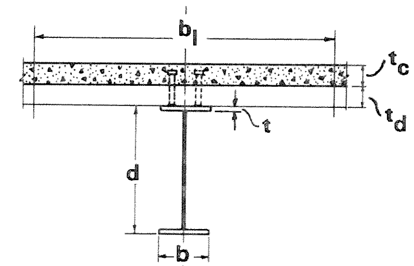
300W
20 MPa

51 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W360X39	1 980	347	327	299	1 310	353	963	M_r 179	2 000	173	5 500	61.1
W14X26	1 630	327	310	286	1 080	339	951	V_r 409	3 000	139	6 000	54.2
b=128	1 270	306	292	269	842	320	934	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	910	284	270	247	603	292	908	I_x 102	4 500	81.3	8 000	37.5
d=353	550	253	239	222	365	249	861	S_x 580	5 000	69.8	9 000	32.5
W360X33	1 980	296	276	252	1 130	300	807	M_r 146	2 000	139	5 500	43.2
W14X22	1 620	286	270	248	1 070	289	797	V_r 361	3 000	108	6 000	38.1
b=127	1 270	266	253	233	842	274	784	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	910	245	233	213	603	252	764	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	218	205	188	365	216	727	S_x 474	5 000	49.7		
W310X129	2 160	800	772	731	1 430	774	2 820	M_r 583	6 000	573	8 500	515
W12X87	1 750	773	748	707	1 160	722	2 760	V_r 742	6 500	562	9 000	504
b=308	1 330	742	717	680	882	658	2 680	L_u 5 580	7 000	550	9 500	492
t=20.6	920	703	680	652	610	580	2 560	I_x 308	7 500	539	10 000	481
d=318	500	652	637	619	331	477	2 370	S_x 1 940	8 000	527		
W310X118	2 160	735	708	670	1 430	712	2 570	M_r 526	6 000	513	8 500	455
W12X79	1 750	708	685	647	1 160	666	2 520	V_r 666	6 500	501	9 000	444
b=307	1 330	679	656	621	882	607	2 450	L_u 5 390	7 000	490	9 500	432
t=18.7	920	642	621	593	610	535	2 340	I_x 275	7 500	478	10 000	421
d=314	500	593	578	561	331	438	2 170	S_x 1 750	8 000	467		
W310X107	2 160	675	649	614	1 430	659	2 350	M_r 478	6 000	461	8 500	404
W12X72	1 750	649	627	593	1 160	617	2 310	V_r 604	6 500	450	9 000	393
b=306	1 330	621	601	569	882	564	2 240	L_u 5 220	7 000	438	9 500	381
t=17.0	920	588	568	542	610	497	2 150	I_x 248	7 500	427	10 000	370
d=311	500	542	527	511	331	406	1 990	S_x 1 590	8 000	415		
W310X86	2 110	568	543	511	1 400	560	1 930	M_r 383	5 000	367	7 500	309
W12X58	1 710	543	522	493	1 130	527	1 890	V_r 503	5 500	355	8 000	297
b=254	1 310	517	500	471	869	484	1 840	L_u 4 250	6 000	344	8 500	285
t=16.3	900	488	470	445	597	427	1 770	I_x 199	6 500	332	9 000	273
d=310	500	447	433	417	331	350	1 650	S_x 1 280	7 000	320	9 500	262
W310X79	2 110	528	503	472	1 400	516	1 760	M_r 346	5 000	327	7 500	270
W12X53	1 710	503	482	454	1 130	485	1 730	V_r 480	5 500	316	8 000	258
b=254	1 310	477	461	433	869	447	1 690	L_u 4 140	6 000	305	8 500	247
t=14.6	900	449	432	407	597	394	1 620	I_x 177	6 500	293	9 000	235
d=306	500	408	395	378	331	322	1 510	S_x 1 160	7 000	282	9 500	221
W310X74	2 060	505	480	449	1 370	494	1 660	M_r 321	4 000	307	7 500	221
W12X50	1 670	480	460	430	1 110	465	1 630	V_r 519	5 000	282	8 000	206
b=205	1 280	455	437	408	849	428	1 590	L_u 3 380	6 000	258	8 500	192
t=16.3	890	426	408	383	590	379	1 520	I_x 165	6 500	245	9 000	179
d=310	500	385	371	355	331	310	1 420	S_x 1 060	7 000	233	9 500	168

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b – mm, t – mm, d – mm, M_r – kN·m, V_r – kN, L_u – mm, I_x – 10^6mm^4, S_x – 10^3mm^3



300W
20 MPa

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3

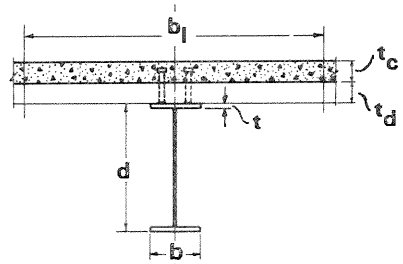
51 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W310X67	2 060	462	438	408	1 370	448	1 490	M_r 286	4 000	270	7 500	184
W12X45	1 670	438	418	391	1 110	423	1 460	V_r 463	5 000	246	8 000	169
b=204	1 280	413	397	371	849	390	1 430	L_u 3 280	6 000	222	8 500	157
t=14.6	890	387	370	347	590	347	1 370	I_x 145	6 500	210	9 000	147
d=306	500	348	335	319	331	283	1 280	S_x 949	7 000	198	9 500	138
W310X60	2 060	422	400	370	1 370	407	1 340	M_r 254	4 000	237	7 500	151
W12X40	1 670	399	380	355	1 110	386	1 320	V_r 405	5 000	214	8 000	139
b=203	1 280	375	360	337	849	357	1 290	L_u 3 200	6 000	191	8 500	129
t=13.1	890	350	336	314	590	319	1 240	I_x 129	6 500	179	9 000	120
d=303	500	315	303	287	331	261	1 160	S_x 849	7 000	166	9 500	112
W310X52	2 020	396	374	345	1 340	387	1 200	M_r 226	3 000	216	6 500	116
W12X35	1 640	373	355	330	1 090	368	1 180	V_r 429	4 000	189	7 000	106
b=167	1 260	350	335	312	835	342	1 160	L_u 2 570	5 000	162	7 500	96.8
t=13.2	880	326	312	290	583	306	1 120	I_x 118	5 500	146	8 000	89.4
d=317	500	292	280	264	331	252	1 050	S_x 747	6 000	130	8 500	83.0
W310X45	2 020	352	331	303	1 340	337	1 030	M_r 191	3 000	180	6 500	87.8
W12X30	1 640	330	312	289	1 090	321	1 020	V_r 368	4 000	155	7 000	79.3
b=166	1 260	307	293	273	835	300	996	L_u 2 490	5 000	128	7 500	72.4
t=11.2	880	284	273	253	583	270	965	I_x 99.2	5 500	111	8 000	66.5
d=313	500	255	243	228	331	224	907	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 020	318	298	271	1 330	297	898	M_r 165	3 000	153	6 500	69.1
W12X26	1 640	297	280	257	1 090	285	887	V_r 320	4 000	130	7 000	62.2
b=165	1 260	275	261	243	835	267	871	L_u 2 440	5 000	103	7 500	56.5
t=9.7	880	252	242	225	583	242	845	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	226	215	201	331	202	798	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 110	561	535	501	1 400	487	1 980	M_r 378	5 000	377	7 500	338
W10X68	1 710	535	513	483	1 130	454	1 930	V_r 560	5 500	369	8 000	330
b=257	1 310	508	490	462	869	414	1 870	L_u 4 950	6 000	361	8 500	323
t=19.6	900	478	460	437	597	361	1 790	I_x 164	6 500	354		
d=264	500	438	425	410	331	292	1 640	S_x 1 240	7 000	346		
W250X89	2 110	507	481	449	1 400	439	1 760	M_r 332	5 000	327	7 500	289
W10X60	1 710	481	460	433	1 130	411	1 720	V_r 496	5 500	320	8 000	281
b=256	1 310	455	438	413	869	376	1 680	L_u 4 690	6 000	312	8 500	273
t=17.3	900	427	411	389	597	328	1 600	I_x 143	6 500	304		
d=260	500	390	378	364	331	265	1 470	S_x 1 100	7 000	296		
W250X80	2 110	463	438	407	1 400	399	1 580	M_r 294	5 000	287	7 500	249
W10X54	1 710	438	417	392	1 130	375	1 550	V_r 429	5 500	280	8 000	242
b=255	1 310	412	396	373	869	343	1 510	L_u 4 520	6 000	272	8 500	234
t=15.6	900	385	372	351	597	301	1 450	I_x 126	6 500	265		
d=256	500	352	340	326	331	242	1 340	S_x 982	7 000	257		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b – mm, t – mm, d – mm, M_r – kN·m, V_r – kN, L_u – mm, I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3



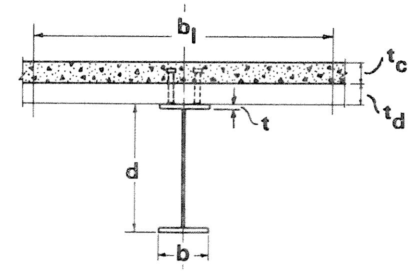
300W
20 MPa

51 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* — Factored Resistances — M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape — Steel Shape Data	Unbraced Condition — L' mm	M_r' kN·m	L' mm	M_r' kN·m
W250X73	2 110	429	405	375	1 400	368	1 440	M_r 266	5 000	257	7 500	220
W10X49	1 710	405	385	360	1 130	346	1 420	V_r 388	5 500	250	8 000	212
b=254	1 310	380	364	343	869	318	1 380	L_u 4 390	6 000	242	8 500	205
t=14.2	900	353	341	322	597	279	1 330	I_x 113	6 500	235		
d=253	500	322	311	298	331	225	1 230	S_x 891	7 000	227		
W250X67	2 060	407	383	353	1 370	349	1 340	M_r 243	4 000	237	6 500	197
W10X45	1 670	383	363	338	1 110	329	1 310	V_r 408	4 500	229	7 000	189
b=204	1 280	358	342	320	849	302	1 280	L_u 3 570	5 000	221	7 500	181
t=15.7	890	333	320	300	590	267	1 230	I_x 104	5 500	213	8 000	174
d=257	500	300	289	275	331	216	1 140	S_x 806	6 000	205		
W250X58	2 060	365	342	313	1 370	306	1 160	M_r 208	4 000	199	6 500	160
W10X39	1 670	341	322	298	1 110	289	1 140	V_r 359	4 500	191	7 000	153
b=203	1 280	317	302	282	849	267	1 110	L_u 3 410	5 000	184	7 500	145
t=13.5	890	293	281	263	590	237	1 070	I_x 87.3	5 500	176	8 000	137
d=252	500	263	253	239	331	192	997	S_x 693	6 000	168		
W250X49	2 060	322	300	272	1 370	262	979	M_r 171	4 000	160	6 500	123
W10X33	1 670	299	281	258	1 110	248	963	V_r 326	4 500	153	7 000	115
b=202	1 280	276	262	243	849	230	942	L_u 3 240	5 000	146	7 500	106
t=11.0	890	252	242	224	590	205	909	I_x 70.6	5 500	138	8 000	97.2
d=247	500	225	215	202	331	167	847	S_x 572	6 000	130		
W250X45	2 000	316	295	267	1 330	265	928	M_r 163	3 000	151	5 500	101
W10X30	1 630	294	276	253	1 080	253	914	V_r 360	3 500	142	6 000	90.6
b=148	1 250	271	257	238	829	235	894	L_u 2 360	4 000	132	6 500	82.2
t=13.0	880	248	237	219	583	211	864	I_x 71.1	4 500	122	7 000	75.2
d=266	500	220	210	197	331	173	807	S_x 534	5 000	112	7 500	69.3
W250X39	2 000	285	264	237	1 330	232	802	M_r 139	3 000	126	5 500	77.5
W10X26	1 630	264	247	224	1 080	221	791	V_r 308	3 500	117	6 000	69.2
b=147	1 250	242	228	210	829	207	775	L_u 2 280	4 000	108	6 500	62.5
t=11.2	880	219	209	193	583	187	751	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	194	185	172	331	155	705	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 000	245	225	201	1 130	198	679	M_r 114	3 000	102	5 500	55.6
W10X22	1 630	235	219	197	1 080	190	670	V_r 280	3 500	93.1	6 000	49.4
b=146	1 250	214	201	183	829	179	657	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	880	192	182	168	583	163	638	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	168	159	147	331	135	601	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 070	436	410	377	1 370	336	1 520	M_r 265	5 000	261	7 500	235
W8X58	1 670	409	388	361	1 110	313	1 490	V_r 514	5 500	256	8 000	230
b=209	1 280	383	366	342	849	284	1 440	L_u 4 620	6 000	251		
t=20.6	890	356	341	320	590	246	1 370	I_x 94.7	6 500	245		
d=222	500	321	310	297	331	195	1 240	S_x 853	7 000	240		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.3

300W
20 MPa

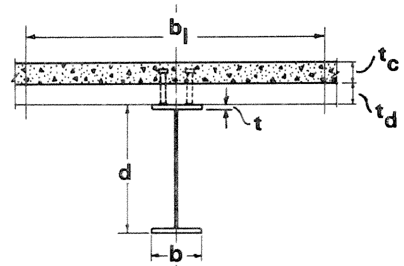
51 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* — Factored Resistances — M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape — Steel Shape Data	Unbraced Condition — L' mm	M_r' kN·m	L' mm	M_r' kN·m
W200X71	2 060	374	350	320	1 370	285	1 270	M_r 217	5 000	208	7 500	183
W8X48	1 670	350	330	305	1 110	267	1 240	V_r 393	5 500	203	8 000	178
b=206	1 280	325	309	288	849	244	1 210	L_u 4 150	6 000	198		
t=17.4	890	299	287	269	590	213	1 150	I_x 76.6	6 500	193		
d=216	500	270	260	247	331	170	1 060	S_x 709	7 000	188		
W200X59	2 060	326	304	274	1 370	241	1 060	M_r 176	4 000	174	6 500	149
W8X40	1 670	303	284	260	1 110	227	1 040	V_r 341	4 500	169	7 000	144
b=205	1 280	279	264	245	849	209	1 010	L_u 3 780	5 000	164	7 500	139
t=14.2	890	254	243	227	590	184	969	I_x 61.1	5 500	159		
d=210	500	227	218	206	331	146	893	S_x 582	6 000	154		
W200X52	2 060	298	276	248	1 370	215	937	M_r 154	4 000	150	6 500	126
W8X35	1 670	275	257	233	1 110	204	920	V_r 290	4 500	145	7 000	121
b=204	1 280	252	237	219	849	188	897	L_u 3 620	5 000	140	7 500	116
t=12.6	890	228	217	203	590	166	862	I_x 52.7	5 500	135		
d=206	500	202	194	183	331	133	798	S_x 512	6 000	131		
W200X46	2 060	274	253	225	1 370	192	828	M_r 134	4 000	129	6 500	105
W8X31	1 670	252	234	211	1 110	182	814	V_r 260	4 500	124	7 000	99.8
b=203	1 280	229	215	197	849	169	795	L_u 3 460	5 000	119	7 500	94.9
t=11.0	890	205	195	182	590	150	766	I_x 45.5	5 500	114		
d=203	500	181	173	162	331	121	712	S_x 448	6 000	109		
W200X42	2 020	259	238	210	1 340	179	754	M_r 120	3 000	119	5 500	93.5
W8X28	1 640	237	220	197	1 090	170	742	V_r 263	3 500	114	6 000	88.4
b=166	1 260	215	201	183	835	158	725	L_u 2 850	4 000	109	6 500	83.4
t=11.8	880	192	182	168	583	141	699	I_x 40.9	4 500	104	7 000	77.7
d=205	500	168	160	149	331	114	651	S_x 399	5 000	98.6		
W200X36	2 020	231	210	185	1 240	156	653	M_r 103	3 000	100	5 500	75.5
W8X24	1 640	215	198	176	1 090	149	643	V_r 222	3 500	95.3	6 000	70.6
b=165	1 260	193	180	162	835	139	629	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	880	171	161	148	583	125	608	I_x 34.4	4 500	85.5		
d=201	500	147	141	131	331	102	569	S_x 342	5 000	80.5		
W200X31	1 990	210	191	167	1 080	146	583	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	1 620	203	187	165	1 070	140	574	V_r 240	3 500	75.3	6 000	45.6
b=134	1 250	182	169	152	829	132	563	L_u 2 150	4 000	69.5		
t=10.2	870	160	150	137	577	119	545	I_x 31.4	4 500	63.6		
d=210	500	137	130	120	331	98.4	512	S_x 299	5 000	57.0		
W200X27	1 990	180	162	141	915	126	496	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	1 620	176	160	140	915	121	489	V_r 214	3 500	59.7	6 000	32.6
b=133	1 240	163	151	134	822	114	479	L_u 2 050	4 000	54.0		
t=8.4	870	142	133	121	577	104	465	I_x 25.8	4 500	47.5		
d=207	500	120	114	104	331	86.8	439	S_x 249	5 000	41.2		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.4



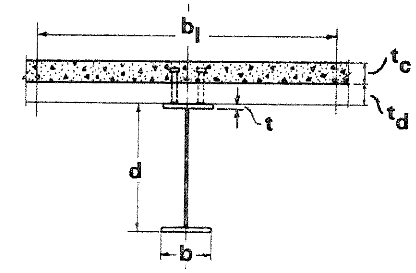
300W
20 MPa

76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for	10^6mm^4	10^3mm^3		L'	M_r'	L'	M_r'
		100%	75%	50%	100%				mm	kN·m	mm	kN·m
WWF700X151	2 560	1 960	1 900	1 800	1 700	3 680	6 460	M_r 1 480	6 000	1 330	14 000	485
	2 170	1 920	1 860	1 760	1 440	3 510	6 380	V_r 846	9 000	942	16 000	404
b=300	1 780	1 870	1 810	1 720	1 180	3 320	6 280	L_u 4 500	10 000	797	18 000	347
t=22.0	1 390	1 810	1 750	1 670	922	3 090	6 140	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 730	1 680	1 620	663	2 810	5 960	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 560	1 860	1 800	1 700	1 700	3 500	6 080	M_r 1 380	6 000	1 220	14 000	420
	2 170	1 830	1 760	1 660	1 440	3 350	6 000	V_r 846	9 000	831	16 000	348
b=300	1 780	1 770	1 710	1 620	1 180	3 160	5 910	L_u 4 420	10 000	700	18 000	297
t=20.0	1 390	1 710	1 650	1 570	922	2 940	5 780	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 630	1 580	1 520	663	2 670	5 610	S_x 4 620	12 000	527	22 000	230
W610X155	2 580	1 770	1 690	1 580	1 710	2 900	5 770	M_r 1 280	6 000	1 180	13 000	516
W24X104	2 160	1 710	1 640	1 540	1 430	2 750	5 680	V_r 1 380	9 000	886	14 000	465
b=324	1 740	1 650	1 580	1 490	1 150	2 570	5 570	L_u 4 740	10 000	762	16 000	388
t=19.0	1 320	1 580	1 510	1 440	875	2 360	5 410	I_x 1 290	11 000	659	18 000	333
d=611	900	1 490	1 440	1 390	597	2 100	5 190	S_x 4 220	12 000	579	20 000	291
W610X140	2 490	1 620	1 540	1 430	1 650	2 650	5 170	M_r 1 120	5 000	946	11 000	373
W24X94	2 090	1 560	1 480	1 380	1 390	2 510	5 080	V_r 1 440	6 000	829	13 000	303
b=230	1 690	1 500	1 420	1 330	1 120	2 350	4 970	L_u 3 320	7 000	695	15 000	255
t=22.2	1 300	1 420	1 360	1 290	862	2 160	4 830	I_x 1 120	8 000	573	17 000	221
d=617	900	1 340	1 290	1 230	597	1 920	4 630	S_x 3 630	9 000	486	19 000	195
W610X125	2 490	1 460	1 390	1 290	1 650	2 410	4 620	M_r 991	5 000	821	11 000	301
W24X84	2 090	1 410	1 340	1 250	1 390	2 290	4 550	V_r 1 300	6 000	708	13 000	243
b=229	1 690	1 350	1 290	1 200	1 120	2 140	4 460	L_u 3 250	7 000	575	15 000	204
t=19.6	1 300	1 280	1 220	1 150	862	1 970	4 330	I_x 985	8 000	470	17 000	176
d=612	900	1 200	1 160	1 100	597	1 750	4 150	S_x 3 220	9 000	396	19 000	155
W610X113	2 480	1 340	1 280	1 180	1 640	2 210	4 190	M_r 888	5 000	719	11 000	247
W24X76	2 090	1 300	1 230	1 140	1 390	2 110	4 130	V_r 1 210	6 000	610	13 000	198
b=228	1 690	1 240	1 180	1 090	1 120	1 970	4 040	L_u 3 180	7 000	481	15 000	166
t=17.3	1 300	1 180	1 120	1 050	862	1 820	3 930	I_x 875	8 000	391	17 000	142
d=608	900	1 100	1 050	999	597	1 620	3 770	S_x 2 880	9 000	328	19 000	125
W610X101	2 480	1 220	1 160	1 070	1 640	2 010	3 760	M_r 783	5 000	619	11 000	199
W24X68	2 090	1 180	1 120	1 030	1 390	1 920	3 710	V_r 1 130	6 000	512	13 000	158
b=228	1 690	1 130	1 070	988	1 120	1 800	3 640	L_u 3 110	7 000	396	15 000	132
t=14.9	1 300	1 070	1 010	943	862	1 660	3 540	I_x 764	8 000	320	17 000	113
d=603	900	991	946	894	597	1 470	3 390	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 470	1 300	1 230	1 140	1 640	1 950	4 150	M_r 867	4 000	794	9 000	361
W21X83	2 050	1 250	1 180	1 100	1 360	1 840	4 070	V_r 1 270	5 000	706	11 000	281
b=212	1 630	1 190	1 130	1 050	1 080	1 710	3 980	L_u 3 100	6 000	613	13 000	230
t=21.2	1 220	1 120	1 070	1 010	809	1 560	3 850	I_x 761	7 000	505	15 000	195
d=544	800	1 040	1 000	958	530	1 350	3 650	S_x 2 800	8 000	421	17 000	170

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.4

300W
20 MPa

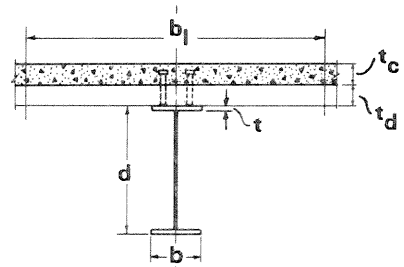
76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for	10^6mm^4	10^3mm^3		L'	M_r'	L'	M_r'
		100%	75%	50%	100%				mm	kN·m	mm	kN·m
W530X109	2 470	1 170	1 120	1 030	1 640	1 770	3 700	M_r 764	4 000	692	9 000	291
W21X73	2 050	1 130	1 070	989	1 360	1 670	3 640	V_r 1 110	5 000	608	11 000	225
b=211	1 630	1 080	1 020	947	1 080	1 560	3 560	L_u 3 040	6 000	517	13 000	183
t=18.8	1 220	1 010	963	903	809	1 420	3 440	I_x 667	7 000	413	15 000	155
d=539	800	935	897	855	530	1 230	3 270	S_x 2 480	8 000	342	17 000	134
W530X101	2 470	1 100	1 050	968	1 640	1 670	3 450	M_r 707	4 000	635	9 000	255
W21X68	2 050	1 060	1 010	930	1 360	1 580	3 390	V_r 1 040	5 000	553	11 000	196
b=210	1 630	1 010	959	889	1 080	1 470	3 320	L_u 2 990	6 000	462	13 000	159
t=17.4	1 220	952	904	845	809	1 340	3 220	I_x 617	7 000	365	15 000	134
d=537	800	877	840	798	530	1 170	3 060	S_x 2 300	8 000	301	17 000	116
W530X92	2 470	1 020	972	895	1 640	1 540	3 150	M_r 637	3 000	633	8 000	253
W21X62	2 050	981	933	858	1 360	1 460	3 100	V_r 969	4 000	565	10 000	185
b=209	1 630	936	886	818	1 080	1 370	3 040	L_u 2 930	5 000	486	12 000	146
t=15.6	1 220	879	832	775	809	1 250	2 950	I_x 552	6 000	393	14 000	120
d=533	800	806	770	728	530	1 080	2 800	S_x 2 070	7 000	309	16 000	103
W530X82	2 470	920	879	808	1 640	1 390	2 800	M_r 559	3 000	551	8 000	204
W21X55	2 050	886	844	773	1 360	1 320	2 760	V_r 894	4 000	487	10 000	148
b=209	1 630	846	799	734	1 080	1 240	2 710	L_u 2 860	5 000	412	12 000	115
t=13.3	1 220	792	748	692	809	1 130	2 630	I_x 479	6 000	321	14 000	94.8
d=528	800	722	687	646	530	983	2 510	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 450	1 010	964	886	1 620	1 380	3 240	M_r 645	3 000	640	8 000	308
W18X71	2 010	972	922	849	1 330	1 300	3 180	V_r 1 050	4 000	579	9 000	266
b=194	1 580	923	874	810	1 050	1 200	3 100	L_u 2 910	5 000	512	11 000	210
t=20.6	1 140	860	817	766	756	1 080	2 980	I_x 488	6 000	444	13 000	174
d=469	700	785	754	720	464	908	2 800	S_x 2 080	7 000	366	15 000	148
W460X97	2 450	936	894	824	1 620	1 280	2 970	M_r 589	3 000	581	8 000	264
W18X65	2 010	899	856	788	1 330	1 210	2 920	V_r 947	4 000	522	9 000	227
b=193	1 570	855	810	750	1 040	1 120	2 850	L_u 2 870	5 000	457	11 000	178
t=19.0	1 140	798	758	709	756	1 010	2 750	I_x 445	6 000	389	13 000	147
d=466	700	726	697	663	464	851	2 580	S_x 1 910	7 000	314	15 000	125
W460X89	2 450	879	838	774	1 620	1 200	2 770	M_r 543	3 000	534	8 000	231
W18X60	2 010	842	803	740	1 330	1 140	2 720	V_r 866	4 000	477	9 000	198
b=192	1 570	802	760	702	1 040	1 060	2 660	L_u 2 830	5 000	414	11 000	155
t=17.7	1 140	749	710	662	756	951	2 570	I_x 410	6 000	343	13 000	127
d=463	700	679	650	617	464	804	2 420	S_x 1 770	7 000	276	15 000	108
W460X82	2 450	815	776	717	1 620	1 110	2 530	M_r 494	3 000	482	8 000	195
W18X55	2 010	780	744	685	1 330	1 060	2 490	V_r 812	4 000	427	9 000	167
b=191	1 570	742	704	649	1 040	982	2 440	L_u 2 770	5 000	365	11 000	129
t=16.0	1 140	693	657	610	756	886	2 360	I_x 370	6 000	292	13 000	106
d=460	700	627	599	567	464	749	2 220	S_x 1 610	7 000	234	15 000	90.0

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.4



300W
20 MPa

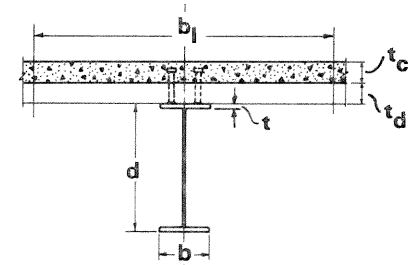
76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b_1 (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W460X74	2 450	755	716	663	1 620	1 030	2 310	M_r 445	3 000	433	8 000	164
W18X50	2 010	720	687	633	1 330	976	2 280	V_r 733	4 000	380	9 000	140
b=190	1 570	685	651	599	1 040	910	2 230	L_u 2 730	5 000	320	10 000	122
t=14.5	1 140	640	606	561	756	823	2 160	I_x 333	6 000	249	12 000	96.9
d=457	700	577	550	518	464	697	2 040	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 450	706	668	616	1 620	951	2 120	M_r 405	3 000	390	8 000	140
W18X46	2 010	672	640	587	1 330	906	2 090	V_r 688	4 000	339	9 000	119
b=190	1 570	637	604	553	1 040	846	2 050	L_u 2 660	5 000	281	10 000	103
t=12.7	1 140	593	559	515	756	767	1 980	I_x 300	6 000	214	12 000	82.0
d=454	700	531	503	471	464	651	1 880	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 450	647	610	561	1 620	856	1 890	M_r 354	3 000	336	8 000	111
W18X41	2 010	613	582	533	1 330	817	1 860	V_r 650	4 000	288	9 000	93.9
b=189	1 570	579	549	500	1 040	765	1 830	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 140	538	506	463	756	696	1 770	I_x 259	6 000	172	12 000	64.1
d=450	700	478	452	420	464	592	1 680	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 440	775	735	678	1 620	982	2 450	M_r 467	3 000	455	8 000	205
W16X57	1 980	737	702	645	1 310	926	2 400	V_r 810	4 000	406	9 000	178
b=181	1 520	697	661	609	1 010	853	2 340	L_u 2 730	5 000	354	10 000	157
t=18.2	1 060	644	611	569	703	756	2 250	I_x 315	6 000	297	11 000	141
d=417	600	575	552	525	398	617	2 080	S_x 1 510	7 000	243	12 000	127
W410X74	2 440	703	664	613	1 620	885	2 170	M_r 408	3 000	394	8 000	163
W16X50	1 980	667	634	583	1 310	837	2 140	V_r 714	4 000	348	9 000	140
b=180	1 520	629	597	549	1 010	774	2 090	L_u 2 670	5 000	297	10 000	124
t=16.0	1 060	581	550	510	703	688	2 010	I_x 275	6 000	239	11 000	110
d=413	600	516	493	467	398	563	1 870	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 440	649	611	564	1 620	811	1 970	M_r 367	3 000	352	8 000	135
W16X45	1 980	613	582	536	1 310	769	1 940	V_r 643	4 000	307	9 000	116
b=179	1 520	576	548	504	1 010	714	1 890	L_u 2 610	5 000	258	10 000	102
t=14.4	1 060	534	505	466	703	637	1 820	I_x 246	6 000	201	11 000	90.5
d=410	600	472	450	424	398	522	1 700	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 430	590	554	509	1 610	730	1 740	M_r 321	3 000	306	8 000	109
W16X40	1 980	556	525	485	1 310	695	1 720	V_r 558	4 000	264	9 000	93.2
b=178	1 520	520	496	456	1 010	648	1 680	L_u 2 580	5 000	217	10 000	81.4
t=12.8	1 060	482	456	421	703	581	1 630	I_x 216	6 000	165	11 000	72.2
d=407	600	425	405	380	398	478	1 520	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 430	545	509	465	1 610	657	1 560	M_r 283	3 000	266	8 000	86.1
W16X36	1 970	511	481	442	1 310	627	1 540	V_r 539	4 000	225	9 000	73.2
b=177	1 520	476	452	414	1 010	586	1 510	L_u 2 480	5 000	176	10 000	63.6
t=10.9	1 060	439	414	379	703	527	1 460	I_x 186	6 000	132	11 000	56.3
d=403	600	384	363	339	398	435	1 360	S_x 924	7 000	104	12 000	50.5

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm, d – mm, M_r – kN·m, V_r – kN, L_u – mm, I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.4



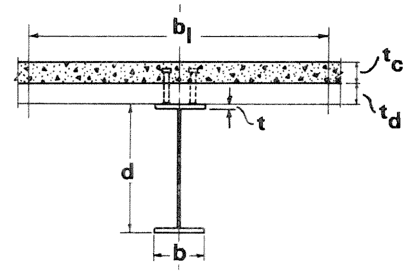
300W
20 MPa

76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b_1 (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410X46	2 400	493	459	415	1 590	579	1 350	M_r 239	2 000	236	7 000	61.7
W16X31	1 950	460	431	394	1 290	554	1 330	V_r 503	3 000	195	8 000	51.8
b=140	1 500	426	403	367	994	520	1 300	L_u 1 930	4 000	142	9 000	44.6
t=11.2	1 050	390	368	334	696	471	1 260	I_x 156	5 000	99.9	10 000	39.2
d=403	600	339	319	295	398	391	1 190	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 400	422	390	351	1 350	497	1 140	M_r 197	2 000	193	6 000	55.2
W16X26	1 950	409	381	346	1 290	477	1 120	V_r 448	3 000	155	7 000	44.1
b=140	1 500	376	353	321	994	450	1 100	L_u 1 860	4 000	105	8 000	36.6
t=8.8	1 050	342	322	290	696	410	1 070	I_x 127	4 500	86.7	9 000	31.3
d=399	600	295	276	253	398	343	1 010	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 460	654	615	568	1 630	749	2 100	M_r 386	4 000	364	8 000	225
W14X53	1 980	616	584	541	1 310	705	2 070	V_r 593	5 000	331	8 500	209
b=205	1 510	578	551	510	1 000	649	2 010	L_u 3 270	6 000	298	9 000	195
t=16.8	1 030	535	508	474	683	569	1 930	I_x 227	7 000	264	10 000	172
d=354	550	473	454	432	365	453	1 780	S_x 1 280	7 500	244	11 000	154
W360X72	2 460	604	566	520	1 630	683	1 900	M_r 346	4 000	322	8 000	186
W14X48	1 980	567	535	496	1 310	645	1 870	V_r 536	5 000	290	8 500	172
b=204	1 510	529	505	466	1 000	595	1 820	L_u 3 190	6 000	257	9 000	161
t=15.1	1 030	489	465	431	683	524	1 750	I_x 201	7 000	222	10 000	141
d=350	550	431	412	391	365	418	1 620	S_x 1 150	7 500	203	11 000	126
W360X64	2 460	557	520	474	1 630	621	1 710	M_r 308	4 000	283	8 000	153
W14X43	1 980	520	489	452	1 310	588	1 680	V_r 476	5 000	252	8 500	141
b=203	1 500	483	459	425	994	544	1 640	L_u 3 110	6 000	220	9 000	131
t=13.5	1 030	445	423	392	683	482	1 580	I_x 178	7 000	183	10 000	115
d=347	550	391	373	352	365	385	1 460	S_x 1 030	7 500	167	11 000	102
W360X57	2 430	523	487	442	1 610	584	1 540	M_r 273	3 000	259	7 000	119
W14X38	1 960	488	457	420	1 300	555	1 510	V_r 504	4 000	225	7 500	109
b=172	1 490	451	427	393	988	515	1 480	L_u 2 550	5 000	189	8 000	99.8
t=13.1	1 020	413	391	360	676	458	1 430	I_x 161	6 000	147	9 000	86.0
d=358	550	360	342	321	365	368	1 320	S_x 897	6 500	132	10 000	75.7
W360X51	2 430	484	448	405	1 610	528	1 370	M_r 241	3 000	227	7 000	97.0
W14X34	1 960	449	420	384	1 300	503	1 360	V_r 455	4 000	195	7 500	88.3
b=171	1 490	413	390	358	988	469	1 330	L_u 2 500	5 000	159	8 000	81.0
t=11.6	1 020	376	357	327	676	419	1 280	I_x 141	6 000	121	9 000	69.5
d=355	550	327	310	289	365	338	1 190	S_x 796	6 500	108	10 000	60.9
W360X45	2 430	442	408	366	1 550	473	1 220	M_r 210	3 000	195	7 000	76.5
W14X30	1 960	413	384	349	1 300	452	1 200	V_r 433	4 000	165	7 500	69.4
b=171	1 490	378	355	325	988	423	1 180	L_u 2 430	5 000	128	8 000	63.4
t=9.8	1 020	342	323	295	676	380	1 140	I_x 122	6 000	96.1	9 000	54.1
d=352	550	294	278	258	365	308	1 060	S_x 691	6 500	85.3	10 000	47.2

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm, d – mm, M_r – kN·m, V_r – kN, L_u – mm, I_x – 10^6mm^4, S_x – 10^3mm^3



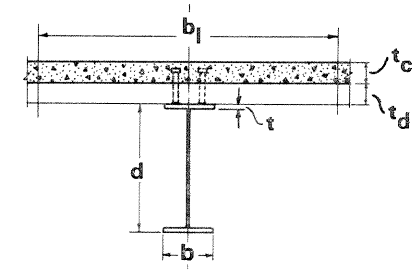
300W
20 MPa

76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W360X39	2 380	390	358	320	1 340	418	1 060	M_r 179	2 000	173	5 500	61.1
W14X26	1 930	376	348	314	1 280	401	1 040	V_r 409	3 000	139	6 000	54.2
b=128	1 470	342	320	291	975	377	1 030	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	1 010	307	289	262	670	340	993	I_x 102	4 500	81.3	8 000	37.5
d=353	550	262	246	226	365	278	928	S_x 580	5 000	69.8	9 000	32.5
W360X33	2 380	329	300	267	1 130	355	887	M_r 146	2 000	139	5 500	43.2
W14X22	1 920	323	297	265	1 130	341	876	V_r 361	3 000	108	6 000	38.1
b=127	1 470	302	281	254	975	323	861	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	1 010	268	252	227	670	294	836	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	227	212	193	365	243	786	S_x 474	5 000	49.7		
W310X129	2 560	870	826	773	1 700	924	3 110	M_r 583	6 000	573	8 500	515
W12X87	2 050	827	792	742	1 360	858	3 040	V_r 742	6 500	562	9 000	504
b=308	1 530	783	751	706	1 010	772	2 930	L_u 5 580	7 000	550	9 500	492
t=20.6	1 020	730	702	667	676	663	2 780	I_x 308	7 500	539	10 000	481
d=318	500	660	643	623	331	513	2 500	S_x 1 940	8 000	527		
W310X118	2 560	803	761	710	1 700	853	2 840	M_r 526	6 000	513	8 500	455
W12X79	2 050	761	727	681	1 360	793	2 770	V_r 666	6 500	501	9 000	444
b=307	1 530	718	689	647	1 010	715	2 680	L_u 5 390	7 000	490	9 500	432
t=18.7	1 020	669	642	608	676	615	2 540	I_x 275	7 500	478	10 000	421
d=314	500	602	584	565	331	474	2 290	S_x 1 750	8 000	467		
W310X107	2 560	743	702	653	1 700	790	2 590	M_r 478	6 000	461	8 500	404
W12X72	2 050	702	669	626	1 360	736	2 540	V_r 604	6 500	450	9 000	393
b=306	1 530	660	633	594	1 010	666	2 460	L_u 5 220	7 000	438	9 500	381
t=17.0	1 020	614	589	557	676	574	2 340	I_x 248	7 500	427	10 000	370
d=311	500	550	534	515	331	441	2 110	S_x 1 590	8 000	415		
W310X86	2 510	635	595	547	1 660	672	2 130	M_r 383	5 000	367	7 500	309
W12X58	2 010	595	563	524	1 330	630	2 090	V_r 503	5 500	355	8 000	297
b=254	1 510	555	530	495	1 000	574	2 030	L_u 4 250	6 000	344	8 500	285
t=16.3	1 000	512	491	460	663	496	1 930	I_x 199	6 500	332	9 000	273
d=310	500	455	439	421	331	382	1 760	S_x 1 280	7 000	320	9 500	262
W310X79	2 510	594	555	508	1 660	619	1 950	M_r 346	5 000	327	7 500	270
W12X53	2 010	555	523	485	1 330	581	1 910	V_r 480	5 500	316	8 000	258
b=254	1 510	515	491	457	1 000	531	1 860	L_u 4 140	6 000	305	8 500	247
t=14.6	1 000	473	452	422	663	460	1 770	I_x 177	6 500	293	9 000	235
d=306	500	417	401	382	331	354	1 610	S_x 1 160	7 000	282	9 500	221
W310X74	2 460	570	532	485	1 630	593	1 830	M_r 321	4 000	307	7 500	221
W12X50	1 970	532	500	461	1 310	558	1 800	V_r 519	5 000	282	8 000	206
b=205	1 480	492	468	432	981	510	1 750	L_u 3 380	6 000	258	8 500	192
t=16.3	990	451	429	398	656	443	1 670	I_x 165	6 500	245	9 000	179
d=310	500	394	378	359	331	342	1 520	S_x 1 060	7 000	233	9 500	168

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21−M 300W Steel

Units: b−mm, t−mm; d−mm; M_r−kN·m, V_r−kN; L_u−mm; I_x−10^6mm^4, S_x−10^3mm^3



300W
20 MPa

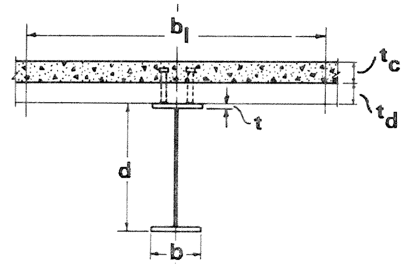
76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W310X67	2 460	526	489	443	1 630	538	1 650	M_r 286	4 000	270	7 500	184
W12X45	1 970	489	458	421	1 310	508	1 620	V_r 463	5 000	246	8 000	169
b=204	1 480	450	426	394	981	466	1 580	L_u 3 280	6 000	222	8 500	157
t=14.6	990	410	390	361	656	406	1 510	I_x 145	6 500	210	9 000	147
d=306	500	357	341	323	331	314	1 370	S_x 949	7 000	198	9 500	138
W310X60	2 460	487	450	405	1 630	489	1 480	M_r 254	4 000	237	7 500	151
W12X40	1 970	450	419	383	1 310	463	1 450	V_r 405	5 000	214	8 000	139
b=203	1 480	412	388	359	981	427	1 420	L_u 3 200	6 000	191	8 500	129
t=13.1	990	373	355	328	656	374	1 360	I_x 129	6 500	179	9 000	120
d=303	500	324	309	291	331	290	1 250	S_x 849	7 000	166	9 500	112
W310X52	2 420	459	424	380	1 600	464	1 330	M_r 226	3 000	216	6 500	116
W12X35	1 940	424	394	358	1 290	440	1 310	V_r 429	4 000	189	7 000	106
b=167	1 460	387	364	334	968	408	1 280	L_u 2 570	5 000	162	7 500	96.8
t=13.2	980	349	331	304	650	360	1 230	I_x 118	5 500	146	8 000	89.4
d=317	500	300	286	268	331	281	1 130	S_x 747	6 000	130	8 500	83.0
W310X45	2 420	409	375	333	1 540	403	1 140	M_r 191	3 000	180	6 500	87.8
W12X30	1 940	380	351	316	1 290	384	1 120	V_r 368	4 000	155	7 000	79.3
b=166	1 460	344	321	294	968	358	1 100	L_u 2 490	5 000	128	7 500	72.4
t=11.2	980	307	291	267	650	318	1 060	I_x 99.2	5 500	111	8 000	66.5
d=313	500	263	249	233	331	251	979	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 420	359	327	289	1 330	356	992	M_r 165	3 000	153	6 500	69.1
W12X26	1 940	346	318	284	1 290	340	979	V_r 320	4 000	130	7 000	62.2
b=165	1 460	311	289	263	968	319	961	L_u 2 440	5 000	103	7 500	56.5
t=9.7	980	275	260	238	650	286	930	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	234	222	206	331	227	863	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 510	628	587	538	1 660	598	2 210	M_r 378	5 000	377	7 500	338
W10X68	2 010	588	554	514	1 330	555	2 160	V_r 560	5 500	369	8 000	330
b=257	1 510	546	520	485	1 000	501	2 090	L_u 4 950	6 000	361	8 500	323
t=19.6	1 000	502	480	452	663	426	1 980	I_x 164	6 500	354		
d=264	500	446	431	414	331	323	1 770	S_x 1 240	7 000	346		
W250X89	2 510	573	534	486	1 660	540	1 970	M_r 332	5 000	327	7 500	289
W10X60	2 010	534	501	462	1 330	504	1 930	V_r 496	5 500	320	8 000	281
b=256	1 510	493	468	436	1 000	456	1 870	L_u 4 690	6 000	312	8 500	273
t=17.3	1 000	451	431	404	663	390	1 770	I_x 143	6 500	304		
d=260	500	398	384	368	331	295	1 590	S_x 1 100	7 000	296		
W250X80	2 510	529	490	443	1 660	491	1 770	M_r 294	5 000	287	7 500	249
W10X54	2 010	490	458	420	1 330	459	1 740	V_r 429	5 500	280	8 000	242
b=255	1 510	450	426	395	1 000	418	1 690	L_u 4 520	6 000	272	8 500	234
t=15.6	1 000	409	391	365	663	359	1 600	I_x 126	6 500	265		
d=256	500	360	347	330	331	271	1 450	S_x 982	7 000	257		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21−M 300W Steel

Units: b−mm, t−mm; d−mm; M_r−kN·m, V_r−kN; L_u−mm; I_x−10^6mm^4, S_x−10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.4



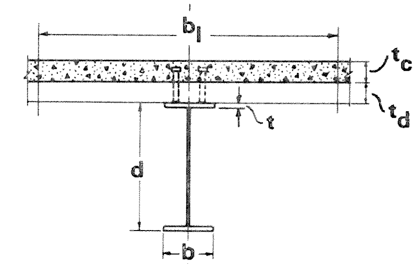
76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

300W
20 MPa

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for	10^6mm^4	10^3mm^3		L'	M_r'	L'	M_r'
		100%	75%	50%	100%				mm	kN·m	mm	kN·m
W250X73	2 510	495	457	411	1 660	453	1 620	M_r 266	5 000	257	7 500	220
W10X49	2 010	457	425	388	1 330	425	1 590	V_r 388	5 500	250	8 000	212
b=254	1 510	417	393	364	1 000	388	1 550	L_u 4 390	6 000	242	8 500	205
t=14.2	1 000	377	360	336	663	334	1 470	I_x 113	6 500	235		
d=253	500	330	317	302	331	253	1 330	S_x 891	7 000	227		
W250X67	2 460	472	434	388	1 630	429	1 500	M_r 243	4 000	237	6 500	197
W10X45	1 970	434	403	366	1 310	403	1 470	V_r 408	4 500	229	7 000	189
b=204	1 480	395	371	342	981	369	1 430	L_u 3 570	5 000	221	7 500	181
t=15.7	990	355	338	314	656	320	1 370	I_x 104	5 500	213	8 000	174
d=257	500	309	296	279	331	243	1 240	S_x 806	6 000	205		
W250X58	2 460	429	392	348	1 630	377	1 300	M_r 208	4 000	199	6 500	160
W10X39	1 970	392	362	326	1 310	356	1 280	V_r 359	4 500	191	7 000	153
b=203	1 480	354	331	303	981	327	1 250	L_u 3 410	5 000	184	7 500	145
t=13.5	990	315	299	276	656	285	1 200	I_x 87.3	5 500	176	8 000	137
d=252	500	272	259	243	331	218	1 090	S_x 693	6 000	168		
W250X49	2 460	385	350	307	1 630	322	1 100	M_r 171	4 000	160	6 500	123
W10X33	1 970	349	320	285	1 310	306	1 080	V_r 326	4 500	153	7 000	115
b=202	1 480	313	290	263	981	283	1 060	L_u 3 240	5 000	146	7 500	106
t=11.0	990	275	259	238	656	249	1 020	I_x 70.6	5 500	138	8 000	97.2
d=247	500	233	221	206	331	191	929	S_x 572	6 000	130		
W250X45	2 400	374	340	298	1 540	325	1 040	M_r 163	3 000	151	5 500	101
W10X30	1 930	344	315	280	1 280	309	1 020	V_r 360	3 500	142	6 000	90.6
b=148	1 450	308	285	258	961	287	1 000	L_u 2 360	4 000	132	6 500	82.2
t=13.0	980	271	255	233	650	254	963	I_x 71.1	4 500	122	7 000	75.2
d=266	500	229	216	201	331	198	884	S_x 534	5 000	112	7 500	69.3
W250X39	2 400	325	294	256	1 330	283	898	M_r 139	3 000	126	5 500	77.5
W10X26	1 930	313	285	251	1 280	271	885	V_r 308	3 500	117	6 000	69.2
b=147	1 450	277	256	229	961	253	867	L_u 2 280	4 000	108	6 500	62.5
t=11.2	980	242	226	206	650	226	837	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	202	191	176	331	178	773	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 400	278	249	216	1 130	243	761	M_r 114	3 000	102	5 500	55.6
W10X22	1 930	272	246	215	1 130	233	751	V_r 280	3 500	93.1	6 000	49.4
b=146	1 450	249	228	203	961	218	736	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	980	214	199	181	650	197	713	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	176	166	152	331	157	662	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 470	502	462	414	1 640	424	1 740	M_r 265	5 000	261	7 500	235
W8X58	1 970	462	429	390	1 310	394	1 700	V_r 514	5 500	256	8 000	230
b=209	1 480	421	396	364	981	354	1 640	L_u 4 620	6 000	251		
t=20.6	990	380	360	334	656	301	1 550	I_x 94.7	6 500	245		
d=222	500	329	316	301	331	223	1 370	S_x 853	7 000	240		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm t–mm d–mm M_r–kN·m V_r–kN L_u–mm I_x–10^6mm^4 S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.4

300W
20 MPa

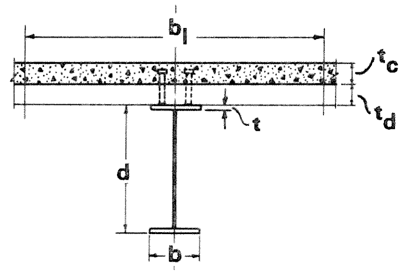
76 mm Deck with 65 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for	10^6mm^4	10^3mm^3		L'	M_r'	L'	M_r'
		100%	75%	50%	100%				mm	kN·m	mm	kN·m
W200X71	2 460	440	401	355	1 630	360	1 450	M_r 217	5 000	208	7 500	183
W8X48	1 970	401	370	332	1 310	337	1 420	V_r 393	5 500	203	8 000	178
b=206	1 480	362	338	309	981	306	1 380	L_u 4 150	6 000	198		
t=17.4	990	322	306	283	656	262	1 300	I_x 76.6	6 500	193		
d=216	500	278	266	252	331	195	1 160	S_x 709	7 000	188		
W200X59	2 460	391	354	309	1 630	305	1 210	M_r 176	4 000	174	6 500	149
W8X40	1 970	354	323	287	1 310	287	1 190	V_r 341	4 500	169	7 000	144
b=205	1 480	316	292	265	981	263	1 150	L_u 3 780	5 000	164	7 500	139
t=14.2	990	277	261	240	656	227	1 100	I_x 61.1	5 500	159		
d=210	500	235	224	210	331	170	991	S_x 582	6 000	154		
W200X52	2 460	362	326	283	1 630	273	1 070	M_r 154	4 000	150	6 500	126
W8X35	1 970	326	296	261	1 310	257	1 050	V_r 290	4 500	145	7 000	121
b=204	1 480	289	266	239	981	237	1 020	L_u 3 620	5 000	140	7 500	116
t=12.6	990	251	235	216	656	206	980	I_x 52.7	5 500	135		
d=206	500	211	200	187	331	156	888	S_x 512	6 000	131		
W200X46	2 460	334	299	257	1 580	244	945	M_r 134	4 000	129	6 500	105
W8X31	1 970	302	273	238	1 310	231	930	V_r 260	4 500	124	7 000	99.8
b=203	1 480	265	243	216	981	213	908	L_u 3 460	5 000	119	7 500	94.9
t=11.0	990	228	213	194	656	187	872	I_x 45.5	5 500	114		
d=203	500	189	179	167	331	142	794	S_x 448	6 000	109		
W200X42	2 420	307	274	235	1 430	226	861	M_r 120	3 000	119	5 500	93.5
W8X28	1 940	286	258	224	1 290	215	847	V_r 263	3 500	114	6 000	88.4
b=166	1 460	251	229	202	968	199	828	L_u 2 850	4 000	109	6 500	83.4
t=11.8	980	214	199	180	650	176	796	I_x 40.9	4 500	104	7 000	77.7
d=205	500	176	166	153	331	135	728	S_x 399	5 000	98.6		
W200X36	2 420	268	237	202	1 240	197	746	M_r 103	3 000	100	5 500	75.5
W8X24	1 940	260	233	200	1 240	188	734	V_r 222	3 500	95.3	6 000	70.6
b=165	1 460	229	207	181	968	175	719	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	980	193	178	160	650	156	693	I_x 34.4	4 500	85.5		
d=201	500	156	147	135	331	122	639	S_x 342	5 000	80.5		
W200X31	2 390	242	214	182	1 080	184	664	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	1 920	236	210	180	1 080	176	655	V_r 240	3 500	75.3	6 000	45.6
b=134	1 450	218	197	171	961	165	641	L_u 2 150	4 000	69.5		
t=10.2	970	182	167	149	643	148	620	I_x 31.4	4 500	63.6		
d=210	500	145	136	124	331	117	574	S_x 299	5 000	57.0		
W200X27	2 390	207	181	154	915	158	566	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	1 920	202	179	152	915	152	557	V_r 214	3 500	59.7	6 000	32.6
b=133	1 440	195	175	151	915	143	546	L_u 2 050	4 000	54.0		
t=8.4	970	164	150	132	643	129	529	I_x 25.8	4 500	47.5		
d=207	500	128	120	108	331	104	494	S_x 249	5 000	41.2		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm t–mm d–mm M_r–kN·m V_r–kN L_u–mm I_x–10^6mm^4 S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5



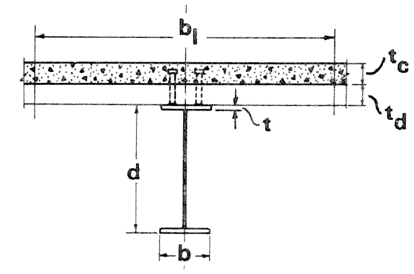
300W
20 MPa

76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam*: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
WWF700X151	2 960	2 120	2 050	1 960	2 720	4 300	6 810	M_r 1 480	6 000	1 330	14 000	485
	2 470	2 060	2 000	1 900	2 270	4 100	6 730	V_r 846	9 000	942	16 000	404
b=300	1 980	2 000	1 940	1 840	1 820	3 870	6 620	L_u 4 500	10 000	797	18 000	347
t=22.0	1 490	1 930	1 860	1 760	1 370	3 560	6 470	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 820	1 760	1 680	918	3 160	6 240	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 960	2 020	1 950	1 860	2 720	4 090	6 410	M_r 1 380	6 000	1 220	14 000	420
	2 470	1 960	1 900	1 800	2 270	3 910	6 340	V_r 846	9 000	831	16 000	348
b=300	1 980	1 900	1 840	1 740	1 820	3 680	6 240	L_u 4 420	10 000	700	18 000	297
t=20.0	1 490	1 830	1 760	1 660	1 370	3 400	6 090	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 720	1 660	1 570	918	3 010	5 880	S_x 4 620	12 000	527	22 000	230
W610X155	2 980	1 940	1 870	1 750	2 740	3 430	6 150	M_r 1 280	6 000	1 180	13 000	516
W24X104	2 460	1 870	1 810	1 680	2 260	3 250	6 060	V_r 1 380	9 000	886	14 000	465
b=324	1 940	1 800	1 730	1 610	1 780	3 040	5 930	L_u 4 740	10 000	762	16 000	388
t=19.0	1 420	1 700	1 630	1 530	1 300	2 760	5 760	I_x 1 290	11 000	659	18 000	333
d=611	900	1 570	1 510	1 440	826	2 380	5 480	S_x 4 220	12 000	579	20 000	291
W610X140	2 890	1 790	1 720	1 590	2 650	3 140	5 520	M_r 1 120	5 000	946	11 000	373
W24X94	2 390	1 730	1 660	1 530	2 190	2 990	5 440	V_r 1 440	6 000	829	13 000	303
b=230	1 890	1 650	1 570	1 450	1 740	2 790	5 320	L_u 3 320	7 000	695	15 000	255
t=22.2	1 400	1 550	1 480	1 380	1 290	2 540	5 160	I_x 1 120	8 000	573	17 000	221
d=617	900	1 420	1 360	1 290	826	2 200	4 910	S_x 3 630	9 000	486	19 000	195
W610X125	2 890	1 620	1 560	1 450	2 650	2 840	4 930	M_r 991	5 000	821	11 000	301
W24X84	2 390	1 560	1 500	1 390	2 190	2 710	4 860	V_r 1 300	6 000	708	13 000	243
b=229	1 890	1 500	1 430	1 320	1 740	2 540	4 760	L_u 3 250	7 000	575	15 000	204
t=19.6	1 400	1 410	1 340	1 240	1 290	2 320	4 630	I_x 985	8 000	470	17 000	176
d=612	900	1 280	1 230	1 160	826	2 010	4 400	S_x 3 220	9 000	396	19 000	155
W610X113	2 880	1 500	1 430	1 330	2 640	2 600	4 460	M_r 888	5 000	719	11 000	247
W24X76	2 390	1 440	1 380	1 270	2 190	2 480	4 400	V_r 1 210	6 000	610	13 000	198
b=228	1 890	1 370	1 310	1 210	1 740	2 330	4 320	L_u 3 180	7 000	481	15 000	166
t=17.3	1 400	1 290	1 230	1 130	1 290	2 140	4 200	I_x 875	8 000	391	17 000	142
d=608	900	1 180	1 120	1 050	826	1 850	4 000	S_x 2 880	9 000	328	19 000	125
W610X101	2 880	1 380	1 310	1 220	2 640	2 360	4 010	M_r 783	5 000	619	11 000	199
W24X68	2 390	1 320	1 260	1 160	2 190	2 260	3 960	V_r 1 130	6 000	512	13 000	158
b=228	1 890	1 260	1 200	1 100	1 740	2 120	3 880	L_u 3 110	7 000	396	15 000	132
t=14.9	1 400	1 180	1 120	1 030	1 290	1 950	3 780	I_x 764	8 000	320	17 000	113
d=603	900	1 070	1 010	945	826	1 690	3 610	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 870	1 460	1 390	1 290	2 630	2 320	4 450	M_r 867	4 000	794	9 000	361
W21X83	2 350	1 400	1 330	1 230	2 160	2 200	4 380	V_r 1 270	5 000	706	11 000	281
b=212	1 830	1 330	1 260	1 160	1 680	2 050	4 280	L_u 3 100	6 000	613	13 000	230
t=21.2	1 320	1 240	1 170	1 090	1 210	1 850	4 140	I_x 761	7 000	505	15 000	195
d=544	800	1 110	1 060	1 000	734	1 560	3 890	S_x 2 800	8 000	421	17 000	170

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5

300W
20 MPa

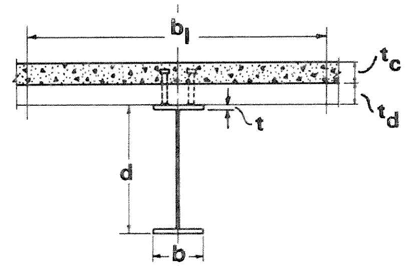
76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam*: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W530X109	2 870	1 320	1 260	1 170	2 630	2 090	3 960	M_r 764	4 000	692	9 000	291
W21X73	2 350	1 260	1 210	1 110	2 160	1 990	3 900	V_r 1 110	5 000	608	11 000	225
b=211	1 830	1 200	1 140	1 050	1 680	1 860	3 820	L_u 3 040	6 000	517	13 000	183
t=18.8	1 320	1 120	1 060	979	1 210	1 680	3 700	I_x 667	7 000	413	15 000	155
d=539	800	1 000	954	897	734	1 420	3 490	S_x 2 480	8 000	342	17 000	134
W530X101	2 870	1 250	1 190	1 100	2 630	1 970	3 690	M_r 707	4 000	635	9 000	255
W21X68	2 350	1 190	1 130	1 050	2 160	1 870	3 630	V_r 1 040	5 000	553	11 000	196
b=210	1 830	1 120	1 070	989	1 680	1 750	3 560	L_u 2 990	6 000	462	13 000	159
t=17.4	1 320	1 050	997	920	1 210	1 590	3 450	I_x 617	7 000	365	15 000	134
d=537	800	942	896	840	734	1 350	3 270	S_x 2 300	8 000	301	17 000	116
W530X92	2 870	1 170	1 100	1 020	2 630	1 810	3 370	M_r 637	3 000	633	8 000	253
W21X62	2 350	1 110	1 050	974	2 160	1 730	3 320	V_r 969	4 000	565	10 000	185
b=209	1 830	1 040	997	916	1 680	1 620	3 260	L_u 2 930	5 000	486	12 000	146
t=15.6	1 320	974	923	849	1 210	1 480	3 160	I_x 552	6 000	393	14 000	120
d=533	800	870	825	770	734	1 250	2 990	S_x 2 070	7 000	309	16 000	103
W530X82	2 870	1 070	1 010	927	2 630	1 620	2 990	M_r 559	3 000	551	8 000	204
W21X55	2 350	1 010	956	883	2 160	1 550	2 950	V_r 894	4 000	487	10 000	148
b=209	1 830	946	904	829	1 680	1 460	2 900	L_u 2 860	5 000	412	12 000	115
t=13.3	1 320	882	835	764	1 210	1 340	2 820	I_x 479	6 000	321	14 000	94.8
d=528	800	784	741	687	734	1 140	2 670	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 850	1 170	1 100	1 010	2 620	1 650	3 490	M_r 645	3 000	640	8 000	308
W18X71	2 310	1 100	1 040	963	2 120	1 560	3 430	V_r 1 050	4 000	579	9 000	266
b=194	1 780	1 030	984	903	1 630	1 450	3 350	L_u 2 910	5 000	512	11 000	210
t=20.6	1 240	954	903	833	1 140	1 290	3 230	I_x 488	6 000	444	13 000	174
d=469	700	840	801	755	643	1 060	3 000	S_x 2 080	7 000	366	15 000	148
W460X97	2 850	1 090	1 020	942	2 620	1 520	3 200	M_r 589	3 000	581	8 000	264
W18X65	2 310	1 020	970	895	2 120	1 450	3 150	V_r 947	4 000	522	9 000	227
b=193	1 770	957	913	839	1 620	1 350	3 080	L_u 2 870	5 000	457	11 000	178
t=19.0	1 240	886	839	774	1 140	1 210	2 970	I_x 445	6 000	389	13 000	147
d=466	700	780	742	697	643	992	2 770	S_x 1 910	7 000	314	15 000	125
W460X89	2 850	1 030	966	885	2 620	1 430	2 970	M_r 543	3 000	534	8 000	231
W18X60	2 310	965	912	842	2 120	1 360	2 930	V_r 866	4 000	477	9 000	198
b=192	1 770	899	857	789	1 620	1 270	2 870	L_u 2 830	5 000	414	11 000	155
t=17.7	1 240	831	789	726	1 140	1 140	2 770	I_x 410	6 000	343	13 000	127
d=463	700	732	696	651	643	938	2 590	S_x 1 770	7 000	276	15 000	108
W460X82	2 850	962	902	823	2 620	1 320	2 720	M_r 494	3 000	482	8 000	195
W18X55	2 310	900	849	782	2 120	1 260	2 680	V_r 812	4 000	427	9 000	167
b=191	1 770	836	795	733	1 620	1 170	2 630	L_u 2 770	5 000	365	11 000	129
t=16.0	1 240	770	732	672	1 140	1 060	2 540	I_x 370	6 000	292	13 000	106
d=460	700	677	643	600	643	876	2 380	S_x 1 610	7 000	234	15 000	90.0

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5



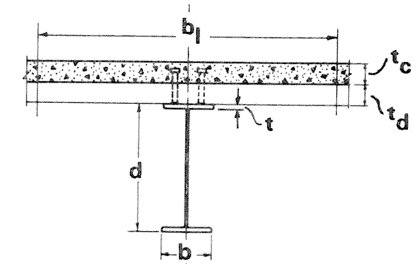
300W
20 MPa

76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W460X74	2 850	895	836	759	2 550	1 210	2 480	M_r 445	3 000	433	8 000	164
W18X50	2 310	839	789	724	2 120	1 160	2 440	V_r 733	4 000	380	9 000	140
b=190	1 770	775	736	679	1 620	1 080	2 400	L_u 2 730	5 000	320	10 000	122
t=14.5	1 240	711	678	622	1 140	983	2 320	I_x 333	6 000	249	12 000	96.9
d=457	700	626	593	551	643	816	2 190	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 850	827	769	696	2 340	1 120	2 270	M_r 405	3 000	390	8 000	140
W18X46	2 310	788	740	676	2 120	1 070	2 240	V_r 688	4 000	339	9 000	119
b=190	1 770	726	687	632	1 620	1 010	2 200	L_u 2 660	5 000	281	10 000	103
t=12.7	1 240	663	630	575	1 140	914	2 140	I_x 300	6 000	214	12 000	82.0
d=454	700	579	546	504	643	762	2 010	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 850	744	688	621	2 100	1 000	2 020	M_r 354	3 000	336	8 000	111
W18X41	2 310	726	679	617	2 100	961	2 000	V_r 650	4 000	288	9 000	93.9
b=189	1 770	667	629	576	1 620	907	1 960	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 240	605	575	522	1 140	828	1 910	I_x 259	6 000	172	12 000	64.1
d=450	700	525	494	453	643	693	1 800	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 840	923	862	782	2 610	1 170	2 640	M_r 467	3 000	455	8 000	205
W16X57	2 280	858	806	740	2 090	1 110	2 600	V_r 810	4 000	406	9 000	178
b=181	1 720	790	749	688	1 580	1 030	2 540	L_u 2 730	5 000	354	10 000	157
t=18.2	1 160	719	682	626	1 060	916	2 440	I_x 315	6 000	297	11 000	141
d=417	600	619	589	553	551	725	2 250	S_x 1 510	7 000	243	12 000	127
W410X74	2 840	846	787	709	2 580	1 050	2 340	M_r 408	3 000	394	8 000	163
W16X50	2 280	785	735	671	2 090	1 000	2 310	V_r 714	4 000	348	9 000	140
b=180	1 720	718	679	624	1 580	934	2 260	L_u 2 670	5 000	297	10 000	124
t=16.0	1 160	650	617	566	1 060	832	2 180	I_x 275	6 000	239	11 000	110
d=413	600	558	529	494	551	663	2 010	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 840	768	711	638	2 320	958	2 120	M_r 367	3 000	352	8 000	135
W16X45	2 280	729	681	618	2 090	916	2 090	V_r 643	4 000	307	9 000	116
b=179	1 720	664	626	575	1 580	857	2 050	L_u 2 610	5 000	258	10 000	102
t=14.4	1 160	597	568	520	1 060	768	1 980	I_x 246	6 000	201	11 000	90.5
d=410	600	512	485	451	551	615	1 830	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 830	684	629	563	2 050	858	1 880	M_r 321	3 000	306	8 000	109
W16X40	2 280	666	619	558	2 050	823	1 850	V_r 558	4 000	264	9 000	93.2
b=178	1 720	606	569	521	1 580	773	1 820	L_u 2 580	5 000	217	10 000	81.4
t=12.8	1 160	540	514	472	1 060	697	1 760	I_x 216	6 000	165	11 000	72.2
d=407	600	463	439	406	551	563	1 640	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 830	617	565	504	1 840	770	1 680	M_r 283	3 000	266	8 000	86.1
W16X36	2 270	603	557	500	1 840	740	1 650	V_r 539	4 000	225	9 000	73.2
b=177	1 720	561	525	478	1 580	698	1 620	L_u 2 480	5 000	176	10 000	63.6
t=10.9	1 160	496	470	430	1 060	632	1 580	I_x 186	6 000	132	11 000	56.3
d=403	600	421	397	365	551	513	1 470	S_x 924	7 000	104	12 000	50.5

Note: *20 MPa, 2300 kg/m³ Concrete. #G40·21–M 300W Steel

Units: b—mm, t—mm; d—mm; M_r—kN·m, V_r—kN; L_u—mm; I_x—10^6mm^4, S_x—10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5

300W
20 MPa

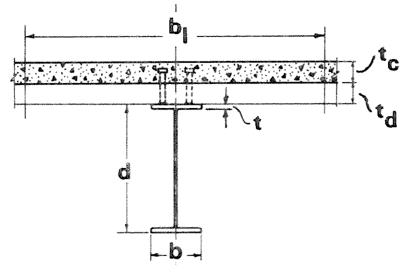
76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410X46	2 800	540	492	437	1 590	676	1 450	M_r 239	2 000	236	7 000	61.7
W16X31	2 250	529	486	434	1 590	651	1 430	V_r 503	3 000	195	8 000	51.8
b=140	1 700	509	474	428	1 560	616	1 410	L_u 1 930	4 000	142	9 000	44.6
t=11.2	1 150	446	421	383	1 060	562	1 370	I_x 156	5 000	99.9	10 000	39.2
d=403	600	375	352	321	551	462	1 280	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 800	461	418	370	1 350	577	1 230	M_r 197	2 000	193	6 000	55.2
W16X26	2 250	453	414	368	1 350	557	1 210	V_r 448	3 000	155	7 000	44.1
b=140	1 700	440	406	365	1 350	530	1 190	L_u 1 860	4 000	105	8 000	36.6
t=8.8	1 150	396	371	337	1 060	487	1 160	I_x 127	4 500	86.7	9 000	31.3
d=399	600	329	308	278	551	405	1 090	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 860	800	740	662	2 630	900	2 280	M_r 386	4 000	364	8 000	225
W14X53	2 280	734	684	620	2 090	854	2 240	V_r 593	5 000	331	8 500	209
b=205	1 710	667	628	578	1 570	792	2 190	L_u 3 270	6 000	298	9 000	195
t=16.8	1 130	595	568	524	1 040	697	2 110	I_x 227	7 000	264	10 000	172
d=354	550	509	485	456	505	537	1 930	S_x 1 280	7 500	244	11 000	154
W360X72	2 860	735	677	602	2 460	818	2 060	M_r 346	4 000	322	8 000	186
W14X48	2 280	683	634	572	2 090	778	2 030	V_r 536	5 000	290	8 500	172
b=204	1 710	617	579	530	1 570	724	1 980	L_u 3 190	6 000	257	9 000	161
t=15.1	1 130	547	520	480	1 040	641	1 910	I_x 201	7 000	222	10 000	141
d=350	550	465	443	414	505	496	1 750	S_x 1 150	7 500	203	11 000	126
W360X64	2 860	663	607	538	2 200	740	1 850	M_r 308	4 000	283	8 000	153
W14X43	2 280	635	587	525	2 090	706	1 820	V_r 476	5 000	252	8 500	141
b=203	1 700	568	531	484	1 560	659	1 780	L_u 3 110	6 000	220	9 000	131
t=13.5	1 130	501	475	438	1 040	588	1 720	I_x 178	7 000	183	10 000	115
d=347	550	424	403	375	505	458	1 590	S_x 1 030	7 500	167	11 000	102
W360X57	2 830	607	553	489	1 950	692	1 660	M_r 273	3 000	259	7 000	119
W14X38	2 260	590	544	485	1 950	662	1 640	V_r 504	4 000	225	7 500	109
b=172	1 690	536	499	453	1 550	621	1 610	L_u 2 550	5 000	189	8 000	99.8
t=13.1	1 120	469	443	406	1 030	556	1 550	I_x 161	6 000	147	9 000	86.0
d=358	550	393	372	344	505	438	1 440	S_x 897	6 500	132	10 000	75.7
W360X51	2 830	546	495	436	1 740	623	1 490	M_r 241	3 000	227	7 000	97.0
W14X34	2 260	532	488	433	1 740	598	1 470	V_r 455	4 000	195	7 500	88.3
b=171	1 690	497	461	415	1 550	562	1 440	L_u 2 500	5 000	159	8 000	81.0
t=11.6	1 120	431	406	372	1 030	507	1 390	I_x 141	6 000	121	9 000	69.5
d=355	550	359	339	312	505	402	1 290	S_x 796	6 500	108	10 000	60.9
W360X45	2 830	488	441	387	1 550	557	1 320	M_r 210	3 000	195	7 000	76.5
W14X30	2 260	477	435	384	1 550	536	1 300	V_r 433	4 000	165	7 500	69.4
b=171	1 690	460	425	380	1 550	506	1 280	L_u 2 430	5 000	128	8 000	63.4
t=9.8	1 120	395	371	338	1 030	458	1 240	I_x 122	6 000	96.1	9 000	54.1
d=352	550	325	306	280	505	367	1 150	S_x 691	6 500	85.3	10 000	47.2

Note: *20 MPa, 2300 kg/m³ Concrete. #G40·21–M 300W Steel

Units: b—mm, t—mm; d—mm; M_r—kN·m, V_r—kN; L_u—mm; I_x—10^6mm^4, S_x—10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5



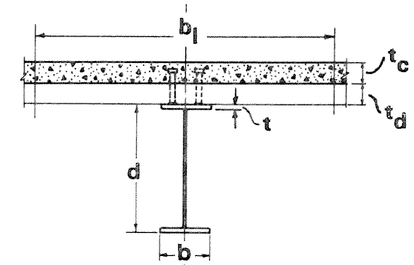
300W
20 MPa

76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W360X39	2 780	429	386	338	1 340	491	1 150	M_r 179	2 000	173	5 500	61.1
W14X26	2 230	421	382	336	1 340	474	1 130	V_r 409	3 000	139	6 000	54.2
b=128	1 670	407	374	332	1 340	449	1 110	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	1 110	360	336	304	1 020	409	1 080	I_x 102	4 500	81.3	8 000	37.5
d=353	550	292	274	248	505	332	1 010	S_x 580	5 000	69.8	9 000	32.5
W360X33	2 780	361	323	282	1 130	415	966	M_r 146	2 000	139	5 500	43.2
W14X22	2 220	355	320	281	1 130	401	952	V_r 361	3 000	108	6 000	38.1
b=127	1 670	346	315	278	1 130	382	935	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	1 110	320	297	267	1 020	351	908	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	255	238	214	505	289	853	S_x 474	5 000	49.7		
W310X129	2 960	1 030	960	874	2 720	1 140	3 410	M_r 583	6 000	573	8 500	515
W12X87	2 350	954	898	829	2 160	1 070	3 340	V_r 742	6 500	562	9 000	504
b=308	1 730	876	834	778	1 590	966	3 230	L_u 5 580	7 000	550	9 500	492
t=20.6	1 120	797	765	717	1 030	824	3 060	I_x 308	7 500	539	10 000	481
d=318	500	694	671	644	459	601	2 710	S_x 1 940	8 000	527		
W310X118	2 960	959	893	809	2 720	1 050	3 100	M_r 526	6 000	513	8 500	455
W12X79	2 350	886	832	764	2 160	985	3 040	V_r 666	6 500	501	9 000	444
b=307	1 730	810	769	716	1 590	894	2 950	L_u 5 390	7 000	490	9 500	432
t=18.7	1 120	733	703	658	1 030	765	2 800	I_x 275	7 500	478	10 000	421
d=314	500	634	613	586	459	558	2 490	S_x 1 750	8 000	467		
W310X107	2 960	897	832	750	2 720	969	2 830	M_r 478	6 000	461	8 500	404
W12X72	2 350	825	772	705	2 160	910	2 780	V_r 604	6 500	450	9 000	393
b=306	1 730	750	710	659	1 590	829	2 700	L_u 5 220	7 000	438	9 500	381
t=17.0	1 120	674	647	605	1 030	713	2 570	I_x 248	7 500	427	10 000	370
d=311	500	582	561	535	459	521	2 290	S_x 1 590	8 000	415		
W310X86	2 910	783	722	642	2 670	818	2 320	M_r 383	5 000	367	7 500	309
W12X58	2 310	714	664	599	2 120	773	2 280	V_r 503	5 500	355	8 000	297
b=254	1 710	643	604	556	1 570	711	2 220	L_u 4 250	6 000	344	8 500	285
t=16.3	1 100	569	543	505	1 010	615	2 120	I_x 199	6 500	332	9 000	273
d=310	500	485	466	441	459	456	1 910	S_x 1 280	7 000	320	9 500	262
W310X79	2 910	740	680	602	2 670	752	2 120	M_r 346	5 000	327	7 500	270
W12X53	2 310	672	623	559	2 120	712	2 090	V_r 480	5 500	316	8 000	258
b=254	1 710	602	564	516	1 570	656	2 040	L_u 4 140	6 000	305	8 500	247
t=14.6	1 100	529	503	467	1 010	571	1 950	I_x 177	6 500	293	9 000	235
d=306	500	447	427	403	459	424	1 760	S_x 1 160	7 000	282	9 500	221
W310X74	2 860	710	651	574	2 560	720	2 000	M_r 321	4 000	307	7 500	221
W12X50	2 270	648	599	536	2 080	682	1 970	V_r 519	5 000	282	8 000	206
b=205	1 680	579	541	493	1 540	630	1 920	L_u 3 380	6 000	258	8 500	192
t=16.3	1 090	507	482	444	1 000	550	1 840	I_x 165	6 500	245	9 000	179
d=310	500	424	404	379	459	410	1 660	S_x 1 060	7 000	233	9 500	168

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



300W
20 MPa

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5

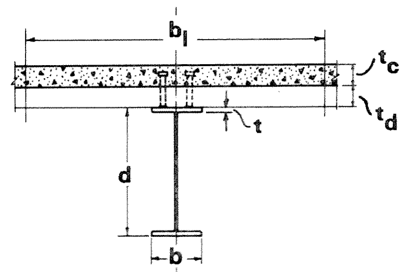
76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W310X67	2 860	642	585	514	2 300	650	1 800	M_r 286	4 000	270	7 500	184
W12X45	2 270	603	556	494	2 080	618	1 770	V_r 463	5 000	246	8 000	169
b=204	1 680	536	499	452	1 540	573	1 730	L_u 3 280	6 000	222	8 500	157
t=14.6	1 090	465	440	405	1 000	503	1 660	I_x 145	6 500	210	9 000	147
d=306	500	386	367	343	459	378	1 500	S_x 949	7 000	198	9 500	138
W310X60	2 860	579	524	458	2 050	588	1 610	M_r 254	4 000	237	7 500	151
W12X40	2 270	560	513	453	2 050	561	1 590	V_r 405	5 000	214	8 000	139
b=203	1 680	496	460	414	1 540	522	1 550	L_u 3 200	6 000	191	8 500	129
t=13.1	1 090	427	402	370	1 000	462	1 490	I_x 129	6 500	179	9 000	120
d=303	500	352	334	311	459	350	1 360	S_x 849	7 000	166	9 500	112
W310X52	2 820	528	477	416	1 800	554	1 450	M_r 226	3 000	216	6 500	116
W12X35	2 240	513	469	413	1 800	530	1 430	V_r 429	4 000	189	7 000	106
b=167	1 660	469	434	389	1 520	495	1 390	L_u 2 570	5 000	162	7 500	96.8
t=13.2	1 080	402	378	346	991	441	1 340	I_x 118	5 500	146	8 000	89.4
d=317	500	328	311	288	459	339	1 230	S_x 747	6 000	130	8 500	83.0
W310X45	2 820	454	408	354	1 540	479	1 240	M_r 191	3 000	180	6 500	87.8
W12X30	2 240	444	402	352	1 540	460	1 220	V_r 368	4 000	155	7 000	79.3
b=166	1 660	425	391	346	1 520	432	1 200	L_u 2 490	5 000	128	7 500	72.4
t=11.2	1 080	359	335	305	991	388	1 160	I_x 99.2	5 500	111	8 000	66.5
d=313	500	289	273	251	459	302	1 070	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 820	397	355	307	1 330	421	1 090	M_r 165	3 000	153	6 500	69.1
W12X26	2 240	389	350	305	1 330	405	1 070	V_r 320	4 000	130	7 000	62.2
b=165	1 660	376	343	302	1 330	383	1 050	L_u 2 440	5 000	103	7 500	56.5
t=9.7	1 080	326	303	274	991	346	1 010	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	258	244	224	459	273	940	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 910	781	717	635	2 670	747	2 450	M_r 378	5 000	377	7 500	338
W10X68	2 310	710	657	591	2 120	700	2 400	V_r 560	5 500	369	8 000	330
b=257	1 710	637	596	546	1 570	636	2 330	L_u 4 950	6 000	361	8 500	323
t=19.6	1 100	560	533	495	1 010	542	2 210	I_x 164	6 500	354		
d=264	500	475	457	433	459	390	1 950	S_x 1 240	7 000	346		
W250X89	2 910	723	661	581	2 670	671	2 180	M_r 332	5 000	327	7 500	289
W10X60	2 310	653	602	538	2 120	631	2 140	V_r 496	5 500	320	8 000	281
b=256	1 710	582	543	494	1 570	577	2 080	L_u 4 690	6 000	312	8 500	273
t=17.3	1 100	507	481	446	1 010	494	1 980	I_x 143	6 500	304		
d=260	500	427	409	386	459	358	1 760	S_x 1 100	7 000	296		
W250X80	2 910	676	615	537	2 670	607	1 960	M_r 294	5 000	287	7 500	249
W10X54	2 310	608	558	494	2 120	573	1 920	V_r 429	5 500	280	8 000	242
b=255	1 710	537	499	451	1 570	526	1 870	L_u 4 520	6 000	272	8 500	234
t=15.6	1 100	464	438	406	1 010	454	1 780	I_x 126	6 500	265		
d=256	500	387	370	349	459	331	1 600	S_x 982	7 000	257		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5



300W
20 MPa

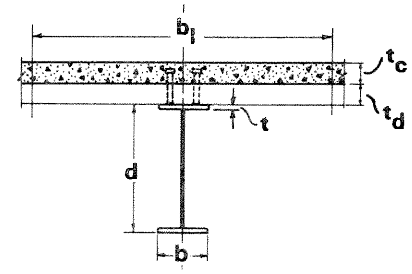
76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W250X73	2 910	627	568	493	2 510	558	1 790	M_r 266	5 000	257	7 500	220
W10X49	2 310	573	524	461	2 120	528	1 760	V_r 388	5 500	250	8 000	212
b=254	1 710	504	466	419	1 570	486	1 710	L_u 4 390	6 000	242	8 500	205
t=14.2	1 100	431	406	375	1 010	422	1 640	I_x 113	6 500	235		
d=253	500	356	341	320	459	309	1 470	S_x 891	7 000	227		
W250X67	2 860	589	531	459	2 310	528	1 660	M_r 243	4 000	237	6 500	197
W10X45	2 270	549	501	439	2 080	500	1 630	V_r 408	4 500	229	7 000	189
b=204	1 680	481	444	397	1 540	462	1 590	L_u 3 570	5 000	221	7 500	181
t=15.7	1 090	410	385	353	1 000	403	1 520	I_x 104	5 500	213	8 000	174
d=257	500	335	319	298	459	298	1 370	S_x 806	6 000	205		
W250X58	2 860	516	462	397	2 000	462	1 440	M_r 208	4 000	199	6 500	160
W10X39	2 270	498	452	392	2 000	439	1 420	V_r 359	4 500	191	7 000	153
b=203	1 680	438	402	356	1 540	408	1 380	L_u 3 410	5 000	184	7 500	145
t=13.5	1 090	369	345	314	1 000	359	1 330	I_x 87.3	5 500	176	8 000	137
d=252	500	297	282	261	459	268	1 200	S_x 693	6 000	168		
W250X49	2 860	440	390	333	1 690	393	1 220	M_r 171	4 000	160	6 500	123
W10X33	2 270	427	383	330	1 690	375	1 200	V_r 326	4 500	153	7 000	115
b=202	1 680	395	360	315	1 540	350	1 170	L_u 3 240	5 000	146	7 500	106
t=11.0	1 090	327	304	274	1 000	311	1 130	I_x 70.6	5 500	138	8 000	97.2
d=247	500	257	243	224	459	236	1 030	S_x 572	6 000	130		
W250X45	2 800	420	373	319	1 540	393	1 150	M_r 163	3 000	151	5 500	101
W10X30	2 230	409	367	317	1 540	376	1 130	V_r 360	3 500	142	6 000	90.6
b=148	1 650	389	355	310	1 510	352	1 100	L_u 2 360	4 000	132	6 500	82.2
t=13.0	1 080	323	300	270	991	315	1 060	I_x 71.1	4 500	122	7 000	75.2
d=266	500	253	239	219	459	242	974	S_x 534	5 000	112	7 500	69.3
W250X39	2 800	364	321	274	1 330	342	994	M_r 139	3 000	126	5 500	77.5
W10X26	2 230	356	317	272	1 330	328	978	V_r 308	3 500	117	6 000	69.2
b=147	1 650	342	309	268	1 330	309	956	L_u 2 280	4 000	108	6 500	62.5
t=11.2	1 080	293	270	241	991	279	924	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	225	212	193	459	217	851	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 800	310	272	231	1 130	292	845	M_r 114	3 000	102	5 500	55.6
W10X22	2 230	304	269	230	1 130	281	832	V_r 280	3 500	93.1	6 000	49.4
b=146	1 650	294	264	227	1 130	266	813	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	1 080	265	243	214	991	241	786	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	198	186	168	459	191	729	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 870	651	589	509	2 630	538	1 950	M_r 265	5 000	261	7 500	235
W8X58	2 270	581	530	465	2 080	504	1 910	V_r 514	5 500	256	8 000	230
b=209	1 680	510	471	421	1 540	459	1 850	L_u 4 620	6 000	251		
t=20.6	1 090	436	410	375	1 000	391	1 750	I_x 94.7	6 500	245		
d=222	500	357	340	319	459	278	1 540	S_x 853	7 000	240		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b–mm, t–mm, d–mm, M_r–kN·m, V_r–kN, L_u–mm, I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.5



300W
20 MPa

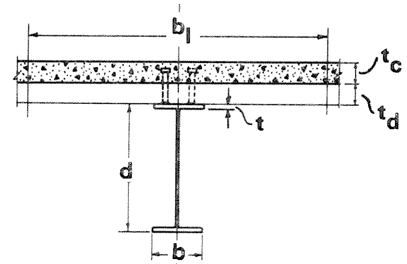
76 mm Deck with 90 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W200X71	2 860	570	512	437	2 460	453	1 620	M_r 217	5 000	208	7 500	183
W8X48	2 270	517	469	406	2 080	428	1 590	V_r 393	5 500	203	8 000	178
b=206	1 680	448	411	363	1 540	392	1 540	L_u 4 150	6 000	198		
t=17.4	1 090	377	352	320	1 000	338	1 470	I_x 76.6	6 500	193		
d=216	500	303	288	269	459	244	1 310	S_x 709	7 000	188		
W200X59	2 860	482	427	361	2 040	382	1 350	M_r 176	4 000	174	6 500	149
W8X40	2 270	463	417	357	2 040	362	1 330	V_r 341	4 500	169	7 000	144
b=205	1 680	400	364	318	1 540	334	1 290	L_u 3 780	5 000	164	7 500	139
t=14.2	1 090	331	306	275	1 000	292	1 240	I_x 61.1	5 500	159		
d=210	500	259	245	227	459	214	1 110	S_x 582	6 000	154		
W200X52	2 860	428	377	317	1 800	340	1 200	M_r 154	4 000	150	6 500	126
W8X35	2 270	414	369	313	1 800	323	1 180	V_r 290	4 500	145	7 000	121
b=204	1 680	372	336	291	1 540	300	1 150	L_u 3 620	5 000	140	7 500	116
t=12.6	1 090	303	279	249	1 000	263	1 100	I_x 52.7	5 500	135		
d=206	500	233	221	203	459	195	994	S_x 512	6 000	131		
W200X46	2 860	380	333	278	1 580	303	1 060	M_r 134	4 000	129	6 500	105
W8X31	2 270	369	326	276	1 580	288	1 040	V_r 260	4 500	124	7 000	99.8
b=203	1 680	347	313	268	1 540	269	1 020	L_u 3 460	5 000	119	7 500	94.9
t=11.0	1 090	280	257	227	1 000	238	977	I_x 45.5	5 500	114		
d=203	500	210	199	183	459	179	889	S_x 448	6 000	109		
W200X42	2 820	349	305	254	1 430	281	969	M_r 120	3 000	119	5 500	93.5
W8X28	2 240	340	299	252	1 430	268	951	V_r 263	3 500	114	6 000	88.4
b=166	1 660	324	291	248	1 430	250	927	L_u 2 850	4 000	109	6 500	83.4
t=11.8	1 080	266	243	213	991	223	892	I_x 40.9	4 500	104	7 000	77.7
d=205	500	197	186	169	459	170	815	S_x 399	5 000	98.6		
W200X36	2 820	303	263	218	1 240	244	841	M_r 103	3 000	100	5 500	75.5
W8X24	2 240	296	259	216	1 240	234	826	V_r 222	3 500	95.3	6 000	70.6
b=165	1 660	284	252	213	1 240	219	806	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	1 080	244	221	192	991	197	776	I_x 34.4	4 500	85.5		
d=201	500	176	165	150	459	152	713	S_x 342	5 000	80.5		
W200X31	2 790	272	236	196	1 080	227	748	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	2 220	267	233	195	1 080	218	736	V_r 240	3 500	75.3	6 000	45.6
b=134	1 650	258	228	192	1 080	205	719	L_u 2 150	4 000	69.5		
t=10.2	1 070	232	210	181	982	185	692	I_x 31.4	4 500	63.6		
d=210	500	166	155	140	459	146	639	S_x 299	5 000	57.0		
W200X27	2 790	232	200	166	915	194	637	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	2 220	228	198	165	915	187	628	V_r 214	3 500	59.7	6 000	32.6
b=133	1 640	222	194	163	915	177	614	L_u 2 050	4 000	54.0		
t=8.4	1 070	208	187	160	915	161	592	I_x 25.8	4 500	47.5		
d=207	500	149	138	123	459	129	549	S_x 249	5 000	41.2		

Note: *20 MPa, 2300 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b–mm, t–mm, d–mm, M_r–kN·m, V_r–kN, L_u–mm, I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6



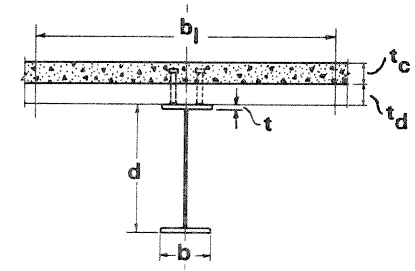
300W
25 MPa

76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
WWF700X151	2 720	2 090	2 020	1 930	2 600	4 050	6 660	M_r 1 480	6 000	1 330	14 000	485
	2 290	2 040	1 980	1 880	2 190	3 870	6 580	V_r 846	9 000	942	16 000	404
b=300	1 860	1 980	1 920	1 820	1 780	3 660	6 480	L_u 4 500	10 000	797	18 000	347
t=22.0	1 430	1 920	1 850	1 750	1 370	3 390	6 340	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 820	1 760	1 680	956	3 050	6 140	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 720	1 990	1 920	1 830	2 600	3 860	6 270	M_r 1 380	6 000	1 220	14 000	420
	2 290	1 940	1 880	1 780	2 190	3 690	6 190	V_r 846	9 000	831	16 000	348
b=300	1 860	1 880	1 830	1 720	1 780	3 480	6 100	L_u 4 420	10 000	700	18 000	297
t=20.0	1 430	1 820	1 750	1 650	1 370	3 230	5 970	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 720	1 660	1 580	956	2 910	5 780	S_x 4 620	12 000	527	22 000	230
W610X155	2 740	1 900	1 830	1 720	2 620	3 210	5 980	M_r 1 280	6 000	1 180	13 000	516
W24X104	2 280	1 840	1 780	1 660	2 180	3 050	5 890	V_r 1 380	9 000	886	14 000	465
b=324	1 820	1 780	1 710	1 590	1 740	2 860	5 780	L_u 4 740	10 000	762	16 000	388
t=19.0	1 360	1 690	1 620	1 520	1 300	2 610	5 620	I_x 1 290	11 000	659	18 000	333
d=611	900	1 580	1 510	1 440	861	2 290	5 380	S_x 4 220	12 000	579	20 000	291
W610X140	2 650	1 760	1 690	1 560	2 530	2 940	5 370	M_r 1 120	5 000	946	11 000	373
W24X94	2 210	1 700	1 630	1 500	2 110	2 800	5 280	V_r 1 440	6 000	829	13 000	303
b=230	1 770	1 630	1 550	1 440	1 690	2 620	5 180	L_u 3 320	7 000	695	15 000	255
t=22.2	1 340	1 540	1 470	1 370	1 280	2 400	5 030	I_x 1 120	8 000	573	17 000	221
d=617	900	1 430	1 360	1 290	861	2 110	4 800	S_x 3 630	9 000	486	19 000	195
W610X125	2 650	1 590	1 520	1 420	2 530	2 670	4 790	M_r 991	5 000	821	11 000	301
W24X84	2 210	1 530	1 470	1 360	2 110	2 540	4 720	V_r 1 300	6 000	708	13 000	243
b=229	1 770	1 480	1 410	1 300	1 690	2 380	4 630	L_u 3 250	7 000	575	15 000	204
t=19.6	1 340	1 400	1 330	1 230	1 280	2 190	4 510	I_x 985	8 000	470	17 000	176
d=612	900	1 290	1 230	1 160	861	1 930	4 310	S_x 3 220	9 000	396	19 000	155
W610X113	2 640	1 460	1 400	1 300	2 520	2 440	4 340	M_r 888	5 000	719	11 000	247
W24X76	2 210	1 410	1 350	1 250	2 110	2 330	4 280	V_r 1 210	6 000	610	13 000	198
b=228	1 770	1 350	1 290	1 190	1 690	2 190	4 200	L_u 3 180	7 000	481	15 000	166
t=17.3	1 340	1 280	1 220	1 130	1 280	2 020	4 090	I_x 875	8 000	391	17 000	142
d=608	900	1 180	1 120	1 050	861	1 780	3 920	S_x 2 880	9 000	328	19 000	125
W610X101	2 640	1 340	1 280	1 190	2 520	2 220	3 900	M_r 783	5 000	619	11 000	199
W24X68	2 210	1 290	1 240	1 140	2 110	2 120	3 850	V_r 1 130	6 000	512	13 000	158
b=228	1 770	1 240	1 180	1 080	1 690	2 000	3 780	L_u 3 110	7 000	396	15 000	132
t=14.9	1 340	1 170	1 110	1 020	1 280	1 840	3 680	I_x 764	8 000	320	17 000	113
d=603	900	1 070	1 010	946	861	1 620	3 530	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 630	1 430	1 360	1 260	2 510	2 170	4 310	M_r 867	4 000	794	9 000	361
W21X83	2 170	1 370	1 310	1 210	2 080	2 060	4 240	V_r 1 270	5 000	706	11 000	281
b=212	1 710	1 310	1 240	1 140	1 640	1 920	4 150	L_u 3 100	6 000	613	13 000	230
t=21.2	1 260	1 230	1 160	1 080	1 200	1 740	4 020	I_x 761	7 000	505	15 000	195
d=544	800	1 110	1 060	1 000	765	1 490	3 800	S_x 2 800	8 000	421	17 000	170

Note: *25 MPa, 2300 kg/m³ Concrete. #G40.21−M 300W Steel

Units: b−mm, t−mm; d−mm; M_r−kN·m, V_r−kN; L_u−mm; I_x−10^6mm^4, S_x−10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6

300W
25 MPa

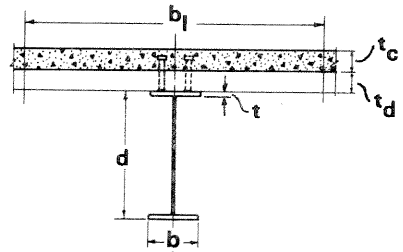
76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W530X109	2 630	1 290	1 230	1 140	2 510	1 960	3 840	M_r 764	4 000	692	9 000	291
W21X73	2 170	1 230	1 180	1 090	2 080	1 860	3 780	V_r 1 110	5 000	608	11 000	225
b=211	1 710	1 180	1 120	1 030	1 640	1 740	3 700	L_u 3 040	6 000	517	13 000	183
t=18.8	1 260	1 110	1 050	971	1 200	1 580	3 590	I_x 667	7 000	413	15 000	155
d=539	800	1 010	956	898	765	1 360	3 410	S_x 2 480	8 000	342	17 000	134
W530X101	2 630	1 220	1 160	1 070	2 510	1 840	3 580	M_r 707	4 000	635	9 000	255
W21X68	2 170	1 160	1 110	1 030	2 080	1 750	3 530	V_r 1 040	5 000	553	11 000	196
b=210	1 710	1 110	1 060	973	1 640	1 640	3 460	L_u 2 990	6 000	462	13 000	159
t=17.4	1 260	1 040	987	912	1 200	1 500	3 360	I_x 617	7 000	365	15 000	134
d=537	800	945	898	841	765	1 290	3 190	S_x 2 300	8 000	301	17 000	116
W530X92	2 630	1 130	1 070	996	2 510	1 700	3 270	M_r 637	3 000	633	8 000	253
W21X62	2 170	1 080	1 030	953	2 080	1 620	3 220	V_r 969	4 000	565	10 000	185
b=209	1 710	1 020	979	900	1 640	1 520	3 160	L_u 2 930	5 000	486	12 000	146
t=15.6	1 260	964	914	841	1 200	1 390	3 070	I_x 552	6 000	393	14 000	120
d=533	800	872	827	771	765	1 200	2 920	S_x 2 070	7 000	309	16 000	103
W530X82	2 630	1 030	976	903	2 510	1 520	2 900	M_r 559	3 000	551	8 000	204
W21X55	2 170	981	932	863	2 080	1 460	2 870	V_r 894	4 000	487	10 000	148
b=209	1 710	928	887	814	1 640	1 370	2 810	L_u 2 860	5 000	412	12 000	115
t=13.3	1 260	872	826	756	1 200	1 260	2 740	I_x 479	6 000	321	14 000	94.8
d=528	800	786	743	688	765	1 090	2 610	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 610	1 130	1 070	988	2 500	1 530	3 370	M_r 645	3 000	640	8 000	308
W18X71	2 130	1 070	1 020	941	2 040	1 450	3 320	V_r 1 050	4 000	579	9 000	266
b=194	1 660	1 010	966	888	1 590	1 350	3 240	L_u 2 910	5 000	512	11 000	210
t=20.6	1 180	944	893	826	1 130	1 210	3 120	I_x 488	6 000	444	13 000	174
d=469	700	843	803	755	669	1 010	2 920	S_x 2 080	7 000	366	15 000	148
W460X97	2 610	1 050	994	918	2 500	1 420	3 090	M_r 589	3 000	581	8 000	264
W18X65	2 130	997	946	875	2 040	1 350	3 050	V_r 947	4 000	522	9 000	227
b=193	1 650	938	895	824	1 580	1 260	2 980	L_u 2 870	5 000	457	11 000	178
t=19.0	1 180	875	830	766	1 130	1 130	2 880	I_x 445	6 000	389	13 000	147
d=466	700	782	744	698	669	944	2 700	S_x 1 910	7 000	314	15 000	125
W460X89	2 610	994	936	861	2 500	1 330	2 880	M_r 543	3 000	534	8 000	231
W18X60	2 130	939	889	823	2 040	1 270	2 830	V_r 866	4 000	477	9 000	198
b=192	1 650	881	840	774	1 580	1 180	2 770	L_u 2 830	5 000	414	11 000	155
t=17.7	1 180	821	779	718	1 130	1 070	2 680	I_x 410	6 000	343	13 000	127
d=463	700	733	696	652	669	892	2 520	S_x 1 770	7 000	276	15 000	108
W460X82	2 610	929	872	799	2 500	1 230	2 630	M_r 494	3 000	482	8 000	195
W18X55	2 130	874	826	763	2 040	1 170	2 590	V_r 812	4 000	427	9 000	167
b=191	1 650	818	778	718	1 580	1 100	2 540	L_u 2 770	5 000	365	11 000	129
t=16.0	1 180	760	723	665	1 130	991	2 460	I_x 370	6 000	292	13 000	106
d=460	700	678	644	600	669	832	2 320	S_x 1 610	7 000	234	15 000	90.0

Note: *25 MPa, 2300 kg/m³ Concrete. #G40.21−M 300W Steel

Units: b−mm, t−mm; d−mm; M_r−kN·m, V_r−kN; L_u−mm; I_x−10^6mm^4, S_x−10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6



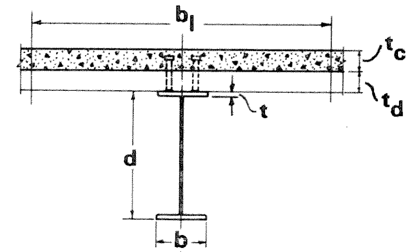
300W
25 MPa

76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W460X74	2 610	866	811	740	2 500	1 130	2 400	M_r 445	3 000	433	8 000	164
W18X50	2 130	813	766	706	2 040	1 080	2 360	V_r 733	4 000	380	9 000	140
b=190	1 650	757	719	665	1 580	1 010	2 320	L_u 2 730	5 000	320	10 000	122
t=14.5	1 180	701	668	614	1 130	920	2 250	I_x 333	6 000	249	12 000	96.9
d=457	700	626	593	551	669	775	2 130	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 610	803	749	682	2 340	1 040	2 200	M_r 405	3 000	390	8 000	140
W18X46	2 130	763	716	658	2 040	999	2 170	V_r 688	4 000	339	9 000	119
b=190	1 650	708	671	618	1 580	939	2 130	L_u 2 660	5 000	281	10 000	103
t=12.7	1 180	653	621	568	1 130	856	2 070	I_x 300	6 000	214	12 000	82.0
d=454	700	580	547	505	669	723	1 960	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 610	722	670	608	2 100	936	1 960	M_r 354	3 000	336	8 000	111
W18X41	2 130	703	658	600	2 040	899	1 930	V_r 650	4 000	288	9 000	93.9
b=189	1 650	649	613	563	1 580	847	1 900	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 180	595	565	514	1 130	775	1 850	I_x 259	6 000	172	12 000	64.1
d=450	700	526	494	453	669	658	1 750	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 600	889	831	758	2 490	1 090	2 540	M_r 467	3 000	455	8 000	205
W16X57	2 100	832	783	721	2 010	1 030	2 500	V_r 810	4 000	406	9 000	178
b=181	1 600	772	733	674	1 530	958	2 450	L_u 2 730	5 000	354	10 000	157
t=18.2	1 100	709	672	618	1 050	852	2 360	I_x 315	6 000	297	11 000	141
d=417	600	620	589	553	574	688	2 180	S_x 1 510	7 000	243	12 000	127
W410X74	2 600	815	759	688	2 490	978	2 260	M_r 408	3 000	394	8 000	163
W16X50	2 100	759	712	652	2 010	931	2 230	V_r 714	4 000	348	9 000	140
b=180	1 600	700	663	611	1 530	867	2 180	L_u 2 670	5 000	297	10 000	124
t=16.0	1 100	640	608	558	1 050	774	2 100	I_x 275	6 000	239	11 000	110
d=413	600	559	530	494	574	628	1 960	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 600	745	691	624	2 320	893	2 040	M_r 367	3 000	352	8 000	135
W16X45	2 100	703	658	600	2 010	852	2 010	V_r 643	4 000	307	9 000	116
b=179	1 600	646	610	562	1 530	796	1 970	L_u 2 610	5 000	258	10 000	102
t=14.4	1 100	587	559	513	1 050	715	1 910	I_x 246	6 000	201	11 000	90.5
d=410	600	513	485	451	574	583	1 780	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 590	662	611	550	2 050	800	1 810	M_r 321	3 000	306	8 000	109
W16X40	2 100	644	600	544	2 010	767	1 790	V_r 558	4 000	264	9 000	93.2
b=178	1 600	589	553	509	1 530	720	1 750	L_u 2 580	5 000	217	10 000	81.4
t=12.8	1 100	531	505	464	1 050	650	1 700	I_x 216	6 000	165	11 000	72.2
d=407	600	464	439	406	574	533	1 590	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 590	597	549	492	1 840	718	1 620	M_r 283	3 000	266	8 000	86.1
W16X36	2 090	585	542	489	1 840	689	1 600	V_r 539	4 000	225	9 000	73.2
b=177	1 600	544	509	466	1 530	650	1 570	L_u 2 480	5 000	176	10 000	63.6
t=10.9	1 100	487	462	422	1 050	589	1 520	I_x 186	6 000	132	11 000	56.3
d=403	600	421	397	365	574	485	1 430	S_x 924	7 000	104	12 000	50.5

Note: *25 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6

300W
25 MPa

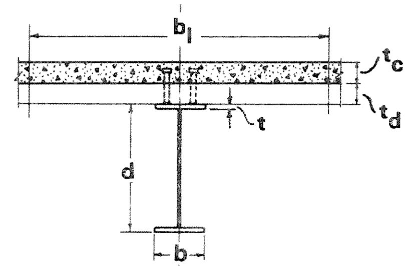
76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410X46	2 560	522	478	427	1 590	631	1 400	M_r 239	2 000	236	7 000	61.7
W16X31	2 070	513	473	424	1 590	607	1 380	V_r 503	3 000	195	8 000	51.8
b=140	1 580	492	458	416	1 510	574	1 360	L_u 1 930	4 000	142	9 000	44.6
t=11.2	1 090	437	412	376	1 040	524	1 320	I_x 156	5 000	99.9	10 000	39.2
d=403	600	375	352	321	574	436	1 240	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 560	444	405	361	1 350	539	1 180	M_r 197	2 000	193	6 000	55.2
W16X26	2 070	438	401	359	1 350	520	1 170	V_r 448	3 000	155	7 000	44.1
b=140	1 580	427	395	356	1 350	494	1 150	L_u 1 860	4 000	105	8 000	36.6
t=8.8	1 090	386	363	330	1 040	454	1 120	I_x 127	4 500	86.7	9 000	31.3
d=399	600	328	307	278	574	382	1 060	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 620	767	710	639	2 510	833	2 190	M_r 386	4 000	364	8 000	225
W14X53	2 100	708	661	602	2 010	789	2 160	V_r 593	5 000	331	8 500	209
b=205	1 590	649	611	565	1 520	731	2 110	L_u 3 270	6 000	298	9 000	195
t=16.8	1 070	586	559	516	1 020	645	2 030	I_x 227	7 000	264	10 000	172
d=354	550	510	486	456	526	507	1 870	S_x 1 280	7 500	244	11 000	154
W360X72	2 620	711	656	587	2 460	757	1 980	M_r 346	4 000	322	8 000	186
W14X48	2 100	657	611	554	2 010	719	1 950	V_r 536	5 000	290	8 500	172
b=204	1 590	599	563	518	1 520	669	1 910	L_u 3 190	6 000	257	9 000	161
t=15.1	1 070	537	512	473	1 020	593	1 840	I_x 201	7 000	222	10 000	141
d=350	550	466	443	414	526	468	1 700	S_x 1 150	7 500	203	11 000	126
W360X64	2 620	641	588	524	2 200	686	1 780	M_r 308	4 000	283	8 000	153
W14X43	2 100	609	564	508	2 010	654	1 750	V_r 476	5 000	252	8 500	141
b=203	1 580	551	515	472	1 510	609	1 710	L_u 3 110	6 000	220	9 000	131
t=13.5	1 070	491	466	431	1 020	544	1 660	I_x 178	7 000	183	10 000	115
d=347	550	424	403	375	526	432	1 540	S_x 1 030	7 500	167	11 000	102
W360X57	2 590	586	536	477	1 950	642	1 600	M_r 273	3 000	259	7 000	119
W14X38	2 080	572	528	473	1 950	614	1 580	V_r 504	4 000	225	7 500	109
b=172	1 570	518	484	440	1 500	575	1 550	L_u 2 550	5 000	189	8 000	99.8
t=13.1	1 060	459	435	399	1 010	515	1 490	I_x 161	6 000	147	9 000	86.0
d=358	550	393	372	344	526	413	1 390	S_x 897	6 500	132	10 000	75.7
W360X51	2 590	526	479	425	1 740	579	1 430	M_r 241	3 000	227	7 000	97.0
W14X34	2 080	515	473	422	1 740	555	1 410	V_r 455	4 000	195	7 500	88.3
b=171	1 570	479	445	403	1 500	521	1 380	L_u 2 500	5 000	159	8 000	81.0
t=11.6	1 060	421	397	365	1 010	470	1 340	I_x 141	6 000	121	9 000	69.5
d=355	550	358	338	312	526	379	1 250	S_x 796	6 500	108	10 000	60.9
W360X45	2 590	470	426	377	1 550	517	1 270	M_r 210	3 000	195	7 000	76.5
W14X30	2 080	461	421	375	1 550	497	1 250	V_r 433	4 000	165	7 500	69.4
b=171	1 570	443	410	368	1 500	469	1 230	L_u 2 430	5 000	128	8 000	63.4
t=9.8	1 060	386	362	331	1 010	425	1 190	I_x 122	6 000	96.1	9 000	54.1
d=352	550	325	306	280	526	345	1 120	S_x 691	6 500	85.3	10 000	47.2

Note: *25 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6



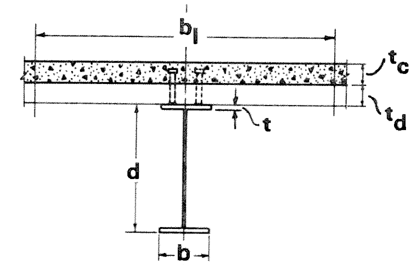
300W
25 MPa

76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b_1 (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W360X39	2 540	412	373	329	1 340	456	1 100	M_r 179	2 000	173	5 500	61.1
W14X26	2 050	406	369	327	1 340	440	1 090	V_r 409	3 000	139	6 000	54.2
b=128	1 550	395	363	324	1 340	416	1 070	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	1 050	350	327	297	1 000	380	1 040	I_x 102	4 500	81.3	8 000	37.5
d=353	550	291	273	248	526	312	974	S_x 580	5 000	69.8	9 000	32.5
W360X33	2 540	347	312	274	1 130	385	923	M_r 146	2 000	139	5 500	43.2
W14X22	2 040	342	310	273	1 130	372	911	V_r 361	3 000	108	6 000	38.1
b=127	1 550	334	305	271	1 130	355	897	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	1 050	310	288	260	1 000	326	873	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	253	238	214	526	272	823	S_x 474	5 000	49.7		
W310X129	2 720	993	929	850	2 600	1 050	3 260	M_r 583	6 000	573	8 500	515
W12X87	2 170	927	874	810	2 080	978	3 200	V_r 742	6 500	562	9 000	504
b=308	1 610	858	817	765	1 540	884	3 100	L_u 5 580	7 000	550	9 500	492
t=20.6	1 060	788	756	710	1 010	758	2 940	I_x 308	7 500	539	10 000	481
d=318	500	695	672	644	478	570	2 630	S_x 1 940	8 000	527		
W310X118	2 720	925	862	785	2 600	964	2 980	M_r 526	6 000	513	8 500	455
W12X79	2 170	860	808	746	2 080	902	2 920	V_r 666	6 500	501	9 000	444
b=307	1 610	792	752	703	1 540	818	2 830	L_u 5 390	7 000	490	9 500	432
t=18.7	1 060	723	694	650	1 010	703	2 690	I_x 275	7 500	478	10 000	421
d=314	500	635	613	586	478	528	2 410	S_x 1 750	8 000	467		
W310X107	2 720	863	802	726	2 600	889	2 710	M_r 478	6 000	461	8 500	404
W12X72	2 170	799	749	687	2 080	834	2 660	V_r 604	6 500	450	9 000	393
b=306	1 610	732	694	647	1 540	759	2 590	L_u 5 220	7 000	438	9 500	381
t=17.0	1 060	665	638	597	1 010	655	2 470	I_x 248	7 500	427	10 000	370
d=311	500	582	561	535	478	493	2 220	S_x 1 590	8 000	415		
W310X86	2 670	750	692	619	2 550	752	2 220	M_r 383	5 000	367	7 500	309
W12X58	2 130	688	640	581	2 040	710	2 180	V_r 503	5 500	355	8 000	297
b=254	1 590	625	588	543	1 520	652	2 130	L_u 4 250	6 000	344	8 500	285
t=16.3	1 040	559	534	498	994	566	2 040	I_x 199	6 500	332	9 000	273
d=310	500	486	466	441	478	429	1 850	S_x 1 280	7 000	320	9 500	262
W310X79	2 670	707	650	578	2 550	691	2 030	M_r 346	5 000	327	7 500	270
W12X53	2 130	647	600	541	2 040	654	2 000	V_r 480	5 500	316	8 000	258
b=254	1 590	584	548	504	1 520	602	1 950	L_u 4 140	6 000	305	8 500	247
t=14.6	1 040	519	495	460	994	524	1 870	I_x 177	6 500	293	9 000	235
d=306	500	447	427	402	478	398	1 700	S_x 1 160	7 000	282	9 500	221
W310X74	2 620	682	626	555	2 510	662	1 910	M_r 321	4 000	307	7 500	221
W12X50	2 090	623	576	518	2 000	627	1 880	V_r 519	5 000	282	8 000	206
b=205	1 560	561	525	481	1 490	578	1 840	L_u 3 380	6 000	258	8 500	192
t=16.3	1 030	498	473	436	985	505	1 760	I_x 165	6 500	245	9 000	179
d=310	500	424	404	379	478	385	1 600	S_x 1 060	7 000	233	9 500	168

Note: *25 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6

300W
25 MPa

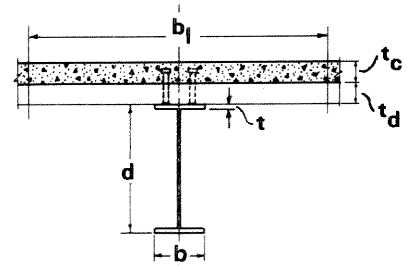
76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam* b_1 (mm)	Factored Resistances M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition L' mm	M_r' kN·m	L' mm	M_r' kN·m
W310X67	2 620	619	566	499	2 300	599	1 720	M_r 286	4 000	270	7 500	184
W12X45	2 090	578	533	476	2 000	568	1 690	V_r 463	5 000	246	8 000	169
b=204	1 560	518	483	439	1 490	526	1 650	L_u 3 280	6 000	222	8 500	157
t=14.6	1 030	456	431	398	985	463	1 590	I_x 145	6 500	210	9 000	147
d=306	500	386	367	343	478	355	1 450	S_x 949	7 000	198	9 500	138
W310X60	2 620	557	506	445	2 050	542	1 540	M_r 254	4 000	237	7 500	151
W12X40	2 090	537	493	438	2 000	516	1 520	V_r 405	5 000	214	8 000	139
b=203	1 560	478	444	401	1 490	480	1 490	L_u 3 200	6 000	191	8 500	129
t=13.1	1 030	417	393	363	985	425	1 430	I_x 129	6 500	179	9 000	120
d=303	500	351	334	310	478	328	1 320	S_x 849	7 000	166	9 500	112
W310X52	2 580	508	461	405	1 800	511	1 380	M_r 226	3 000	216	6 500	116
W12X35	2 060	496	454	401	1 800	488	1 360	V_r 429	4 000	189	7 000	106
b=167	1 540	452	418	377	1 470	456	1 340	L_u 2 570	5 000	162	7 500	96.8
t=13.2	1 020	392	369	339	975	407	1 290	I_x 118	5 500	146	8 000	89.4
d=317	500	328	311	287	478	317	1 190	S_x 747	6 000	130	8 500	83.0
W310X45	2 580	437	393	344	1 540	442	1 190	M_r 191	3 000	180	6 500	87.8
W12X30	2 060	427	388	342	1 540	424	1 170	V_r 368	4 000	155	7 000	79.3
b=166	1 540	408	375	334	1 470	399	1 150	L_u 2 490	5 000	128	7 500	72.4
t=11.2	1 020	349	327	299	975	358	1 110	I_x 99.2	5 500	111	8 000	66.5
d=313	500	288	273	251	478	283	1 030	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 580	381	342	298	1 330	389	1 030	M_r 165	3 000	153	6 500	69.1
W12X26	2 060	374	338	296	1 330	374	1 020	V_r 320	4 000	130	7 000	62.2
b=165	1 540	363	332	294	1 330	353	1 000	L_u 2 440	5 000	103	7 500	56.5
t=9.7	1 020	317	295	267	975	320	973	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	257	244	223	478	256	907	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 670	746	686	611	2 550	679	2 330	M_r 378	5 000	377	7 500	338
W10X68	2 130	683	634	573	2 040	635	2 290	V_r 560	5 500	369	8 000	330
b=257	1 590	618	580	534	1 520	577	2 220	L_u 4 950	6 000	361	8 500	323
t=19.6	1 040	550	524	488	994	493	2 100	I_x 164	6 500	354		
d=264	500	475	457	433	478	365	1 870	S_x 1 240	7 000	346		
W250X89	2 670	689	631	557	2 550	611	2 070	M_r 332	5 000	327	7 500	289
W10X60	2 130	627	579	519	2 040	574	2 030	V_r 496	5 500	320	8 000	281
b=256	1 590	564	527	481	1 520	524	1 980	L_u 4 690	6 000	312	8 500	273
t=17.3	1 040	497	472	439	994	450	1 880	I_x 143	6 500	304		
d=260	500	426	409	386	478	335	1 690	S_x 1 100	7 000	296		
W250X80	2 670	642	585	513	2 550	553	1 860	M_r 294	5 000	287	7 500	249
W10X54	2 130	582	535	476	2 040	522	1 830	V_r 429	5 500	280	8 000	242
b=255	1 590	520	483	439	1 520	478	1 780	L_u 4 520	6 000	272	8 500	234
t=15.6	1 040	454	430	399	994	413	1 700	I_x 126	6 500	265		
d=256	500	386	370	348	478	309	1 530	S_x 982	7 000	257		

Note: *25 MPa, 2300 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6



300W
25 MPa

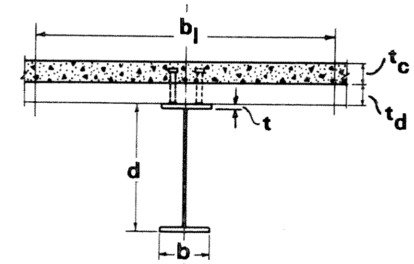
76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* — Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape — Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W250X73	2 670	603	547	477	2 510	508	1 700	M_r 266	5 000	257	7 500	220
W10X49	2 130	547	501	444	2 040	481	1 670	V_r 388	5 500	250	8 000	212
b=254	1 590	486	450	406	1 520	442	1 630	L_u 4 390	6 000	242	8 500	205
t=14.2	1 040	422	397	368	994	384	1 560	I_x 113	6 500	235		
d=253	500	356	340	319	478	288	1 410	S_x 891	7 000	227		
W250X67	2 620	565	512	445	2 310	481	1 580	M_r 243	4 000	237	6 500	197
W10X45	2 090	523	478	421	2 000	456	1 550	V_r 408	4 500	229	7 000	189
b=204	1 560	463	428	384	1 490	420	1 510	L_u 3 570	5 000	221	7 500	181
t=15.7	1 030	400	376	346	985	367	1 450	I_x 104	5 500	213	8 000	174
d=257	500	335	319	297	478	277	1 310	S_x 806	6 000	205		
W250X58	2 620	495	444	384	2 000	421	1 370	M_r 208	4 000	199	6 500	160
W10X39	2 090	479	435	380	2 000	400	1 350	V_r 359	4 500	191	7 000	153
b=203	1 560	421	386	344	1 490	371	1 320	L_u 3 410	5 000	184	7 500	145
t=13.5	1 030	359	336	307	985	327	1 260	I_x 87.3	5 500	176	8 000	137
d=252	500	296	281	261	478	249	1 150	S_x 693	6 000	168		
W250X49	2 620	421	375	322	1 690	358	1 150	M_r 171	4 000	160	6 500	123
W10X33	2 090	410	369	319	1 690	342	1 140	V_r 326	4 500	153	7 000	115
b=202	1 560	378	344	303	1 490	319	1 110	L_u 3 240	5 000	146	7 500	106
t=11.0	1 030	318	295	267	985	284	1 070	I_x 70.6	5 500	138	8 000	97.2
d=247	500	256	242	223	478	219	985	S_x 572	6 000	130		
W250X45	2 560	402	359	309	1 540	360	1 090	M_r 163	3 000	151	5 500	101
W10X30	2 050	393	354	307	1 540	344	1 070	V_r 360	3 500	142	6 000	90.6
b=148	1 530	371	339	298	1 460	322	1 050	L_u 2 360	4 000	132	6 500	82.2
t=13.0	1 020	314	291	263	975	289	1 020	I_x 71.1	4 500	122	7 000	75.2
d=266	500	252	238	218	478	225	935	S_x 534	5 000	112	7 500	69.3
W250X39	2 560	348	309	265	1 330	313	941	M_r 139	3 000	126	5 500	77.5
W10X26	2 050	341	305	263	1 330	300	928	V_r 308	3 500	117	6 000	69.2
b=147	1 530	329	298	260	1 330	282	910	L_u 2 280	4 000	108	6 500	62.5
t=11.2	1 020	284	262	234	975	255	881	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	224	211	193	478	202	817	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 560	296	261	223	1 130	267	799	M_r 114	3 000	102	5 500	55.6
W10X22	2 050	291	259	222	1 130	257	788	V_r 280	3 500	93.1	6 000	49.4
b=146	1 530	283	254	220	1 130	243	772	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	1 020	256	234	207	975	221	749	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	197	185	168	478	178	699	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 630	617	559	486	2 510	485	1 840	M_r 265	5 000	261	7 500	235
W8X58	2 090	555	507	447	2 000	454	1 800	V_r 514	5 500	256	8 000	230
b=209	1 560	492	455	409	1 490	412	1 750	L_u 4 620	6 000	251		
t=20.6	1 030	427	401	368	985	352	1 660	I_x 94.7	6 500	245		
d=222	500	356	339	318	478	257	1 470	S_x 853	7 000	240		

Note: *25 MPa, 2300 kg/m³ Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.6



300W
25 MPa

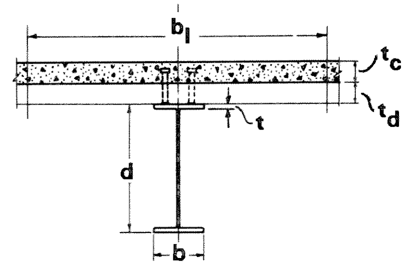
76 mm Deck with 75 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam* — Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape — Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W200X71	2 620	547	492	422	2 460	409	1 530	M_r 217	5 000	208	7 500	183
W8X48	2 090	492	446	388	2 000	385	1 500	V_r 393	5 500	203	8 000	178
b=206	1 560	431	395	351	1 490	353	1 460	L_u 4 150	6 000	198		
t=17.4	1 030	368	343	313	985	305	1 390	I_x 76.6	6 500	193		
d=216	500	302	288	268	478	225	1 250	S_x 709	7 000	188		
W200X59	2 620	460	409	348	2 040	345	1 280	M_r 176	4 000	174	6 500	149
W8X40	2 090	441	397	342	2 000	326	1 250	V_r 341	4 500	169	7 000	144
b=205	1 560	382	348	305	1 490	301	1 220	L_u 3 780	5 000	164	7 500	139
t=14.2	1 030	321	297	269	985	263	1 170	I_x 61.1	5 500	159		
d=210	500	258	244	226	478	197	1 060	S_x 582	6 000	154		
W200X52	2 620	408	361	305	1 800	306	1 130	M_r 154	4 000	150	6 500	126
W8X35	2 090	396	354	302	1 800	291	1 110	V_r 290	4 500	145	7 000	121
b=204	1 560	354	321	279	1 490	270	1 080	L_u 3 620	5 000	140	7 500	116
t=12.6	1 030	294	271	243	985	238	1 040	I_x 52.7	5 500	135		
d=206	500	232	220	202	478	180	948	S_x 512	6 000	131		
W200X46	2 620	362	318	268	1 580	273	998	M_r 134	4 000	129	6 500	105
W8X31	2 090	353	313	265	1 580	260	982	V_r 260	4 500	124	7 000	99.8
b=203	1 560	330	297	256	1 490	242	960	L_u 3 460	5 000	119	7 500	94.9
t=11.0	1 030	271	248	220	985	215	924	I_x 45.5	5 500	114		
d=203	500	209	198	182	478	164	847	S_x 448	6 000	109		
W200X42	2 580	332	291	245	1 430	253	910	M_r 120	3 000	119	5 500	93.5
W8X28	2 060	324	287	243	1 430	242	895	V_r 263	3 500	114	6 000	88.4
b=166	1 540	311	279	239	1 430	226	875	L_u 2 850	4 000	109	6 500	83.4
t=11.8	1 020	256	234	206	975	201	844	I_x 40.9	4 500	104	7 000	77.7
d=205	500	196	185	168	478	156	776	S_x 399	5 000	98.6		
W200X36	2 580	288	251	210	1 240	220	789	M_r 103	3 000	100	5 500	75.5
W8X24	2 060	282	247	208	1 240	211	776	V_r 222	3 500	95.3	6 000	70.6
b=165	1 540	272	242	206	1 240	198	759	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	1 020	235	213	186	975	178	734	I_x 34.4	4 500	85.5		
d=201	500	175	164	149	478	140	679	S_x 342	5 000	80.5		
W200X31	2 550	259	225	188	1 080	204	703	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	2 040	254	223	187	1 080	196	691	V_r 240	3 500	75.3	6 000	45.6
b=134	1 530	247	218	185	1 080	185	677	L_u 2 150	4 000	69.5		
t=10.2	1 010	223	202	175	966	168	655	I_x 31.4	4 500	63.6		
d=210	500	165	154	139	478	134	609	S_x 299	5 000	57.0		
W200X27	2 550	220	191	159	915	175	598	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	2 040	217	189	158	915	169	589	V_r 214	3 500	59.7	6 000	32.6
b=133	1 520	211	186	157	915	160	577	L_u 2 050	4 000	54.0		
t=8.4	1 010	200	180	154	915	146	559	I_x 25.8	4 500	47.5		
d=207	500	148	137	122	478	118	523	S_x 249	5 000	41.2		

Note: *25 MPa, 2300 kg/m³ Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7



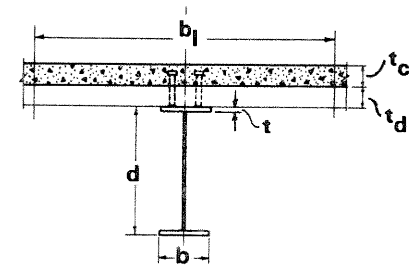
300W
25 MPa

51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*: b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L′ mm	M_r′ kN·m	L′ mm	M_r′ kN·m
WWF700X151	2 480	2 050	1 990	1 920	2 690	3 580	6 340	M_r 1 480	6 000	1 330	14 000	485
	2 110	2 000	1 960	1 870	2 290	3 430	6 260	V_r 846	9 000	942	16 000	404
b=300	1 740	1 960	1 910	1 820	1 890	3 250	6 170	L_u 4 500	10 000	797	18 000	347
t=22.0	1 370	1 910	1 850	1 760	1 480	3 040	6 050	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 840	1 780	1 690	1 080	2 780	5 900	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 480	1 950	1 890	1 820	2 690	3 410	5 960	M_r 1 380	6 000	1 220	14 000	420
	2 110	1 900	1 860	1 770	2 290	3 260	5 890	V_r 846	9 000	831	16 000	348
b=300	1 740	1 860	1 820	1 720	1 890	3 090	5 810	L_u 4 420	10 000	700	18 000	297
t=20.0	1 370	1 810	1 750	1 660	1 480	2 890	5 690	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 740	1 670	1 590	1 080	2 640	5 540	S_x 4 620	12 000	527	22 000	230
W610X155	2 500	1 860	1 810	1 710	2 710	2 810	5 650	M_r 1 280	6 000	1 180	13 000	516
W24X104	2 100	1 810	1 760	1 650	2 280	2 670	5 560	V_r 1 380	9 000	886	14 000	465
b=324	1 700	1 760	1 700	1 590	1 840	2 510	5 460	L_u 4 740	10 000	762	16 000	388
t=19.0	1 300	1 690	1 620	1 530	1 410	2 310	5 320	I_x 1 290	11 000	659	18 000	333
d=611	900	1 590	1 530	1 460	975	2 080	5 130	S_x 4 220	12 000	579	20 000	291
W610X140	2 410	1 720	1 660	1 550	2 610	2 560	5 050	M_r 1 120	5 000	946	11 000	373
W24X94	2 030	1 670	1 610	1 500	2 200	2 440	4 970	V_r 1 440	6 000	829	13 000	303
b=230	1 650	1 620	1 550	1 440	1 790	2 290	4 870	L_u 3 320	7 000	695	15 000	255
t=22.2	1 280	1 540	1 470	1 370	1 390	2 120	4 740	I_x 1 120	8 000	573	17 000	221
d=617	900	1 440	1 380	1 300	975	1 900	4 560	S_x 3 630	9 000	486	19 000	195
W610X125	2 410	1 550	1 500	1 400	2 610	2 330	4 510	M_r 991	5 000	821	11 000	301
W24X84	2 030	1 500	1 450	1 360	2 200	2 220	4 450	V_r 1 300	6 000	708	13 000	243
b=229	1 650	1 450	1 400	1 300	1 790	2 080	4 360	L_u 3 250	7 000	575	15 000	204
t=19.6	1 280	1 390	1 330	1 240	1 390	1 930	4 250	I_x 985	8 000	470	17 000	176
d=612	900	1 300	1 240	1 170	975	1 730	4 090	S_x 3 220	9 000	396	19 000	155
W610X113	2 400	1 420	1 370	1 290	2 600	2 130	4 090	M_r 888	5 000	719	11 000	247
W24X76	2 030	1 380	1 330	1 240	2 200	2 040	4 030	V_r 1 210	6 000	610	13 000	198
b=228	1 650	1 330	1 280	1 190	1 790	1 920	3 950	L_u 3 180	7 000	481	15 000	166
t=17.3	1 280	1 280	1 220	1 130	1 390	1 770	3 860	I_x 875	8 000	391	17 000	142
d=608	900	1 190	1 130	1 060	975	1 590	3 710	S_x 2 880	9 000	328	19 000	125
W610X101	2 400	1 300	1 250	1 180	2 600	1 940	3 670	M_r 783	5 000	619	11 000	199
W24X68	2 030	1 260	1 220	1 130	2 200	1 850	3 620	V_r 1 130	6 000	512	13 000	158
b=228	1 650	1 210	1 170	1 080	1 790	1 740	3 550	L_u 3 110	7 000	396	15 000	132
t=14.9	1 280	1 160	1 110	1 020	1 390	1 620	3 470	I_x 764	8 000	320	17 000	113
d=603	900	1 080	1 030	958	975	1 450	3 340	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 390	1 380	1 330	1 250	2 590	1 870	4 040	M_r 867	4 000	794	9 000	361
W21X83	1 990	1 340	1 290	1 200	2 160	1 780	3 970	V_r 1 270	5 000	706	11 000	281
b=212	1 590	1 290	1 230	1 140	1 720	1 660	3 880	L_u 3 100	6 000	613	13 000	230
t=21.2	1 200	1 220	1 160	1 080	1 300	1 520	3 760	I_x 761	7 000	505	15 000	195
d=544	800	1 130	1 070	1 010	867	1 330	3 590	S_x 2 800	8 000	421	17 000	170

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7

300W
25 MPa

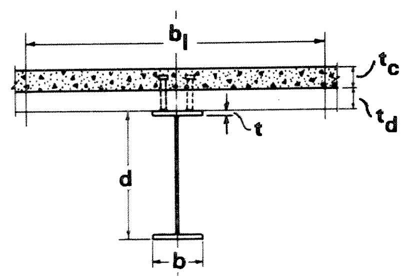
51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*: b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L′ mm	M_r′ kN·m	L′ mm	M_r′ kN·m
W530X109	2 390	1 250	1 200	1 130	2 590	1 700	3 600	M_r 764	4 000	692	9 000	291
W21X73	1 990	1 200	1 160	1 080	2 160	1 610	3 540	V_r 1 110	5 000	608	11 000	225
b=211	1 590	1 150	1 110	1 030	1 720	1 510	3 470	L_u 3 040	6 000	517	13 000	183
t=18.8	1 200	1 100	1 050	974	1 300	1 380	3 370	I_x 667	7 000	413	15 000	155
d=539	800	1 010	967	908	867	1 210	3 220	S_x 2 480	8 000	342	17 000	134
W530X101	2 390	1 170	1 130	1 060	2 590	1 600	3 350	M_r 707	4 000	635	9 000	255
W21X68	1 990	1 130	1 090	1 020	2 160	1 520	3 300	V_r 1 040	5 000	553	11 000	196
b=210	1 590	1 080	1 040	970	1 720	1 420	3 240	L_u 2 990	6 000	462	13 000	159
t=17.4	1 200	1 030	986	915	1 300	1 300	3 150	I_x 617	7 000	365	15 000	134
d=537	800	953	908	851	867	1 150	3 010	S_x 2 300	8 000	301	17 000	116
W530X92	2 390	1 090	1 040	980	2 590	1 470	3 060	M_r 637	3 000	633	8 000	253
W21X62	1 990	1 050	1 010	943	2 160	1 400	3 020	V_r 969	4 000	565	10 000	185
b=209	1 590	1 000	966	897	1 720	1 320	2 960	L_u 2 930	5 000	486	12 000	146
t=15.6	1 200	954	911	843	1 300	1 210	2 880	I_x 552	6 000	393	14 000	120
d=533	800	879	836	780	867	1 060	2 750	S_x 2 070	7 000	309	16 000	103
W530X82	2 390	990	947	886	2 590	1 330	2 720	M_r 559	3 000	551	8 000	204
W21X55	1 990	948	910	852	2 160	1 270	2 690	V_r 894	4 000	487	10 000	148
b=209	1 590	904	872	809	1 720	1 190	2 630	L_u 2 860	5 000	412	12 000	115
t=13.3	1 200	859	823	758	1 300	1 090	2 560	I_x 479	6 000	321	14 000	94.8
d=528	800	792	752	697	867	961	2 450	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 370	1 090	1 040	973	2 570	1 310	3 140	M_r 645	3 000	640	8 000	308
W18X71	1 950	1 040	1 000	931	2 110	1 240	3 080	V_r 1 050	4 000	579	9 000	266
b=194	1 540	992	954	884	1 670	1 150	3 010	L_u 2 910	5 000	512	11 000	210
t=20.6	1 120	935	891	827	1 210	1 040	2 900	I_x 488	6 000	444	13 000	174
d=469	700	849	811	763	759	889	2 740	S_x 2 080	7 000	366	15 000	148
W460X97	2 370	1 010	965	901	2 570	1 220	2 880	M_r 589	3 000	581	8 000	264
W18X65	1 950	964	924	864	2 110	1 160	2 830	V_r 947	4 000	522	9 000	227
b=193	1 530	916	882	819	1 660	1 070	2 770	L_u 2 870	5 000	457	11 000	178
t=19.0	1 120	865	827	767	1 210	972	2 680	I_x 445	6 000	389	13 000	147
d=466	700	787	751	705	759	832	2 530	S_x 1 910	7 000	314	15 000	125
W460X89	2 370	951	907	844	2 570	1 150	2 680	M_r 543	3 000	534	8 000	231
W18X60	1 950	906	867	811	2 110	1 090	2 640	V_r 866	4 000	477	9 000	198
b=192	1 530	858	826	769	1 660	1 010	2 580	L_u 2 830	5 000	414	11 000	155
t=17.7	1 120	809	775	719	1 210	917	2 500	I_x 410	6 000	343	13 000	127
d=463	700	738	704	659	759	785	2 360	S_x 1 770	7 000	276	15 000	108
W460X82	2 370	886	843	782	2 570	1 060	2 450	M_r 494	3 000	482	8 000	195
W18X55	1 950	841	804	751	2 110	1 010	2 420	V_r 812	4 000	427	9 000	167
b=191	1 530	794	763	712	1 660	939	2 360	L_u 2 770	5 000	365	11 000	129
t=16.0	1 120	747	718	665	1 210	853	2 290	I_x 370	6 000	292	13 000	106
d=460	700	682	650	607	759	731	2 170	S_x 1 610	7 000	234	15 000	90.0

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7



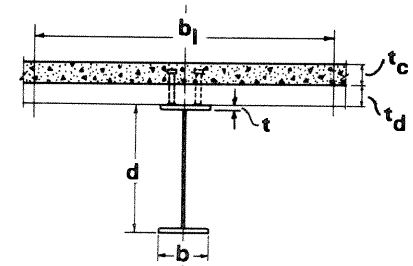
300W
25 MPa

51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam*: Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t $10^6 mm^4$	S_t $10^3 mm^3$	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W460X74	2 370	822	781	722	2 550	976	2 240	M_r 445	3 000	433	8 000	164
W18X50	1 950	780	744	693	2 110	929	2 200	V_r 733	4 000	380	9 000	140
b=190	1 530	734	704	658	1 660	869	2 160	L_u 2 730	5 000	320	10 000	122
t=14.5	1 120	688	662	614	1 210	791	2 100	I_x 333	6 000	249	12 000	96.9
d=457	700	629	599	558	759	679	1 990	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 370	760	718	662	2 340	903	2 050	M_r 405	3 000	390	8 000	140
W18X46	1 950	729	694	645	2 110	862	2 020	V_r 688	4 000	339	9 000	119
b=190	1 530	685	656	611	1 660	808	1 980	L_u 2 660	5 000	281	10 000	103
t=12.7	1 120	639	615	567	1 210	736	1 930	I_x 300	6 000	214	12 000	82.0
d=454	700	582	553	511	759	633	1 830	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 370	684	643	590	2 100	812	1 830	M_r 354	3 000	336	8 000	111
W18X41	1 950	668	634	586	2 100	776	1 800	V_r 650	4 000	288	9 000	93.9
b=189	1 530	626	597	556	1 660	729	1 770	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 120	581	559	514	1 210	667	1 720	I_x 259	6 000	172	12 000	64.1
d=450	700	528	500	459	759	574	1 630	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 360	846	803	741	2 560	930	2 360	M_r 467	3 000	455	8 000	205
W16X57	1 920	799	761	709	2 080	879	2 320	V_r 810	4 000	406	9 000	178
b=181	1 480	749	718	668	1 600	813	2 260	L_u 2 730	5 000	354	10 000	157
t=18.2	1 040	697	667	618	1 130	725	2 180	I_x 315	6 000	297	11 000	141
d=417	600	624	595	558	650	601	2 030	S_x 1 510	7 000	243	12 000	127
W410X74	2 360	772	730	671	2 560	838	2 100	M_r 408	3 000	394	8 000	163
W16X50	1 920	726	690	640	2 080	794	2 060	V_r 714	4 000	348	9 000	140
b=180	1 480	678	648	604	1 600	737	2 020	L_u 2 670	5 000	297	10 000	124
t=16.0	1 040	627	602	557	1 130	659	1 940	I_x 275	6 000	239	11 000	110
d=413	600	562	535	500	650	547	1 820	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 360	702	661	605	2 320	767	1 900	M_r 367	3 000	352	8 000	135
W16X45	1 920	670	636	587	2 080	729	1 870	V_r 643	4 000	307	9 000	116
b=179	1 480	623	595	554	1 600	678	1 830	L_u 2 610	5 000	258	10 000	102
t=14.4	1 040	574	552	511	1 130	609	1 770	I_x 246	6 000	201	11 000	90.5
d=410	600	515	490	456	650	507	1 660	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 350	625	585	533	2 050	689	1 680	M_r 321	3 000	306	8 000	109
W16X40	1 920	609	576	529	2 050	658	1 660	V_r 558	4 000	264	9 000	93.2
b=178	1 480	565	538	500	1 600	615	1 630	L_u 2 580	5 000	217	10 000	81.4
t=12.8	1 040	517	497	462	1 130	554	1 580	I_x 216	6 000	165	11 000	72.2
d=407	600	464	442	411	650	463	1 480	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 350	564	525	477	1 840	620	1 500	M_r 283	3 000	266	8 000	86.1
W16X36	1 910	551	518	474	1 840	592	1 480	V_r 539	4 000	225	9 000	73.2
b=177	1 480	520	494	457	1 600	555	1 450	L_u 2 480	5 000	176	10 000	63.6
t=10.9	1 040	473	453	420	1 130	502	1 410	I_x 186	6 000	132	11 000	56.3
d=403	600	421	400	370	650	420	1 330	S_x 924	7 000	104	12 000	50.5

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7

300W
25 MPa

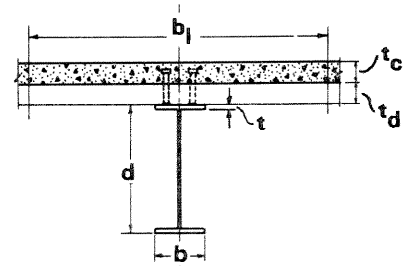
51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam*: Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t $10^6 mm^4$	S_t $10^3 mm^3$	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410X46	2 320	494	458	414	1 590	546	1 300	M_r 239	2 000	236	7 000	61.7
W16X31	1 890	484	452	411	1 590	523	1 280	V_r 503	3 000	195	8 000	51.8
b=140	1 460	468	443	407	1 580	492	1 260	L_u 1 930	4 000	142	9 000	44.6
t=11.2	1 030	423	404	373	1 120	447	1 220	I_x 156	5 000	99.9	10 000	39.2
d=403	600	374	355	325	650	377	1 150	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 320	421	388	350	1 350	468	1 100	M_r 197	2 000	193	6 000	55.2
W16X26	1 890	414	384	348	1 350	450	1 080	V_r 448	3 000	155	7 000	44.1
b=140	1 460	403	378	345	1 350	425	1 070	L_u 1 860	4 000	105	8 000	36.6
t=8.8	1 030	373	354	327	1 120	389	1 040	I_x 127	4 500	86.7	9 000	31.3
d=399	600	326	309	282	650	329	980	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 380	724	681	621	2 580	704	2 020	M_r 386	4 000	364	8 000	225
W14X53	1 920	675	639	589	2 080	664	1 990	V_r 593	5 000	331	8 500	209
b=205	1 470	626	596	557	1 590	613	1 940	L_u 3 270	6 000	298	9 000	195
t=16.8	1 010	573	552	514	1 090	543	1 870	I_x 227	7 000	264	10 000	172
d=354	550	511	489	460	596	439	1 730	S_x 1 280	7 500	244	11 000	154
W360X72	2 380	665	624	566	2 460	642	1 830	M_r 346	4 000	322	8 000	186
W14X48	1 920	624	589	541	2 080	607	1 800	V_r 536	5 000	290	8 500	172
b=204	1 470	576	548	509	1 590	562	1 760	L_u 3 190	6 000	257	9 000	161
t=15.1	1 010	524	504	470	1 090	499	1 690	I_x 201	7 000	222	10 000	141
d=350	550	466	446	418	596	404	1 570	S_x 1 150	7 500	203	11 000	126
W360X64	2 380	601	559	505	2 200	583	1 640	M_r 308	4 000	283	8 000	153
W14X43	1 920	576	542	495	2 080	553	1 620	V_r 476	5 000	252	8 500	141
b=203	1 460	528	500	463	1 580	513	1 580	L_u 3 110	6 000	220	9 000	131
t=13.5	1 010	478	458	428	1 090	458	1 520	I_x 178	7 000	183	10 000	115
d=347	550	424	405	379	596	372	1 420	S_x 1 030	7 500	167	11 000	102
W360X57	2 350	551	511	461	1 950	548	1 480	M_r 273	3 000	259	7 000	119
W14X38	1 900	536	502	457	1 950	521	1 460	V_r 504	4 000	225	7 500	109
b=172	1 450	495	468	432	1 570	486	1 430	L_u 2 550	5 000	189	8 000	99.8
t=13.1	1 000	446	427	396	1 080	434	1 380	I_x 161	6 000	147	9 000	86.0
d=358	550	393	374	347	596	355	1 280	S_x 897	6 500	132	10 000	75.7
W360X51	2 350	495	457	411	1 740	495	1 320	M_r 241	3 000	227	7 000	97.0
W14X34	1 900	483	451	408	1 740	472	1 300	V_r 455	4 000	195	7 500	88.3
b=171	1 450	456	430	394	1 570	441	1 280	L_u 2 500	5 000	159	8 000	81.0
t=11.6	1 000	408	389	361	1 080	396	1 230	I_x 141	6 000	121	9 000	69.5
d=355	550	357	340	315	596	325	1 150	S_x 796	6 500	108	10 000	60.9
W360X45	2 350	443	407	364	1 550	444	1 180	M_r 210	3 000	195	7 000	76.5
W14X30	1 900	433	401	362	1 550	424	1 160	V_r 433	4 000	165	7 500	69.4
b=171	1 450	418	393	358	1 550	398	1 140	L_u 2 430	5 000	128	8 000	63.4
t=9.8	1 000	372	354	328	1 080	359	1 100	I_x 122	6 000	96.1	9 000	54.1
d=352	550	323	307	283	596	295	1 030	S_x 691	6 500	85.3	10 000	47.2

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40·21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7



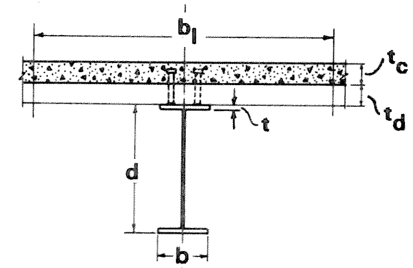
300W
25 MPa

51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W360X39	2 300	389	356	318	1 340	392	1 020	M_r 179	2 000	173	5 500	61.1
W14X26	1 870	382	352	316	1 340	376	1 000	V_r 409	3 000	139	6 000	54.2
b=128	1 430	371	346	313	1 340	354	985	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	990	337	319	294	1 070	321	955	I_x 102	4 500	81.3	8 000	37.5
d=353	550	289	275	251	596	266	897	S_x 580	5 000	69.8	9 000	32.5
W360X33	2 300	328	299	265	1 130	332	855	M_r 146	2 000	139	5 500	43.2
W14X22	1 860	323	296	264	1 130	320	843	V_r 361	3 000	108	6 000	38.1
b=127	1 430	315	291	262	1 130	303	828	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	990	297	280	255	1 070	277	804	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	250	238	217	596	232	758	S_x 474	5 000	49.7		
W310X129	2 480	950	900	833	2 690	868	2 980	M_r 583	6 000	573	8 500	515
W12X87	1 990	894	853	798	2 160	808	2 920	V_r 742	6 500	562	9 000	504
b=308	1 490	835	803	758	1 610	731	2 820	L_u 5 580	7 000	550	9 500	492
t=20.6	1 000	776	750	708	1 080	634	2 680	I_x 308	7 500	539	10 000	481
d=318	500	697	675	648	542	500	2 440	S_x 1 940	8 000	527		
W310X118	2 480	882	833	768	2 690	800	2 720	M_r 526	6 000	513	8 500	455
W12X79	1 990	827	786	733	2 160	746	2 660	V_r 666	6 500	501	9 000	444
b=307	1 490	769	738	695	1 610	676	2 580	L_u 5 390	7 000	490	9 500	432
t=18.7	1 000	711	687	648	1 080	587	2 460	I_x 275	7 500	478	10 000	421
d=314	500	637	616	589	542	461	2 240	S_x 1 750	8 000	467		
W310X107	2 480	819	773	709	2 690	740	2 490	M_r 478	6 000	461	8 500	404
W12X72	1 990	766	727	674	2 160	691	2 440	V_r 604	6 500	450	9 000	393
b=306	1 490	709	679	639	1 610	628	2 360	L_u 5 220	7 000	438	9 500	381
t=17.0	1 000	652	631	595	1 080	546	2 250	I_x 248	7 500	427	10 000	370
d=311	500	584	564	538	542	428	2 060	S_x 1 590	8 000	415		
W310X86	2 430	706	662	601	2 630	628	2 040	M_r 383	5 000	367	7 500	309
W12X58	1 950	655	618	568	2 110	590	2 000	V_r 503	5 500	355	8 000	297
b=254	1 470	602	573	535	1 590	540	1 950	L_u 4 250	6 000	344	8 500	285
t=16.3	980	546	526	496	1 060	470	1 860	I_x 199	6 500	332	9 000	273
d=310	500	486	468	444	542	370	1 710	S_x 1 280	7 000	320	9 500	262
W310X79	2 430	663	621	561	2 630	578	1 870	M_r 346	5 000	327	7 500	270
W12X53	1 950	613	578	528	2 110	544	1 830	V_r 480	5 500	316	8 000	258
b=254	1 470	561	533	495	1 590	499	1 780	L_u 4 140	6 000	305	8 500	247
t=14.6	980	506	487	457	1 060	435	1 710	I_x 177	6 500	293	9 000	235
d=306	500	447	430	405	542	342	1 570	S_x 1 160	7 000	282	9 500	221
W310X74	2 380	637	596	537	2 560	554	1 760	M_r 321	4 000	307	7 500	221
W12X50	1 910	590	554	505	2 070	521	1 720	V_r 519	5 000	282	8 000	206
b=205	1 440	539	510	472	1 560	479	1 680	L_u 3 380	6 000	258	8 500	192
t=16.3	970	485	465	434	1 050	419	1 610	I_x 165	6 500	245	9 000	179
d=310	500	425	407	382	542	330	1 470	S_x 1 060	7 000	233	9 500	168

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7

300W
25 MPa

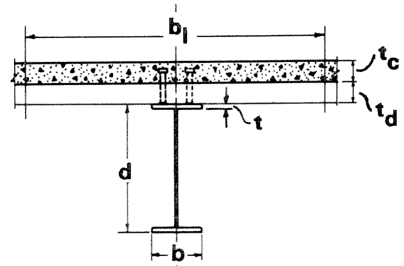
51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W310X67	2 380	577	535	480	2 300	502	1 580	M_r 286	4 000	270	7 500	184
W12X45	1 910	545	511	463	2 070	474	1 550	V_r 463	5 000	246	8 000	169
b=204	1 440	495	468	431	1 560	437	1 510	L_u 3 280	6 000	222	8 500	157
t=14.6	970	443	423	395	1 050	384	1 450	I_x 145	6 500	210	9 000	147
d=306	500	386	369	346	542	302	1 330	S_x 949	7 000	198	9 500	138
W310X60	2 380	520	479	428	2 050	456	1 420	M_r 254	4 000	237	7 500	151
W12X40	1 910	503	470	423	2 050	432	1 390	V_r 405	5 000	214	8 000	139
b=203	1 440	455	429	393	1 560	399	1 360	L_u 3 200	6 000	191	8 500	129
t=13.1	970	404	385	359	1 050	353	1 310	I_x 129	6 500	179	9 000	120
d=303	500	350	335	313	542	279	1 210	S_x 849	7 000	166	9 500	112
W310X52	2 340	476	437	390	1 800	433	1 270	M_r 226	3 000	216	6 500	116
W12X35	1 880	463	430	386	1 800	411	1 250	V_r 429	4 000	189	7 000	106
b=167	1 420	429	403	368	1 540	382	1 220	L_u 2 570	5 000	162	7 500	96.8
t=13.2	960	380	361	335	1 040	339	1 180	I_x 118	5 500	146	8 000	89.4
d=317	500	327	312	290	542	270	1 090	S_x 747	6 000	130	8 500	83.0
W310X45	2 340	410	374	332	1 540	376	1 090	M_r 191	3 000	180	6 500	87.8
W12X30	1 880	400	369	329	1 540	359	1 080	V_r 368	4 000	155	7 000	79.3
b=166	1 420	384	360	325	1 540	335	1 050	L_u 2 490	5 000	128	7 500	72.4
t=11.2	960	336	318	294	1 040	299	1 020	I_x 99.2	5 500	111	8 000	66.5
d=313	500	286	273	253	542	240	945	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 340	358	325	287	1 330	332	954	M_r 165	3 000	153	6 500	69.1
W12X26	1 880	351	321	286	1 330	317	940	V_r 320	4 000	130	7 000	62.2
b=165	1 420	339	315	283	1 330	298	921	L_u 2 440	5 000	103	7 500	56.5
t=9.7	960	304	286	263	1 040	268	892	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	254	244	225	542	217	832	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 430	703	657	594	2 630	554	2 110	M_r 378	5 000	377	7 500	338
W10X68	1 950	651	612	560	2 110	516	2 060	V_r 560	5 500	369	8 000	330
b=257	1 470	596	565	525	1 590	468	2 000	L_u 4 950	6 000	361	8 500	323
t=19.6	980	538	517	485	1 060	402	1 890	I_x 164	6 500	354		
d=264	500	476	459	436	542	310	1 710	S_x 1 240	7 000	346		
W250X89	2 430	646	601	540	2 630	500	1 880	M_r 332	5 000	327	7 500	289
W10X60	1 950	594	557	507	2 110	467	1 840	V_r 496	5 500	320	8 000	281
b=256	1 470	541	512	473	1 590	425	1 780	L_u 4 690	6 000	312	8 500	273
t=17.3	980	485	464	436	1 060	366	1 700	I_x 143	6 500	304		
d=260	500	426	410	388	542	283	1 540	S_x 1 100	7 000	296		
W250X80	2 430	599	556	496	2 630	454	1 690	M_r 294	5 000	287	7 500	249
W10X54	1 950	549	513	463	2 110	426	1 660	V_r 429	5 500	280	8 000	242
b=255	1 470	497	468	430	1 590	388	1 610	L_u 4 520	6 000	272	8 500	234
t=15.6	980	441	422	395	1 060	336	1 530	I_x 126	6 500	265		
d=256	500	385	371	351	542	260	1 390	S_x 982	7 000	257		

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7



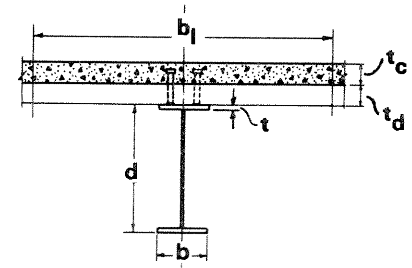
300W
25 MPa

51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*: b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections = 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W250X73	2 430	556	514	456	2 510	418	1 540	M_r 266	5 000	257	7 500	220
W10X49	1 950	514	479	431	2 110	393	1 510	V_r 388	5 500	250	8 000	212
b=254	1 470	463	435	398	1 590	360	1 470	L_u 4 390	6 000	242	8 500	205
t=14.2	980	409	389	364	1 060	312	1 410	I_x 113	6 500	235		
d=253	500	354	341	321	542	242	1 280	S_x 891	7 000	227		
W250X67	2 380	523	481	426	2 310	396	1 430	M_r 243	4 000	237	6 500	197
W10X45	1 910	490	456	408	2 070	373	1 400	V_r 408	4 500	229	7 000	189
b=204	1 440	440	413	376	1 560	342	1 370	L_u 3 570	5 000	221	7 500	181
t=15.7	970	388	368	342	1 050	299	1 310	I_x 104	5 500	213	8 000	174
d=257	500	333	320	299	542	232	1 190	S_x 806	6 000	205		
W250X58	2 380	459	418	368	2 000	348	1 240	M_r 208	4 000	199	6 500	160
W10X39	1 910	442	409	363	2 000	329	1 220	V_r 359	4 500	191	7 000	153
b=203	1 440	397	371	335	1 560	303	1 190	L_u 3 410	5 000	184	7 500	145
t=13.5	970	347	328	303	1 050	266	1 140	I_x 87.3	5 500	176	8 000	137
d=252	500	293	282	263	542	207	1 040	S_x 693	6 000	168		
W250X49	2 380	391	353	308	1 690	297	1 050	M_r 171	4 000	160	6 500	123
W10X33	1 910	379	347	305	1 690	282	1 030	V_r 326	4 500	153	7 000	115
b=202	1 440	354	329	294	1 560	261	1 010	L_u 3 240	5 000	146	7 500	106
t=11.0	970	305	287	263	1 050	231	968	I_x 70.6	5 500	138	8 000	97.2
d=247	500	253	242	225	542	181	890	S_x 572	6 000	130		
W250X45	2 320	375	339	297	1 540	300	994	M_r 163	3 000	151	5 500	101
W10X30	1 870	365	334	294	1 540	286	977	V_r 360	3 500	142	6 000	90.6
b=148	1 410	348	324	289	1 530	266	954	L_u 2 360	4 000	132	6 500	82.2
t=13.0	960	301	283	259	1 040	237	919	I_x 71.1	4 500	122	7 000	75.2
d=266	500	250	238	220	542	187	848	S_x 534	5 000	112	7 500	69.3
W250X39	2 320	325	292	254	1 330	262	860	M_r 139	3 000	126	5 500	77.5
W10X26	1 870	318	288	252	1 330	250	846	V_r 308	3 500	117	6 000	69.2
b=147	1 410	306	281	249	1 330	234	827	L_u 2 280	4 000	108	6 500	62.5
t=11.2	960	271	253	230	1 040	210	798	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	221	211	194	542	168	741	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 320	277	248	214	1 130	225	731	M_r 114	3 000	102	5 500	55.6
W10X22	1 870	272	245	213	1 130	215	719	V_r 280	3 500	93.1	6 000	49.4
b=146	1 410	263	240	211	1 130	202	703	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	960	242	226	203	1 040	182	680	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	194	184	169	542	147	633	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 390	574	530	468	2 590	388	1 650	M_r 265	5 000	261	7 500	235
W8X58	1 910	522	485	434	2 070	361	1 610	V_r 514	5 500	256	8 000	230
b=209	1 440	469	440	400	1 560	326	1 550	L_u 4 620	6 000	251		
t=20.6	970	414	393	364	1 050	279	1 470	I_x 94.7	6 500	245		
d=222	500	355	341	320	542	211	1 310	S_x 853	7 000	240		

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40·21—M 300W Steel

Units: b—mm, t—mm; d—mm; M_r—kN·m, V_r—kN; L_u—mm; I_x—10^6mm^4, S_x—10^3mm^3



COMPOSITE MEMBERS
Trial Selection Tables
Table 4.7

300W
25 MPa

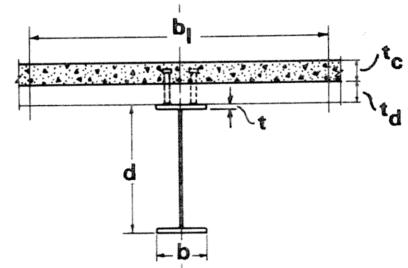
51 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	Composite Beam*: b_1 (mm)	Factored Resistances: M_{rc} (kN·m) for Shear Connections = 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L' mm	M_r' kN·m	L' mm	M_r' kN·m
W200X71	2 380	500	459	401	2 460	329	1 370	M_r 217	5 000	208	7 500	183
W8X48	1 910	459	424	375	2 070	308	1 340	V_r 393	5 500	203	8 000	178
b=206	1 440	408	380	343	1 560	281	1 300	L_u 4 150	6 000	198		
t=17.4	970	355	335	309	1 050	242	1 240	I_x 76.6	6 500	193		
d=216	500	300	288	270	542	184	1 110	S_x 709	7 000	188		
W200X59	2 380	423	383	331	2 040	279	1 150	M_r 176	4 000	174	6 500	149
W8X40	1 910	406	373	327	2 040	262	1 120	V_r 341	4 500	169	7 000	144
b=205	1 440	359	333	297	1 560	240	1 090	L_u 3 780	5 000	164	7 500	139
t=14.2	970	308	289	264	1 050	209	1 040	I_x 61.1	5 500	159		
d=210	500	255	244	227	542	160	944	S_x 582	6 000	154		
W200X52	2 380	376	338	291	1 800	249	1 020	M_r 154	4 000	150	6 500	126
W8X35	1 910	363	330	287	1 800	235	996	V_r 290	4 500	145	7 000	121
b=204	1 440	331	305	270	1 560	216	969	L_u 3 620	5 000	140	7 500	116
t=12.6	970	281	263	238	1 050	189	927	I_x 52.7	5 500	135		
d=206	500	229	219	204	542	146	845	S_x 512	6 000	131		
W200X46	2 380	335	298	255	1 580	222	900	M_r 134	4 000	129	6 500	105
W8X31	1 910	324	292	252	1 580	211	882	V_r 260	4 500	124	7 000	99.8
b=203	1 440	307	282	247	1 560	195	859	L_u 3 460	5 000	119	7 500	94.9
t=11.0	970	258	240	216	1 050	171	824	I_x 45.5	5 500	114		
d=203	500	206	197	182	542	133	755	S_x 448	6 000	109		
W200X42	2 340	307	273	233	1 430	207	820	M_r 120	3 000	119	5 500	93.5
W8X28	1 880	299	268	231	1 430	196	805	V_r 263	3 500	114	6 000	88.4
b=166	1 420	285	261	227	1 430	182	784	L_u 2 850	4 000	109	6 500	83.4
t=11.8	960	243	226	202	1 040	161	753	I_x 40.9	4 500	104	7 000	77.7
d=205	500	193	183	169	542	126	692	S_x 399	5 000	98.6		
W200X36	2 340	267	235	200	1 240	180	712	M_r 103	3 000	100	5 500	75.5
W8X24	1 880	261	232	198	1 240	172	699	V_r 222	3 500	95.3	6 000	70.6
b=165	1 420	250	226	196	1 240	160	681	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	960	221	204	181	1 040	143	656	I_x 34.4	4 500	85.5		
d=201	500	172	163	150	542	113	606	S_x 342	5 000	80.5		
W200X31	2 310	240	212	180	1 080	169	636	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	1 860	236	209	179	1 080	161	624	V_r 240	3 500	75.3	6 000	45.6
b=134	1 410	228	205	177	1 080	151	609	L_u 2 150	4 000	69.5		
t=10.2	950	210	193	170	1 030	135	587	I_x 31.4	4 500	63.6		
d=210	500	162	153	139	542	109	544	S_x 299	5 000	57.0		
W200X27	2 310	205	180	152	915	145	542	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	1 860	202	178	151	915	139	533	V_r 214	3 500	59.7	6 000	32.6
b=133	1 400	196	175	150	915	131	520	L_u 2 050	4 000	54.0		
t=8.4	950	185	168	147	915	118	502	I_x 25.8	4 500	47.5		
d=207	500	144	135	123	542	96.1	468	S_x 249	5 000	41.2		

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40·21—M 300W Steel

Units: b—mm, t—mm; d—mm; M_r—kN·m, V_r—kN; L_u—mm; I_x—10^6mm^4, S_x—10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

76 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

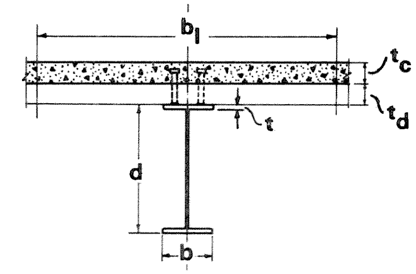
Steel Shape#	b_1 (mm)	M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	L' mm	M_r' kN·m	L' mm	M_r' kN·m
WWF700X151	2 880	2 170	2 090	1 990	3 120	3 950	6 650	M_r 1 480	6 000	1 330	14 000	485
	2 410	2 100	2 040	1 940	2 610	3 760	6 560	V_r 846	9 000	942	16 000	404
b=300	1 940	2 030	1 980	1 880	2 100	3 530	6 440	L_u 4 500	10 000	797	18 000	347
t=22.0	1 470	1 960	1 900	1 800	1 590	3 250	6 290	I_x 1 740	11 000	688	20 000	303
d=700	1 000	1 860	1 800	1 710	1 080	2 900	6 060	S_x 4 980	12 000	605	22 000	270
WWF700X141	2 880	2 070	1 990	1 890	3 120	3 760	6 260	M_r 1 380	6 000	1 220	14 000	420
	2 410	2 000	1 940	1 840	2 610	3 580	6 180	V_r 846	9 000	831	16 000	348
b=300	1 940	1 940	1 880	1 780	2 100	3 370	6 070	L_u 4 420	10 000	700	18 000	297
t=20.0	1 470	1 870	1 800	1 700	1 590	3 100	5 920	I_x 1 620	11 000	602	20 000	259
d=700	1 000	1 760	1 690	1 610	1 080	2 760	5 710	S_x 4 620	12 000	527	22 000	230
W610X155	2 900	1 980	1 900	1 790	3 140	3 130	5 970	M_r 1 280	6 000	1 180	13 000	516
W24X104	2 400	1 910	1 840	1 730	2 600	2 960	5 870	V_r 1 380	9 000	886	14 000	465
b=324	1 900	1 840	1 770	1 650	2 060	2 760	5 740	L_u 4 740	10 000	762	16 000	388
t=19.0	1 400	1 750	1 670	1 560	1 520	2 500	5 560	I_x 1 290	11 000	659	18 000	333
d=611	900	1 610	1 550	1 470	975	2 180	5 290	S_x 4 220	12 000	579	20 000	291
W610X140	2 810	1 840	1 760	1 640	3 050	2 870	5 360	M_r 1 120	5 000	946	11 000	373
W24X94	2 330	1 770	1 700	1 570	2 530	2 720	5 260	V_r 1 440	6 000	829	13 000	303
b=230	1 850	1 700	1 620	1 500	2 000	2 530	5 140	L_u 3 320	7 000	695	15 000	255
t=22.2	1 380	1 600	1 520	1 410	1 500	2 300	4 980	I_x 1 120	8 000	573	17 000	221
d=617	900	1 470	1 400	1 320	975	2 000	4 720	S_x 3 630	9 000	486	19 000	195
W610X125	2 810	1 670	1 590	1 490	3 050	2 610	4 790	M_r 991	5 000	821	11 000	301
W24X84	2 330	1 600	1 530	1 430	2 530	2 470	4 710	V_r 1 300	6 000	708	13 000	243
b=229	1 850	1 530	1 470	1 350	2 000	2 310	4 610	L_u 3 250	7 000	575	15 000	204
t=19.6	1 380	1 450	1 380	1 270	1 500	2 100	4 460	I_x 985	8 000	470	17 000	176
d=612	900	1 330	1 260	1 180	975	1 820	4 240	S_x 3 220	9 000	396	19 000	155
W610X113	2 800	1 540	1 470	1 370	3 030	2 390	4 340	M_r 888	5 000	719	11 000	247
W24X76	2 330	1 470	1 410	1 310	2 530	2 270	4 270	V_r 1 210	6 000	610	13 000	198
b=228	1 850	1 410	1 350	1 240	2 000	2 120	4 180	L_u 3 180	7 000	481	15 000	166
t=17.3	1 380	1 330	1 260	1 170	1 500	1 940	4 060	I_x 875	8 000	391	17 000	142
d=608	900	1 210	1 150	1 080	975	1 680	3 860	S_x 2 880	9 000	328	19 000	125
W610X101	2 800	1 420	1 350	1 250	3 030	2 180	3 900	M_r 783	5 000	619	11 000	199
W24X68	2 330	1 350	1 290	1 200	2 530	2 070	3 840	V_r 1 130	6 000	512	13 000	158
b=228	1 850	1 290	1 230	1 130	2 000	1 940	3 760	L_u 3 110	7 000	396	15 000	132
t=14.9	1 380	1 220	1 150	1 060	1 500	1 770	3 650	I_x 764	8 000	320	17 000	113
d=603	900	1 110	1 050	971	975	1 540	3 470	S_x 2 530	9 000	267	19 000	98.4
W530X123	2 790	1 500	1 430	1 330	3 020	2 120	4 310	M_r 867	4 000	794	9 000	361
W21X83	2 290	1 430	1 370	1 270	2 480	2 000	4 240	V_r 1 270	5 000	706	11 000	281
b=212	1 790	1 360	1 300	1 190	1 940	1 850	4 130	L_u 3 100	6 000	613	13 000	230
t=21.2	1 300	1 270	1 210	1 120	1 410	1 670	3 980	I_x 761	7 000	505	15 000	195
d=544	800	1 150	1 090	1 020	867	1 410	3 740	S_x 2 800	8 000	421	17 000	170

Composite Beam* columns: b_1 through S_t (Factored Resistances: M_{rc} and Q_r). Non-Composite Shape columns: Steel Shape Data and Unbraced Condition (L', M_r').

Note: *25 MPa, 1850 kg/m³ Concrete. #G40-21–M 300W Steel

Units: b–mm, t–mm, d–mm, M_r–kN·m, V_r–kN, L_u–mm, I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

76 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

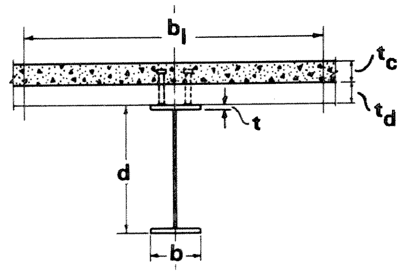
Steel Shape#	b_1 (mm)	M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	L' mm	M_r' kN·m	L' mm	M_r' kN·m
W530X109	2 790	1 370	1 290	1 200	3 020	1 920	3 840	M_r 764	4 000	692	9 000	291
W21X73	2 290	1 300	1 240	1 150	2 480	1 820	3 780	V_r 1 110	5 000	608	11 000	225
b=211	1 790	1 230	1 170	1 080	1 940	1 690	3 690	L_u 3 040	6 000	517	13 000	183
t=18.8	1 300	1 150	1 090	1 010	1 410	1 520	3 560	I_x 667	7 000	413	15 000	155
d=539	800	1 040	984	919	867	1 290	3 350	S_x 2 480	8 000	342	17 000	134
W530X101	2 790	1 290	1 220	1 130	3 020	1 810	3 580	M_r 707	4 000	635	9 000	255
W21X68	2 290	1 220	1 160	1 080	2 480	1 720	3 530	V_r 1 040	5 000	553	11 000	196
b=210	1 790	1 150	1 100	1 020	1 940	1 600	3 450	L_u 2 990	6 000	462	13 000	159
t=17.4	1 300	1 080	1 030	947	1 410	1 440	3 330	I_x 617	7 000	365	15 000	134
d=537	800	974	924	861	867	1 220	3 140	S_x 2 300	8 000	301	17 000	116
W530X92	2 790	1 210	1 140	1 050	3 020	1 670	3 280	M_r 637	3 000	633	8 000	253
W21X62	2 290	1 140	1 080	1 000	2 480	1 590	3 220	V_r 969	4 000	565	10 000	185
b=209	1 790	1 070	1 020	945	1 940	1 480	3 150	L_u 2 930	5 000	486	12 000	146
t=15.6	1 300	1 000	954	875	1 410	1 340	3 050	I_x 552	6 000	393	14 000	120
d=533	800	901	853	791	867	1 130	2 880	S_x 2 070	7 000	309	16 000	103
W530X82	2 790	1 090	1 020	940	2 830	1 500	2 910	M_r 559	3 000	551	8 000	204
W21X55	2 290	1 040	985	910	2 480	1 430	2 870	V_r 894	4 000	487	10 000	148
b=209	1 790	975	928	856	1 940	1 340	2 810	L_u 2 860	5 000	412	12 000	115
t=13.3	1 300	906	864	790	1 410	1 210	2 720	I_x 479	6 000	321	14 000	94.8
d=528	800	814	768	708	867	1 030	2 570	S_x 1 810	7 000	251	16 000	80.5
W460X106	2 770	1 210	1 140	1 040	3 000	1 510	3 380	M_r 645	3 000	640	8 000	308
W18X71	2 250	1 140	1 080	992	2 440	1 420	3 320	V_r 1 050	4 000	579	9 000	266
b=194	1 740	1 060	1 010	931	1 890	1 310	3 230	L_u 2 910	5 000	512	11 000	210
t=20.6	1 220	984	932	858	1 320	1 160	3 100	I_x 488	6 000	444	13 000	174
d=469	700	868	825	773	759	952	2 870	S_x 2 080	7 000	366	15 000	148
W460X97	2 770	1 130	1 060	968	3 000	1 400	3 100	M_r 589	3 000	581	8 000	264
W18X65	2 250	1 060	999	922	2 440	1 320	3 050	V_r 947	4 000	522	9 000	227
b=193	1 730	986	938	865	1 870	1 220	2 970	L_u 2 870	5 000	457	11 000	178
t=19.0	1 220	911	866	797	1 320	1 090	2 860	I_x 445	6 000	389	13 000	147
d=466	700	806	765	715	759	893	2 660	S_x 1 910	7 000	314	15 000	125
W460X89	2 770	1 070	1 000	910	3 000	1 310	2 890	M_r 543	3 000	534	8 000	231
W18X60	2 250	1 000	941	866	2 440	1 240	2 840	V_r 866	4 000	477	9 000	198
b=192	1 730	928	881	814	1 870	1 150	2 770	L_u 2 830	5 000	414	11 000	155
t=17.7	1 220	854	814	749	1 320	1 030	2 670	I_x 410	6 000	343	13 000	127
d=463	700	757	718	668	759	844	2 490	S_x 1 770	7 000	276	15 000	108
W460X82	2 770	986	920	834	2 810	1 220	2 640	M_r 494	3 000	482	8 000	195
W18X55	2 250	934	877	804	2 440	1 150	2 600	V_r 812	4 000	427	9 000	167
b=191	1 730	864	818	756	1 870	1 070	2 540	L_u 2 770	5 000	365	11 000	129
t=16.0	1 220	792	755	694	1 320	957	2 450	I_x 370	6 000	292	13 000	106
d=460	700	701	664	617	759	788	2 290	S_x 1 610	7 000	234	15 000	90.0

Composite Beam* columns: b_1 through S_t (Factored Resistances: M_{rc} and Q_r). Non-Composite Shape columns: Steel Shape Data and Unbraced Condition (L', M_r').

Note: *25 MPa, 1850 kg/m³ Concrete. #G40-21–M 300W Steel

Units: b–mm, t–mm, d–mm, M_r–kN·m, V_r–kN, L_u–mm, I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

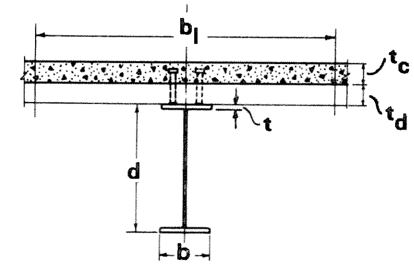
76 mm Deck with 85 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	b_1 (mm)	Composite Beam*: Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W460X74	2 770	902	837	757	2 550	1 120	2 410	M_r 445	3 000	433	8 000	164
W18X50	2 250	872	817	745	2 440	1 060	2 370	V_r 733	4 000	380	9 000	140
b=190	1 730	803	758	700	1 870	990	2 320	L_u 2 730	5 000	320	10 000	122
t=14.5	1 220	732	699	642	1 320	889	2 240	I_x 333	6 000	249	12 000	96.9
d=457	700	648	613	567	759	734	2 100	S_x 1 460	7 000	198	14 000	80.6
W460X67	2 770	832	770	695	2 340	1 040	2 210	M_r 405	3 000	390	8 000	140
W18X46	2 250	814	759	690	2 340	987	2 180	V_r 688	4 000	339	9 000	119
b=190	1 730	753	709	652	1 870	921	2 130	L_u 2 660	5 000	281	10 000	103
t=12.7	1 220	684	651	596	1 320	829	2 060	I_x 300	6 000	214	12 000	82.0
d=454	700	601	567	521	759	686	1 930	S_x 1 320	7 000	169	14 000	68.0
W460X61	2 770	747	688	619	2 100	932	1 970	M_r 354	3 000	336	8 000	111
W18X41	2 250	732	680	616	2 100	890	1 940	V_r 650	4 000	288	9 000	93.9
b=189	1 730	693	651	596	1 870	833	1 900	L_u 2 580	5 000	231	10 000	81.3
t=10.8	1 220	625	593	542	1 320	753	1 840	I_x 259	6 000	172	12 000	64.1
d=450	700	547	514	469	759	625	1 730	S_x 1 150	7 000	135	14 000	53.0
W410X85	2 760	957	889	802	2 920	1 080	2 560	M_r 467	3 000	455	8 000	205
W16X57	2 220	892	835	761	2 410	1 020	2 510	V_r 810	4 000	406	9 000	178
b=181	1 680	818	772	711	1 820	935	2 450	L_u 2 730	5 000	354	10 000	157
t=18.2	1 140	740	704	646	1 240	821	2 340	I_x 315	6 000	297	11 000	141
d=417	600	640	607	566	650	650	2 150	S_x 1 510	7 000	243	12 000	127
W410X74	2 760	853	789	708	2 580	971	2 270	M_r 408	3 000	394	8 000	163
W16X50	2 220	817	762	691	2 410	918	2 240	V_r 714	4 000	348	9 000	140
b=180	1 680	745	701	644	1 820	848	2 180	L_u 2 670	5 000	297	10 000	124
t=16.0	1 140	670	637	584	1 240	749	2 090	I_x 275	6 000	239	11 000	110
d=413	600	578	547	508	650	594	1 930	S_x 1 330	7 000	194	12 000	99.8
W410X67	2 760	773	712	637	2 320	888	2 060	M_r 367	3 000	352	8 000	135
W16X45	2 220	755	701	632	2 320	843	2 030	V_r 643	4 000	307	9 000	116
b=179	1 680	690	648	593	1 820	782	1 980	L_u 2 610	5 000	258	10 000	102
t=14.4	1 140	616	586	538	1 240	693	1 900	I_x 246	6 000	201	11 000	90.5
d=410	600	531	502	464	650	552	1 760	S_x 1 200	7 000	161	12 000	81.7
W410X60	2 750	686	629	561	2 050	798	1 830	M_r 321	3 000	306	8 000	109
W16X40	2 220	672	621	557	2 050	761	1 800	V_r 558	4 000	264	9 000	93.2
b=178	1 680	632	590	537	1 820	708	1 760	L_u 2 580	5 000	217	10 000	81.4
t=12.8	1 140	559	530	488	1 240	632	1 700	I_x 216	6 000	165	11 000	72.2
d=407	600	480	454	419	650	506	1 570	S_x 1 060	7 000	131	12 000	65.0
W410X54	2 750	618	564	502	1 840	718	1 630	M_r 283	3 000	266	8 000	86.1
W16X36	2 210	607	558	499	1 840	685	1 610	V_r 539	4 000	225	9 000	73.2
b=177	1 680	586	546	493	1 820	641	1 580	L_u 2 480	5 000	176	10 000	63.6
t=10.9	1 140	515	486	445	1 240	574	1 520	I_x 186	6 000	132	11 000	56.3
d=403	600	438	413	378	650	461	1 410	S_x 924	7 000	104	12 000	50.5

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

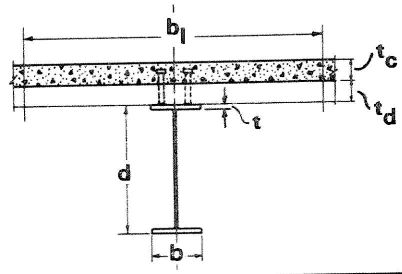
76 mm Deck with 85 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	b_1 (mm)	Composite Beam*: Factored Resistances: M_{rc} (kN·m) for Shear Connections= 100%	75%	50%	Q_r(kN) for 100%	I_t 10^6mm^4	S_t 10^3mm^3	Non-Composite Shape: Steel Shape Data	Unbraced Condition: L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
W410X46	2 720	540	491	435	1 590	633	1 410	M_r 239	2 000	236	7 000	61.7
W16X31	2 190	531	486	433	1 590	606	1 390	V_r 503	3 000	195	8 000	51.8
b=140	1 660	517	478	429	1 590	569	1 370	L_u 1 930	4 000	142	9 000	44.6
t=11.2	1 130	465	436	398	1 220	512	1 320	I_x 156	5 000	99.9	10 000	39.2
d=403	600	391	367	333	650	415	1 230	S_x 773	6 000	76.4	11 000	35.0
W410X39	2 720	460	416	368	1 350	542	1 190	M_r 197	2 000	193	6 000	55.2
W16X26	2 190	453	412	366	1 350	521	1 180	V_r 448	3 000	155	7 000	44.1
b=140	1 660	443	407	364	1 350	491	1 160	L_u 1 860	4 000	105	8 000	36.6
t=8.8	1 130	414	386	350	1 220	446	1 120	I_x 127	4 500	86.7	9 000	31.3
d=399	600	342	322	290	650	365	1 050	S_x 634	5 000	73.1	10 000	27.4
W360X79	2 780	817	751	668	2 730	827	2 210	M_r 386	4 000	364	8 000	225
W14X53	2 220	767	712	640	2 410	779	2 170	V_r 593	5 000	331	8 500	209
b=205	1 670	693	649	594	1 810	716	2 120	L_u 3 270	6 000	298	9 000	195
t=16.8	1 110	615	584	540	1 200	624	2 020	I_x 227	7 000	264	10 000	172
d=354	550	526	500	467	596	480	1 840	S_x 1 280	7 500	244	11 000	154
W360X72	2 780	741	678	600	2 460	754	2 000	M_r 346	4 000	322	8 000	186
W14X48	2 220	716	661	591	2 410	712	1 960	V_r 536	5 000	290	8 500	172
b=204	1 670	643	600	546	1 810	657	1 920	L_u 3 190	6 000	257	9 000	161
t=15.1	1 110	566	536	495	1 200	574	1 840	I_x 201	7 000	222	10 000	141
d=350	550	481	457	425	596	443	1 670	S_x 1 150	7 500	203	11 000	126
W360X64	2 780	667	607	536	2 200	685	1 790	M_r 308	4 000	283	8 000	153
W14X43	2 220	650	597	531	2 200	649	1 770	V_r 476	5 000	252	8 500	141
b=203	1 660	594	552	499	1 800	600	1 720	L_u 3 110	6 000	220	9 000	131
t=13.5	1 110	519	490	452	1 200	528	1 660	I_x 178	7 000	183	10 000	115
d=347	550	439	417	386	596	409	1 520	S_x 1 030	7 500	167	11 000	102
W360X57	2 750	609	553	487	1 950	643	1 620	M_r 273	3 000	259	7 000	119
W14X38	2 200	595	545	484	1 950	611	1 590	V_r 504	4 000	225	7 500	109
b=172	1 650	561	520	468	1 790	567	1 560	L_u 2 550	5 000	189	8 000	99.8
t=13.1	1 100	487	458	420	1 190	502	1 500	I_x 161	6 000	147	9 000	86.0
d=358	550	408	385	355	596	391	1 370	S_x 897	6 500	132	10 000	75.7
W360X51	2 750	546	494	434	1 740	580	1 450	M_r 241	3 000	227	7 000	97.0
W14X34	2 200	535	488	432	1 740	553	1 420	V_r 455	4 000	195	7 500	88.3
b=171	1 650	517	478	427	1 740	516	1 390	L_u 2 500	5 000	159	8 000	81.0
t=11.6	1 100	449	421	385	1 190	459	1 340	I_x 141	6 000	121	9 000	69.5
d=355	550	372	352	322	596	360	1 240	S_x 796	6 500	108	10 000	60.9
W360X45	2 750	487	439	385	1 550	520	1 290	M_r 210	3 000	195	7 000	76.5
W14X30	2 200	479	434	383	1 550	498	1 270	V_r 433	4 000	165	7 500	69.4
b=171	1 650	464	426	379	1 550	466	1 240	L_u 2 430	5 000	128	8 000	63.4
t=9.8	1 100	413	385	350	1 190	416	1 200	I_x 122	6 000	96.1	9 000	54.1
d=352	550	338	319	290	596	329	1 110	S_x 691	6 500	85.3	10 000	47.2

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b – mm, t – mm; d – mm; M_r – kN·m, V_r – kN; L_u – mm; I_x – 10^6mm^4, S_x – 10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

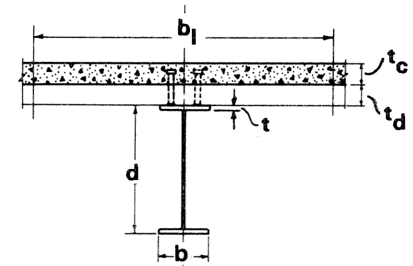
76 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam*						Non-Composite Shape				
		Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W360X39	2 700	428	384	336	1 340	460	1 120	M_r 179	2 000	173	5 500	61.1
W14X26	2 170	421	380	334	1 340	441	1 100	V_r 409	3 000	139	6 000	54.2
b=128	1 630	410	374	331	1 340	414	1 080	L_u 1 790	4 000	97.2	7 000	44.3
t=10.7	1 090	377	350	316	1 180	373	1 040	I_x 102	4 500	81.3	8 000	37.5
d=353	550	304	286	258	596	297	966	S_x 580	5 000	69.8	9 000	32.5
W360X33	2 700	359	321	280	1 130	390	937	M_r 146	2 000	139	5 500	43.2
W14X22	2 160	355	319	279	1 130	375	924	V_r 361	3 000	108	6 000	38.1
b=127	1 630	347	315	277	1 130	355	906	L_u 1 720	4 000	70.3	7 000	30.8
t=8.5	1 090	332	306	273	1 130	322	879	I_x 82.7	4 500	58.4	8 000	25.9
d=349	550	265	249	224	596	260	819	S_x 474	5 000	49.7		
W310X129	2 880	1 070	997	901	3 120	1 030	3 280	M_r 583	6 000	573	8 500	515
W12X87	2 290	991	928	851	2 480	956	3 200	V_r 742	6 500	562	9 000	504
b=308	1 690	905	857	798	1 830	858	3 090	L_u 5 580	7 000	550	9 500	492
t=20.6	1 100	818	784	734	1 190	728	2 920	I_x 308	7 500	539	10 000	481
d=318	500	710	685	655	542	540	2 580	S_x 1 940	8 000	527		
W310X118	2 880	1 000	929	835	3 120	951	2 990	M_r 526	6 000	513	8 500	455
W12X79	2 290	923	862	785	2 480	884	2 930	V_r 666	6 500	501	9 000	444
b=307	1 690	838	792	734	1 830	795	2 830	L_u 5 390	7 000	490	9 500	432
t=18.7	1 100	752	721	674	1 190	676	2 670	I_x 275	7 500	478	10 000	421
d=314	500	650	626	596	542	499	2 370	S_x 1 750	8 000	467		
W310X107	2 880	940	868	776	3 120	880	2 730	M_r 478	6 000	461	8 500	404
W12X72	2 290	861	801	727	2 480	820	2 680	V_r 604	6 500	450	9 000	393
b=306	1 690	778	732	676	1 830	740	2 590	L_u 5 220	7 000	438	9 500	381
t=17.0	1 100	694	663	620	1 190	631	2 460	I_x 248	7 500	427	10 000	370
d=311	500	597	574	545	542	466	2 180	S_x 1 590	8 000	415		
W310X86	2 830	816	748	661	2 970	748	2 240	M_r 383	5 000	367	7 500	309
W12X58	2 250	748	692	620	2 440	701	2 200	V_r 503	5 500	355	8 000	297
b=254	1 670	670	626	571	1 810	638	2 140	L_u 4 250	6 000	344	8 500	285
t=16.3	1 080	587	558	519	1 170	546	2 030	I_x 199	6 500	332	9 000	273
d=310	500	499	478	451	542	406	1 820	S_x 1 280	7 000	320	9 500	262
W310X79	2 830	753	687	604	2 730	689	2 060	M_r 346	5 000	327	7 500	270
W12X53	2 250	706	650	579	2 440	647	2 020	V_r 480	5 500	316	8 000	258
b=254	1 670	629	586	532	1 810	591	1 960	L_u 4 140	6 000	305	8 500	247
t=14.6	1 080	547	518	480	1 170	507	1 870	I_x 177	6 500	293	9 000	235
d=306	500	461	440	412	542	376	1 670	S_x 1 160	7 000	282	9 500	221
W310X74	2 780	717	653	573	2 560	660	1 940	M_r 321	4 000	307	7 500	221
W12X50	2 210	681	626	556	2 400	621	1 900	V_r 519	5 000	282	8 000	206
b=205	1 640	605	562	509	1 780	567	1 850	L_u 3 380	6 000	258	8 500	192
t=16.3	1 070	526	497	457	1 160	489	1 760	I_x 165	6 500	245	9 000	179
d=310	500	439	417	389	542	364	1 580	S_x 1 060	7 000	233	9 500	168

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40-21–M 300W Steel

Units: b–mm d–mm M_r–kN·m L_u–mm I_x–10^6mm^4
t–mm V_r–kN S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

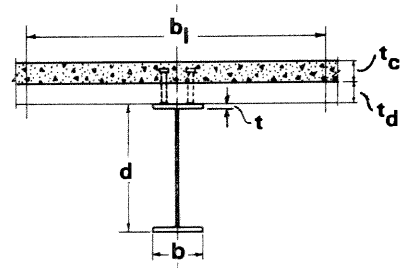
76 mm Deck with 85 mm Slab
ϕ=0.90, ϕ_c=0.60, ϕ_{sc}=0.80

Steel Shape#	b_1 (mm)	Composite Beam*						Non-Composite Shape				
		Factored Resistances				I_t	S_t	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for							
		100%	75%	50%	100%	10^6mm^4	10^3mm^3		L' mm	M_r' kN·m	L' mm	M_r' kN·m
W310X67	2 780	647	586	512	2 300	599	1 740	M_r 286	4 000	270	7 500	184
W12X45	2 210	628	575	507	2 300	565	1 710	V_r 463	5 000	246	8 000	169
b=204	1 640	561	519	467	1 780	518	1 660	L_u 3 280	6 000	222	8 500	157
t=14.6	1 070	483	455	418	1 160	449	1 590	I_x 145	6 500	210	9 000	147
d=306	500	399	379	352	542	335	1 430	S_x 949	7 000	198	9 500	138
W310X60	2 780	581	523	456	2 050	543	1 560	M_r 254	4 000	237	7 500	151
W12X40	2 210	566	515	452	2 050	514	1 540	V_r 405	5 000	214	8 000	139
b=203	1 640	521	480	429	1 780	474	1 500	L_u 3 200	6 000	191	8 500	129
t=13.1	1 070	444	416	382	1 160	414	1 440	I_x 129	6 500	179	9 000	120
d=303	500	364	346	320	542	310	1 300	S_x 849	7 000	166	9 500	112
W310X52	2 740	529	476	414	1 800	513	1 400	M_r 226	3 000	216	6 500	116
W12X35	2 180	517	469	411	1 800	488	1 380	V_r 429	4 000	189	7 000	106
b=167	1 620	493	454	403	1 760	452	1 350	L_u 2 570	5 000	162	7 500	96.8
t=13.2	1 060	419	392	357	1 150	397	1 290	I_x 118	5 500	146	8 000	89.4
d=317	500	340	322	297	542	301	1 170	S_x 747	6 000	130	8 500	83.0
W310X45	2 740	454	406	352	1 540	446	1 200	M_r 191	3 000	180	6 500	87.8
W12X30	2 180	445	401	350	1 540	425	1 190	V_r 368	4 000	155	7 000	79.3
b=166	1 620	431	393	347	1 540	396	1 160	L_u 2 490	5 000	128	7 500	72.4
t=11.2	1 060	376	349	316	1 150	351	1 120	I_x 99.2	5 500	111	8 000	66.5
d=313	500	299	284	260	542	269	1 020	S_x 634	6 000	98.2	8 500	61.6
W310X39	2 740	396	353	305	1 330	393	1 050	M_r 165	3 000	153	6 500	69.1
W12X26	2 180	389	349	304	1 330	376	1 040	V_r 320	4 000	130	7 000	62.2
b=165	1 620	378	343	301	1 330	353	1 010	L_u 2 440	5 000	103	7 500	56.5
t=9.7	1 060	343	317	284	1 150	315	979	I_x 85.1	5 500	88.5	8 000	51.8
d=310	500	267	254	232	542	244	900	S_x 549	6 000	77.7	8 500	47.8
W250X101	2 830	823	752	660	3 070	674	2 360	M_r 378	5 000	377	7 500	338
W10X68	2 250	745	686	612	2 440	626	2 300	V_r 560	5 500	369	8 000	330
b=257	1 670	664	619	562	1 810	564	2 220	L_u 4 950	6 000	361	8 500	323
t=19.6	1 080	579	549	509	1 170	475	2 090	I_x 164	6 500	354		
d=264	500	489	469	442	542	344	1 840	S_x 1 240	7 000	346		
W250X89	2 830	764	695	606	3 070	608	2 100	M_r 332	5 000	327	7 500	289
W10X60	2 250	688	631	558	2 440	568	2 050	V_r 496	5 500	320	8 000	281
b=256	1 670	609	565	510	1 810	513	1 990	L_u 4 690	6 000	312	8 500	273
t=17.3	1 080	526	496	459	1 170	435	1 880	I_x 143	6 500	304		
d=260	500	439	420	395	542	315	1 660	S_x 1 100	7 000	296		
W250X80	2 830	691	624	541	2 750	553	1 890	M_r 294	5 000	287	7 500	249
W10X54	2 250	641	586	514	2 440	518	1 850	V_r 429	5 500	280	8 000	242
b=255	1 670	564	521	467	1 810	470	1 790	L_u 4 520	6 000	272	8 500	234
t=15.6	1 080	482	453	417	1 170	400	1 700	I_x 126	6 500	265		
d=256	500	399	381	357	542	291	1 510	S_x 982	7 000	257		

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40-21–M 300W Steel

Units: b–mm d–mm M_r–kN·m L_u–mm I_x–10^6mm^4
t–mm V_r–kN S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

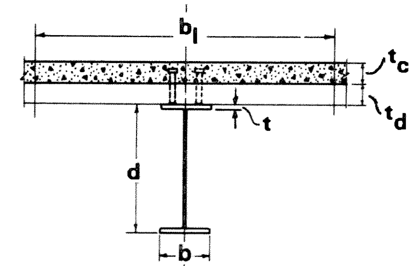
76 mm Deck with 85 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for 100%				L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
		100%	75%	50%								
W250X73	2 830	633	569	491	2 510	509	1 720	M_r 266	5 000	257	7 500	220
W10X49	2 250	606	551	481	2 440	478	1 690	V_r 388	5 500	250	8 000	212
b=254	1 670	530	487	434	1 810	436	1 640	L_u 4 390	6 000	242	8 500	205
t=14.2	1 080	449	421	385	1 170	373	1 560	I_x 113	6 500	235		
d=253	500	367	351	328	542	272	1 390	S_x 891	7 000	227		
W250X67	2 780	593	532	458	2 310	483	1 600	M_r 243	4 000	237	6 500	197
W10X45	2 210	574	521	453	2 310	454	1 570	V_r 408	4 500	229	7 000	189
b=204	1 640	506	464	412	1 780	415	1 520	L_u 3 570	5 000	221	7 500	181
t=15.7	1 070	428	400	364	1 160	357	1 450	I_x 104	5 500	213	8 000	174
d=257	500	347	330	306	542	262	1 290	S_x 806	6 000	205		
W250X58	2 780	518	461	395	2 000	424	1 390	M_r 208	4 000	199	6 500	160
W10X39	2 210	504	453	391	2 000	400	1 370	V_r 359	4 500	191	7 000	153
b=203	1 640	463	423	371	1 780	368	1 330	L_u 3 410	5 000	184	7 500	145
t=13.5	1 070	387	359	324	1 160	319	1 270	I_x 87.3	5 500	176	8 000	137
d=252	500	307	292	270	542	235	1 140	S_x 693	6 000	168		
W250X49	2 780	440	389	331	1 690	362	1 180	M_r 171	4 000	160	6 500	123
W10X33	2 210	430	383	328	1 690	344	1 150	V_r 326	4 500	153	7 000	115
b=202	1 640	412	373	324	1 690	318	1 130	L_u 3 240	5 000	146	7 500	106
t=11.0	1 070	345	318	284	1 160	278	1 080	I_x 70.6	5 500	138	8 000	97.2
d=247	500	267	253	231	542	207	976	S_x 572	6 000	130		
W250X45	2 720	420	372	317	1 540	364	1 110	M_r 163	3 000	151	5 500	101
W10X30	2 170	411	367	315	1 540	346	1 090	V_r 360	3 500	142	6 000	90.6
b=148	1 610	396	358	311	1 540	321	1 060	L_u 2 360	4 000	132	6 500	82.2
t=13.0	1 060	341	314	280	1 150	283	1 020	I_x 71.1	4 500	122	7 000	75.2
d=266	500	263	249	227	542	214	927	S_x 534	5 000	112	7 500	69.3
W250X39	2 720	362	319	272	1 330	317	959	M_r 139	3 000	126	5 500	77.5
W10X26	2 170	356	316	270	1 330	303	944	V_r 308	3 500	117	6 000	69.2
b=147	1 610	345	310	267	1 330	283	922	L_u 2 280	4 000	108	6 500	62.5
t=11.2	1 060	310	284	251	1 150	252	888	I_x 60.1	4 500	98.6	7 000	57.0
d=262	500	234	221	201	542	192	812	S_x 459	5 000	88.0	7 500	52.4
W250X33	2 720	308	270	229	1 130	272	815	M_r 114	3 000	102	5 500	55.6
W10X22	2 170	304	268	228	1 130	260	802	V_r 280	3 500	93.1	6 000	49.4
b=146	1 610	296	263	226	1 130	244	784	L_u 2 180	4 000	84.0	6 500	44.4
t=9.1	1 060	280	254	222	1 130	219	757	I_x 48.9	4 500	74.1	7 000	40.3
d=258	500	207	194	176	542	170	696	S_x 379	5 000	63.6	7 500	36.9
W200X86	2 790	689	621	532	3 000	485	1 870	M_r 265	5 000	261	7 500	235
W8X58	2 210	615	558	486	2 400	450	1 820	V_r 514	5 500	256	8 000	230
b=209	1 640	537	493	437	1 780	405	1 760	L_u 4 620	6 000	251		
t=20.6	1 070	455	425	387	1 160	340	1 660	I_x 94.7	6 500	245		
d=222	500	369	351	327	542	241	1 440	S_x 853	7 000	240		

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

COMPOSITE MEMBERS
Trial Selection Tables
Table 4.8



300W
25 MPa

76 mm Deck with 85 mm Slab
$\phi=0.90$, $\phi_c=0.60$, $\phi_{sc}=0.80$

Steel Shape#	Composite Beam*							Non-Composite Shape				
	b_1 (mm)	Factored Resistances				I_t 10^6mm^4	S_t 10^3mm^3	Steel Shape Data	Unbraced Condition			
		M_{rc} (kN·m) for Shear Connections=			Q_r(kN) for 100%				L′ mm	M_r' kN·m	L′ mm	M_r' kN·m
		100%	75%	50%								
W200X71	2 780	576	513	436	2 460	411	1 560	M_r 217	5 000	208	7 500	183
W8X48	2 210	549	496	426	2 400	385	1 520	V_r 393	5 500	203	8 000	178
b=206	1 640	474	432	379	1 780	349	1 480	L_u 4 150	6 000	198		
t=17.4	1 070	395	367	331	1 160	297	1 400	I_x 76.6	6 500	193		
d=216	500	313	298	276	542	212	1 230	S_x 709	7 000	188		
W200X59	2 780	484	427	359	2 040	348	1 300	M_r 176	4 000	174	6 500	149
W8X40	2 210	469	418	355	2 040	328	1 270	V_r 341	4 500	169	7 000	144
b=205	1 640	425	384	333	1 780	299	1 240	L_u 3 780	5 000	164	7 500	139
t=14.2	1 070	348	320	286	1 160	257	1 180	I_x 61.1	5 500	159		
d=210	500	268	254	234	542	186	1 050	S_x 582	6 000	154		
W200X52	2 780	429	376	315	1 800	311	1 150	M_r 154	4 000	150	6 500	126
W8X35	2 210	417	369	312	1 800	293	1 130	V_r 290	4 500	145	7 000	121
b=204	1 640	396	356	306	1 780	270	1 100	L_u 3 620	5 000	140	7 500	116
t=12.6	1 070	321	294	260	1 160	233	1 050	I_x 52.7	5 500	135		
d=206	500	242	229	210	542	170	940	S_x 512	6 000	131		
W200X46	2 780	380	331	276	1 580	278	1 020	M_r 134	4 000	129	6 500	105
W8X31	2 210	371	326	274	1 580	263	1 000	V_r 260	4 500	124	7 000	99.8
b=203	1 640	355	317	270	1 580	243	975	L_u 3 460	5 000	119	7 500	94.9
t=11.0	1 070	297	271	237	1 160	212	934	I_x 45.5	5 500	114		
d=203	500	220	207	189	542	156	841	S_x 448	6 000	109		
W200X42	2 740	348	303	252	1 430	258	930	M_r 120	3 000	119	5 500	93.5
W8X28	2 180	341	299	250	1 430	245	913	V_r 263	3 500	114	6 000	88.4
b=166	1 620	328	291	247	1 430	227	890	L_u 2 850	4 000	109	6 500	83.4
t=11.8	1 060	283	256	223	1 150	199	853	I_x 40.9	4 500	104	7 000	77.7
d=205	500	207	194	176	542	148	772	S_x 399	5 000	98.6		
W200X36	2 740	301	261	216	1 240	225	808	M_r 103	3 000	100	5 500	75.5
W8X24	2 180	296	258	215	1 240	214	793	V_r 222	3 500	95.3	6 000	70.6
b=165	1 620	286	252	212	1 240	200	773	L_u 2 730	4 000	90.4	6 500	64.7
t=10.2	1 060	260	235	202	1 150	177	743	I_x 34.4	4 500	85.5		
d=201	500	186	173	156	542	134	678	S_x 342	5 000	80.5		
W200X31	2 710	270	234	194	1 080	209	720	M_r 90.4	3 000	81.1	5 500	50.6
W8X21	2 160	266	231	193	1 080	200	707	V_r 240	3 500	75.3	6 000	45.6
b=134	1 610	259	227	191	1 080	187	690	L_u 2 150	4 000	69.5		
t=10.2	1 050	244	219	187	1 080	167	664	I_x 31.4	4 500	63.6		
d=210	500	176	163	146	542	129	609	S_x 299	5 000	57.0		
W200X27	2 710	230	198	164	915	180	614	M_r 75.3	3 000	65.4	5 500	36.4
W8X18	2 160	227	196	163	915	173	603	V_r 214	3 500	59.7	6 000	32.6
b=133	1 600	222	193	162	915	162	589	L_u 2 050	4 000	54.0		
t=8.4	1 050	211	187	159	915	146	568	I_x 25.8	4 500	47.5		
d=207	500	158	145	129	542	114	524	S_x 249	5 000	41.2		

Note: *25 MPa, 1850 kg/m^3 Concrete. #G40.21–M 300W Steel

Units: b–mm, t–mm; d–mm; M_r–kN·m, V_r–kN; L_u–mm; I_x–10^6mm^4, S_x–10^3mm^3

Chapter 5

5.0 COMPOSITE OPEN WEB STEEL JOISTS AND TRUSSES

5.1 INTRODUCTION

Compositely designed trusses for floor framing systems have gradually developed into one of the "menu" selections available to the structural designer. They are particularly attractive in spans greater than 10 metres where members appropriately sized for strength and stiffness offer "free" web openings large enough to accommodate a substantial amount of air ducting as well as other services.

Their origin stems from a non-composite member, the open web steel joist[5.1] (OWSJ, or simply "joist") which may also be viable as a composite member. These members were developed as alternatives to solid web members and for reasons of economy. S16.1 defines OWSJ as simply supported steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems, proportioned to span between structural supporting members, providing direct top chord support for floor or roof deck. Joists were first fabricated in 1923[5.2], using top and bottom chords of double round bars and webs composed of single round continuous bent bars to form a Warren truss type of web. Since then, various other joist chord types, web types and web configurations have been developed as proprietary products and manufactured by individual joist manufacturers.

Composite Open Web Steel Joists

The structural action of an OWSJ acting compositely with a 1 640 mm wide concrete slab on corrugated steel forms was first studied and reported by Lembeck[5.3]. The slab to joist shear connection in his test specimens was achieved by lowering the double angle top chord so that the web member ends extended above the top chord to be embedded in the concrete slab.

More studies on composite joists were reported by Wang and Kaley[5.4]. The researchers evaluated two composite joists fabricated according to a structural system devised by the K-System, Inc., of New York City. Performance of the composite joists was compared with the results of two additional non-composite OWSJ tests. Composite action of the joists was achieved in this case by embedment of the top chords in solid concrete slabs, without the use of specific shear connectors.

Later work reported by Tide and Galambos[5.5], was initiated to determine the behaviour of stud shear connectors in composite open web steel joist framing systems incorporating solid concrete slabs 75 mm thick. Five different OWSJ specimens were obtained from several steel joist manufacturing plants. Stud shear connectors were welded to the top chords using standard installation equipment and inspection procedures. Many factors governing the behaviour of stud shear connectors during composite action of joists were identified. The study also showed that only the web system of OWSJs should be counted on to carry the total design shear, a fundamental concept also applicable to the design of non-composite joists.

The composite joist tests described above were similar in that the joist spans were short (approximately 4 to 6 metres) and the joists were shallow in depth (250 to 400 mm). In addition, the spacing of the composite joists tested was generally no more than 700 millimetres. The researchers verified that reliable composite interaction with concrete slabs could be achieved by various means.

Tests on composite long-span OWSJs utilizing deck-slab systems incorporating a 38 millimetre deep steel deck were reported by Stelco[5.6] and by Atkinson and Cran[5.7]. A two part report on the same research[5.7] reviews the results of two long-span OWSJ tests as well as several push-out tests of shear connections connected to typical joist chord members. More information on design considerations, and cost comparisons of composite versus non-composite joist floors, is provided in further documentation of the same research. As indicated in both references, there are several building design considerations which demonstrate that composite OWSJ floor systems in certain span ranges should be considered equally feasible as other steel framed floor systems. The reporters found that with joists spaced 1.5 metres apart and joist spans of more than 11 metres, compositely designed joists were more economical than non-compositely designed joists. Other building design considerations include:

- ability to provide long column free spans,
- ability to permit relocation of heating, ventilation and air conditioning ducts and sprinkler pipes,
- ease in accommodating new electrical and communication requirements,
- better plenum space utilization, and
- reduced storey heights.

Tests on six joists (composite with a deck-slab system) were carried out during the early seventies by Azmi[5.8]. The joists spanned 15.24 metres and two specimens were obtained from each of three different joist manufacturers. The joist chords selected included a cold formed hat shape, hot rolled double angles, and a hot rolled hat shape. The overall composite depth of the system measured approximately 960 millimetres, with a deck-slab composed of a 38 millimetre deep deck plus a 65 millimetre thick normal density cover slab. The width of deck-slab, acting as the top chord of the composite OWSJ, was measured at 1 524 millimetres. Different types of shear connectors and different degrees of connection were used. Test results correlated well with analytical results and an ultimate design method was derived which gave good agreement with the experimental results.

Two additional composite OWSJ tests were reported by Fahmy[5.9] and results were compared to the previous six test specimens reported by Azmi. Based on those eight tests, Robinson[5.10] found that composite joists with a deck-slab system not only gave better stiffness but also offered better ductility than a comparable non-composite joist. Researchers[5.11] also pointed out the need to check compressive resistances of steel top chords of composite joists when the amount of supplied shear connection falls below 75% of the full connection case. They had determined that with lower levels of connection, the compressive capacity of a top chord could be governed by the buckling strength of the steel chord.

Composite Truss Evaluation

Like a composite OWSJ, a composite truss is normally designed as a non-shored member during construction, again with composite action achieved through direct connection of steel top chord to concrete top flange by means of shear connections. The viability of OWSJ is based on mass production of a standardized product in jigs. To justify the use of composite trusses (or non-composite for that matter) as the floor framing component of a structural system, a project should be framed to permit a large number of similar trusses which can then be built using a jig, without the significant cost premium which would result from "short run" or "one-off" production. The use of simple joist-type connection details, without gusset plates, is essential for economy. A major difference between an OWSJ and a floor truss of this type lies in the type of structural components selected to manufacture the chord and web members. Floor trusses for composite design are manufactured using members selected by the designer from a relatively wide range of steel products available. Although the design approach used for composite interaction of deck-slabs with OWSJs and trusses is similar, design applications for trusses are frequently beyond the capacity of conventional joists listed in a joist manufacturer's catalogue. Wider spacings and larger spans are the main features offered by a compositely designed truss system. The availability of wide-rib profile decks also makes a composite truss design a more viable solution for long span floor construction than that offered by joists.

A number of full scale composite truss tests have contributed to the development of a design methodology for this structural component. These tests were tied to specific projects and are reported in the following paragraphs.

As part of the development of an optimum floor system for the 110 storey Sears Tower in Chicago, composite trusses were tested to validate this method of construction. Iyengar and Zils[5.12] credit this floor system with a reduction in floor steel, increased flexural stiffness as well as with providing the long-span feature unique to this bundled-tube structure, without inhibiting the passage of mechanical ducts within the floor-ceiling sandwich. The composite trusses tested were spaced 4 570 millimetres apart with a span of 22.85 metres. The deck-slab system was constructed with a 76 millimetre deep composite wide-rib profile steel deck and topped with a 65 millimetre thick structural semi-low density concrete. A comparison of the test results with a theoretical analysis showed good correlation. The structural analysis was performed in two stages. For the wet concrete condition prior to the hardening of concrete, the truss was analysed non-compositely with all joints considered rigid. Effective design depth of the non-composite trusses was based on C.G. distances between the top and bottom tee chords. For the superimposed dead and live loads, composite properties of the truss with the deck-slab system were used.

A research test on a typical full-scale composite floor truss intended for use on a highrise project in downtown Edmonton, was conducted and reported on by Bjorhovde[5.13]. The span of the truss tested was 12 metres, with an out-to-out depth of 850 millimetres. The deck-slab system, connected to the truss by means of stud shear connectors, was composed of a 76 millimetre deep composite wide-rib profile steel deck and a 65 millimetre normal density concrete cover slab. The test showed full elastic response of the truss in the service load range, and the ultimate load exceeded the limit states design prediction by about 7%. Overall failure was precipitated by buckling of a compression diagonal and recommendations were drawn with regard to design and construction procedures.

Today, a composite open web steel joist (or truss) design is a fully recognized form of construction, with general slab design rules covered by the "composite beam" section of S16.1. A list of some U.S. and Canadian buildings using this form of construction is included in Table 5.1. For more information about Canadian examples, the reader is referred to a paper by Ritchie[5.14]. The following sections of this chapter are devoted to the specific design considerations and methodology of composite OWSJ and truss floor systems.

5.2 CHORD AND WEB STEEL SHAPES AND WEB FRAMING CONFIGURATIONS

Whether one is designing a joist or a floor truss (either composite or non-composite), the key to selection of economical and compatible chord and web member types, and web framing configuration usually lies in the ability to produce a "gussetless" truss assembly, suitable for a jigged production process during fabrication. (See Section 5.6 for information on connections and details.)

For composite truss design, chord members may be selected from steel products such as angles, tees, and rectangular or square hollow structural sections (HSS). At the same time, several steel products are offered for web member selection, i.e. angles, various types of HSS shapes, flats, and others. Only certain combinations of chord and web member types may be considered compatible for ease of jigging, ease of welding and therefore overall economy of production. See Table 5.2 (also see Section 5.6 for more details).

TABLE 5.1 SOME IMPORTANT CANADIAN AND U.S. COMPOSITE OWSJ AND TRUSS BUILDINGS

Project Name	No. of Storeys	Total Steel Content* GFA (kg/m^2)	Remark
Oxford Square Towers, Calgary	33 and 37	45.0 (gravity steel)	Comp. OWSJ 11.8 m at 2030 c/c
Stelco Tower, Hamilton	26	36.8 (gravity steel)	Comp. OWSJ 12.4 m at 1520 c/c
Guardian Royal Exchange Tower, Toronto	23	47.8 (gravity steel)	Comp. OWSJ 12.5 m at 2290 c/c
Campeau Corp. Principal Plaza, Edmonton	29	40.4 (gravity steel)	Comp. Truss 12.0 m at 3000 c/c
Edmonton Centre 3, Edmonton	29	39.3 (gravity steel)	Comp. Truss 10.7 m at 2950 c/c
Edmonton Centre 5, Edmonton	32	40.8 (gravity steel)	Comp. Truss 10.8 m at 3050 c/c
The World Trade Centre, Twin Office Towers, New York	110	180 (gravity plus lateral steel)	Comp. OWSJ 18.3 m at 1006 c/c
Sears Tower, Chicago	109	161 (gravity plus lateral steel)	Comp. Truss 22.9 m at 4573 c/c

*Columns and supporting steel girders, etc, are included.

Three types of web-framing configurations are common in floor truss and joist designs. These are:

- Pratt,
- Warren, and
- Modified Warren.

A truss or joist design may be affected structurally and non-structurally by the selection of its web-framing configuration in the following ways:

- The efficiency of various web members in resisting vertical shear forces may be affected by the choice of a web-framing configuration, e.g., the selection of Pratt web over Warren web may effectively shorten compression diagonals resulting in more efficient use of these members.
- Web configuration may also influence the unsupported panel length of the compression top chord, affecting its ability to carry local bending moments, e.g.,the middle post in a modified Warren configuration provides mid-panel support to the top chord.
- Web to chord connection details may be partly resolved by selecting an appropriate web framing configuration, although in most cases also by selecting a deeper chord member.

TABLE 5.2 RECOMMENDED COMBINATIONS OF WEB AND CHORD MEMBER TYPES FOR TRUSSES OF A PRE-SELECTED WEB FRAMING CONFIGURATION

Truss* Type	Name of Component Member	WEB CONFIGURATION Pratt	Warren	Modified Warren
1	– Top and bottom chords	SHS, RHS	SHS, RHS	SHS, RHS
	– Diagonals and Verticals	2L's	2L's	2L's
	– Modified Warren Web Posts			L, SHS, Round
2	– Top and bottom chords	Tees	Tees	Tees
	– Diagonals and Verticals	L or 2L's	L or 2L's	L or 2L's
	– Modified Warren Web Posts			L, SHS, Round
3	– Top and bottom chords	2L's	2L's	2L's
	– Diagonals and Verticals	SHS, RHS	SHS, RHS	SHS, RHS
	– Modified Warren Web Posts			SHS, RHS
4	– Top and bottom chords	SHS, RHS	SHS, RHS	SHS, RHS
	– Diagonals and Verticals	HSS	HSS	HSS
	– Modified Warren Web Posts			HSS

* Arranged in the ascending order of costs.
(Gussetless trusses are assumed)

- Availability of adequate web openings to accommodate ducts and other services may also be affected by this choice. Warren and modified Warren will usually permit larger web openings.
- The number of web component pieces handled during fabrication is yet another result of the web-framing-configuration selection.

Experience has shown that both Pratt and Warren configurations of web framing are suitable for short span trusses and joists with shallow depths. For truss or joist members with spans greater than 10 metres, or effective depths larger than 700 millimetres, a modified Warren configuration is generally more appropriate. Table 5.2 outlines the possible choices of compatible chord and web member types in relation to web framing configurations of composite truss members.

5.3 PROPOSED DESIGN CRITERIA FOR COMPOSITE OWSJ AND TRUSS MEMBERS

Unless otherwise noted, the following proposed design criteria are applicable to both composite OWSJs and composite trusses under generally uniform floor loading. These proposed design criteria, embodied in Sections 5.4 and 5.5, are in the form of descriptive discussions relating to design considerations concerning strength and serviceability requirements of such structural members. Basic structural assumptions and computational methods dealing with analysis (for the computation of factored design web or chord forces) are discussed and illustrated in an example in Section 5.8. Detailed formulas will not be given; designers are referred to the current relevant design clauses of S16.1 for the assessment of factored structural resistance of individual components.

To facilitate the steel cost comparison of a composite truss or joist floor with other floor framing systems, information provided by Reference 5.15 may prove to be useful.

5.4 STRENGTH DESIGN CONSIDERATIONS

Like an unshored hollow composite beam, a composite truss or joist must provide adequate factored resistance against collapse under loading conditions as follows:

- Deck placement,
- Concrete placement, and
- Occupancy loading.

Unlike a hollow composite beam, the factored resistance against collapse of a composite truss or joist depends on the factored resistance against failure of each individual component. During construction stages, particularly prior to concrete placement and again prior to full curing of concrete, the strength of the top steel chord must be evaluated. Under the occupancy load condition, the concrete top flange is assumed to participate structurally (in lieu of steel top chord) as required by S16.1. Although detailed design rules for web to chord connections have not been provided in this publication, the compatibility of web and chord members for the development of desired connection forces and for the development of economical connection details must not be overlooked. See worked example, Section 5.8.

Itemized discussions relating to strength design of the following component members are provided in the remainder of this section.

- Steel top chord member,
- Concrete deck-slab as a top flange,
- Steel bottom chord member,
- Web framing members, and
- Stud shear connectors.

a) Steel Top Chord Member

In composite design, the steel top chord member of a truss or joist is designed to carry construction loads prior to composite action. For OWSJ members, design of the top chord is governed by S16.1, Clauses 16.5.8, and 16.7; and for trusses, each panel of top chord steel member must be designed (under factored construction load condition) for axial and local bending effects based on actual conditions of restraint (provided by either additional steel support members, or through deck-steel direct connections). Structural analysis of construction load effects for truss types as described in Table 5.2 should be carried out assuming either pin-jointed or rigid-jointed web to chord framing.

Rules provided in Chapter 2, for stud shear connector design, require thickness of steel top chord material to be no less than stud diameter divided by 2.5. Otherwise, reduced stud shear resistance should be used during design.

Under occupancy loading, the steel top chord of a truss or joist is neglected in strength evaluations, except that the top chord should be treated as a horizontal shear transfer medium which collects horizontal components of web forces to be transmitted to the concrete top flange via shear connectors (normally studs).

To ensure overall member stability during construction, top chords of joists or trusses must be restrained against lateral movement through provision of direct structural support such as bridging or other means. Some truss chord configurations may have sufficient lateral stability to support steel deck and construction loads, permitting erection without bridging; however, adequate welding of the deck to the truss chord, before additional loads are carried, is essential.

b) Concrete Deck-Slab

The factored ultimate bending resistance of a composite joist or truss is computed using only the effective slab area of deck-slab system (excluding the steel top chord). Only the full shear connection case is to be used in computing the moment resistance. Refer to S16.1 Clause 17.4.3(a).

The effective width of concrete top flange for the analysis and design of a composite joist or truss

should be calculated using S16.1 rules (also as shown in Section 1.4 and Figure 1.13 of this publication).

Figure 5.1, illustrates the force equilibrium of a composite joist or truss cross section, assuming full shear connection. The plastic neutral axis (P.N.A.), in this case, lies within the depth of the cover slab. The factored compressive resistance (newtons) of the effective concrete slab is computed as $0.85\,\phi_c\,f'_c\,a\,b_1$, and the factored tensile resistance (newtons) of the steel bottom chord is computed as $\phi\,A_sF_y$, where,

ϕ_c = performance factor for concrete in flexure, 0.60
ϕ = performance factor for steel in flexure, 0.90
b_1 = effective width of concrete slab, mm
a = depth of concrete slab under ultimate compression, mm
f'_c = specified compressive strength of concrete at 28 days, MPa
A_s = cross sectional area of steel bottom chord, mm^2
F_y = specified minimum yield strength of steel bottom chord, MPa

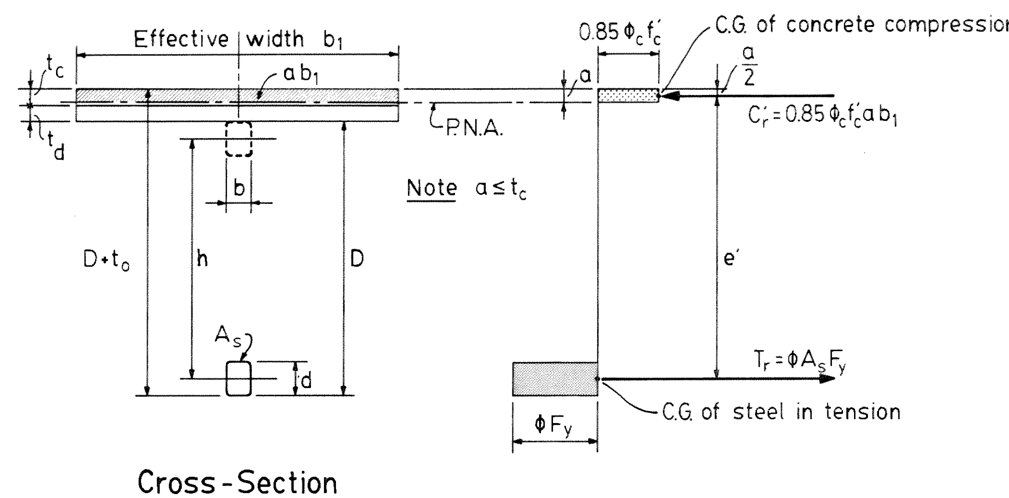


Figure 5.1
Force Equilibrium of Composite Truss
(or Joist) Section

Equating factored compression to factored tension,

$$0.85\,\phi_c\,f'_c\,a\,b_1 = \phi\,A_s\,F_y$$

and solving for 'a',

$$a = \frac{\phi\,A_s\,F_y}{0.85\,\phi_c\,f'_c\,b_1} \qquad 5.1$$

and 'a' must not exceed t_c, i.e. Case 1, Clauses 17.4.2, 17.4.3(a)

The factored moment resistance of the composite section may be computed as

$$M_{rc} = e'\,\phi\,A_s\,F_y \qquad 5.2$$

where e′ represents the distance from C.G. of steel bottom chord to C.G. of concrete in compression.

c) Steel Bottom Chord Member

The prime function of the steel bottom chord is to provide the "tensile" component of moment resistance to a composite joist or truss under occupancy loading. However, during the selection of such a member, one must also provide adequate member stiffness to facilitate handling and erection.

Rules for strength and stability design of OWSJ bottom chords are provided by S16.1, Clauses 16.5.7, 16.6 and 16.7. Other strength considerations, when selecting chord members, include the availability of adequate depth of chord sections for the development of connection forces.

d) Web Framing Members

In composite design, web framing members of joists or trusses are proportioned to carry the total vertical shear as required by Cl. 17.3.1.2 of S16.1. In other words, factored design axial forces in web framing members under occupancy load may be conservatively analysed by resolving a statically determinate (pin-jointed) truss model neglecting any shear contribution from the concrete cover slab and steel chords. To illustrate the above, an example is provided in Section 5.8, showing the analysis method and rationale as well as the applications of S16.1 design clauses.

A joist or truss member rarely possesses true pin-joints, and local moment occurring at web-to-chord joints can be redistributed into web members. Strength and stability of compression web members may be affected, and it is prudent for a designer to include such design forces during the selection of web compression members. There are four main causes of local bending in web members:

- Floor loadings on equal or unequal top chord panels (Fig. 5.2).
- Web to chord joint eccentricity (Fig. 5.3).
- Connection eccentricity (Fig. 5.4).
- Localized overturning due to steel-to-concrete shear connection (Fig. 5.5).

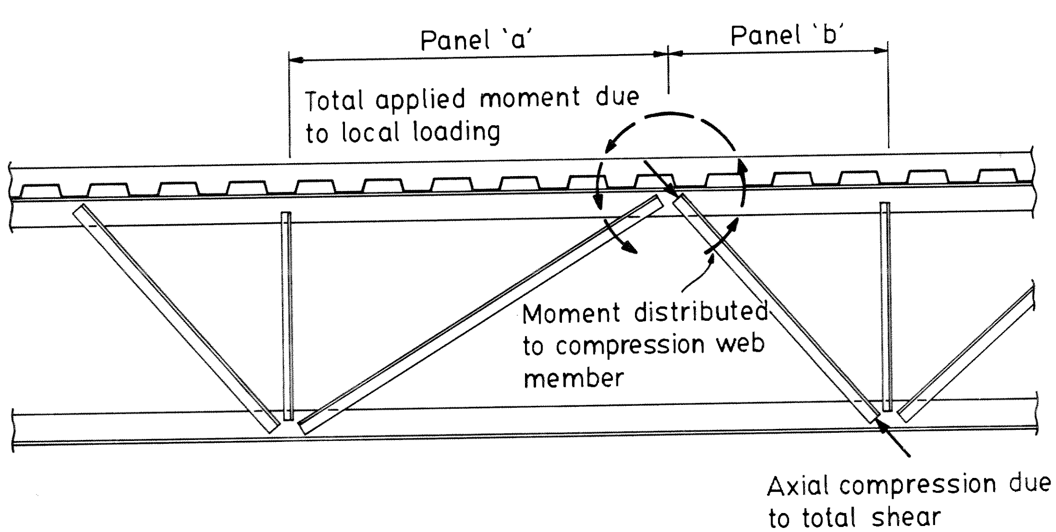


Figure 5.2
Induced Bending due to Floor Loads Acting at Top Chord

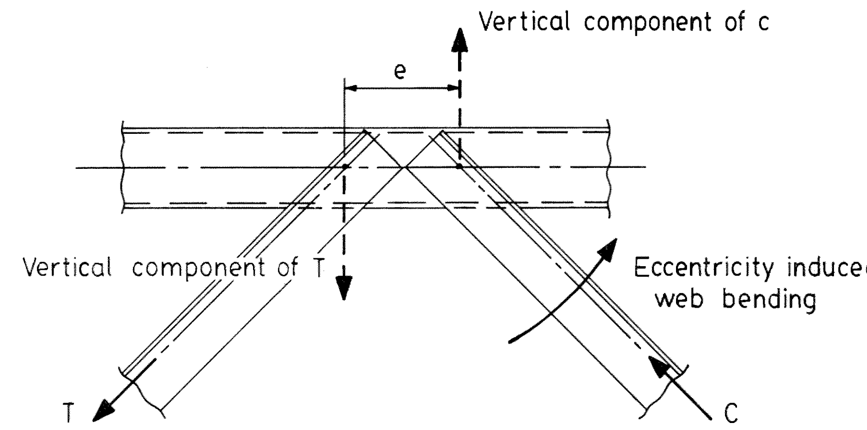


Figure 5.3
Bending due to Joint Eccentricity

The effects of local bending are generally neglected in the design of tensile web members due to the fact that such effects are often too small to affect member sizing. It is proposed that both OWSJ and truss web members in tension are required to meet a limiting slenderness ratio of 300 to facilitate handling and erection. In addition, analysis should be made to provide adequate compressive strength to tension members to account for stress reversal under patterned loading conditions and during handling and erection.

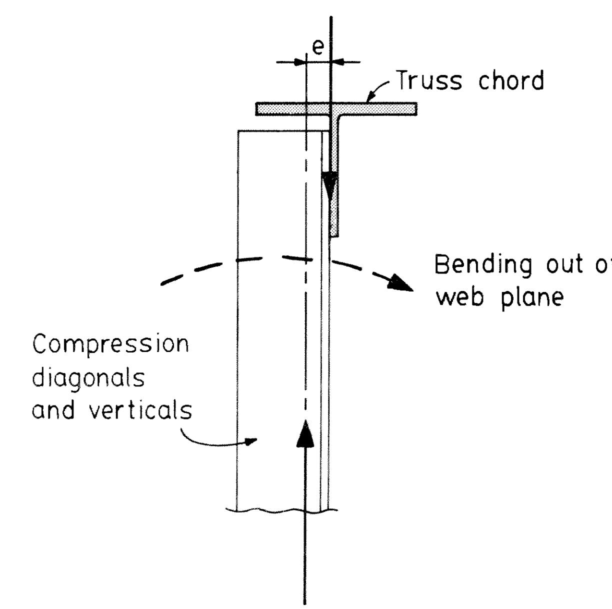


Figure 5.4
Induced Bending due to Connection Eccentricity

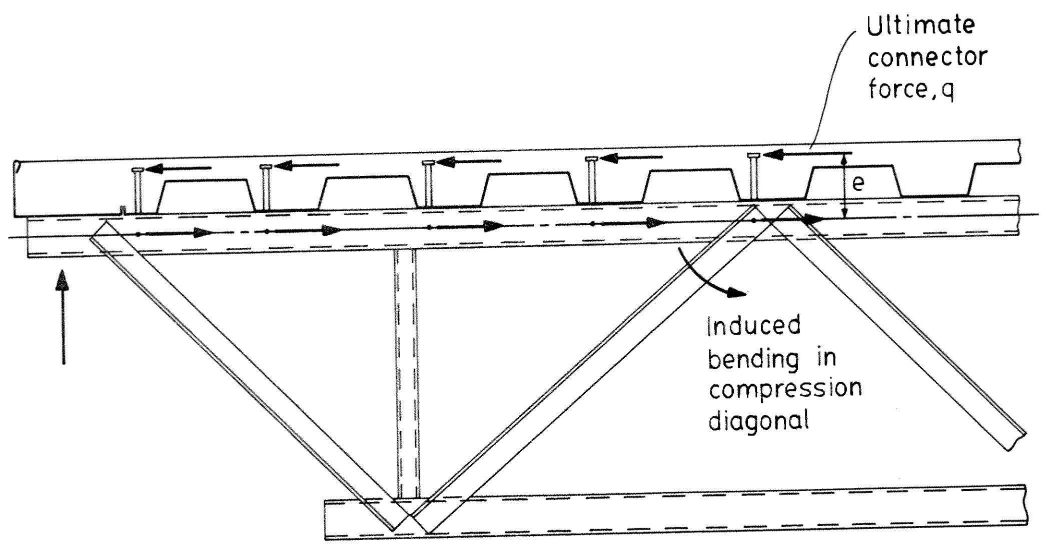


Figure 5.5
Induced Bending due to Localized Overturning of Stud Connections

e) Stud Shear Connections

S16.1 requires compositely designed joists or trusses to have full shear connectors between concrete top flange and steel top chord. Therefore the total horizontal factored shear of a composite joist or truss between the point of maximum bending and each adjacent point of zero moment can be represented as,

$$V_h = \phi A_s F_y \tag{5.3}$$

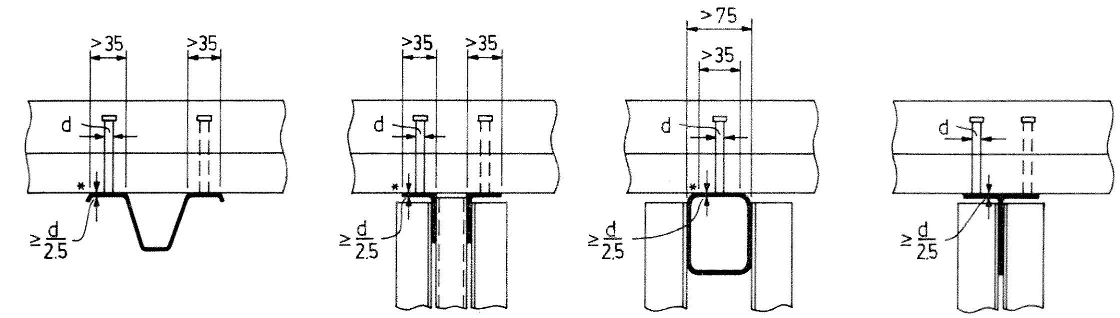
Stud shear connectors are commonly used to provide shear transfer at interfaces of steel and concrete. Factored shear resistances of studs in solid slab or deck-slab systems may be computed using the information provided in Chapter 2. When joist top chord segments between stud connectors are slender, it may be desirable for the designer to check the buckling strength of the top chord segments between shear connectors under the fresh-concrete condition loading.

Stud shear connectors must be distributed along the top chord member to enable a smooth transfer of interface shear (produced by the horizontal components of web forces transmitted through the steel top chord) to be carried by the concrete compression flange. Field welding of studs to steel joists or trusses through steel decks may be greatly facilitated if the stud-receiving flat width of a top chord component member is limited to not less than 35 mm, and the out to out width of a top chord is limited to not less than 75 mm. Figure 5.6 illustrates proposed limits of top chord dimensions for field welded stud application to OWSJs and trusses.

5.5 SERVICEABILITY DESIGN CONSIDERATIONS

The serviceability limit states design of a composite floor member often includes the following important considerations:

- deflection of steel member under fresh-concrete condition load,
- deflection of the composite member under occupancy live load and part of the superimposed dead load,



* Thinner flanges may be used, provided that stud shear resistance is reduced in design calculations and that flange materials are thick enough to prevent weld-through of studs during application.

Figure 5.6
Proposed Top Chord Selection Criteria to Facilitate Shear Stud Application

- floor vibration due to occupancy activity,
- deflection of floor member due to slab shrinkage,
- deflection of floor member due to creep of concrete top flange.

Member deflection under fresh-concrete load

The traditional method of fabrication for OWSJ is to provide camber by setting the manufacturing jig to an amount specified by Clause 16.5.15 of S16.1 unless otherwise specified by the building designer. The amount of camber suggested, in this case, is generally appropriate for joists of relatively light non-composite construction such as in floors where joists are closely spaced, or in non-composite roof construction, and would normally be inappropriate for compositely designed members.

In specifying compositely designed trusses or OWSJ for floor construction, it is advisable for the designer to indicate the amount of camber (usually equal to the elastic deflection of the non-composite truss or joist under the concrete slab load) to the fabricator. Thus, a "flat" floor is achieved, avoiding additional loads created by concrete slab "ponding" on deflected steel members (See Section 4.8).

Joist or truss deflection under fresh-concrete condition loads may be computed using sophisticated computer programs assuming rigidly connected web-chord members, or by a process of manual computation using a commonly accepted approximation method, such as the method described in S16.1 Cl. 16.5.14.2. For more detail, see worked example in Section 5.8.

Composite member deflection

Even though the deflection of composite joists or trusses under live loads is rarely critical when compared to the S16.1 recommended maximum values for deflections of floor members, it is still considered good practice to complete calculations to substantiate this fact.

Deflection of composite joists or trusses under occupancy loads may be computed manually or with the aid of a computer in the following manner:

Manual computation procedure:

Step 1 Compute effective slab area of the concrete top flange and transform into equivalent steel area.

Step 2 Compute gross moment of inertia of the composite member, I_t, using only the steel bottom chord and the transformed concrete top chord.

Step 3 Estimate reduction of moment of inertia of the steel member, I_r, using I computed from steel chord members and multiply the result by 0.15. (This is an estimated reduction of moment of inertia to account for the web's contribution of member deflection.)

Step 4 Subtract value obtained in Step 3 from the value obtained in Step 2. The resulting value is then divided by $(1+0.15+0.15)$ to allow for the effect of increased flexibility due to slip and creep.

Step 5 Compute composite joist (or truss) deflection using the moment of inertia value obtained in Step 4.

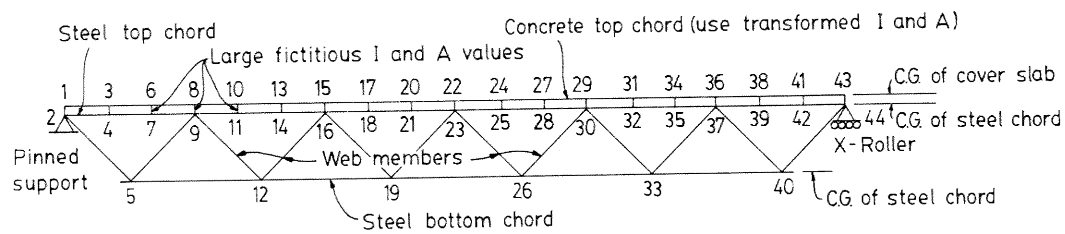


Figure 5.7
Composite Truss Modelling Technique for "Detailed" Structural Analysis

Computer stiffness analysis procedure:

Step 1 A composite truss (or joist) may be modelled as shown in Fig. 5.7. The effective concrete top flange is transformed to an equivalent steel area. The interconnection of concrete slab and steel top chord is modelled by short bar members having large moments of inertia and area.

Step 2 Analyse the model using a 'plane frame' type of computer program, to obtain member deflection under the specified floor loads. Note that the computed truss deflection need not be arbitrarily increased by a further 15 percent to account for slip and flexibility of concrete ribs formed by steel deck, due to the fact that some vierendeel effect was accounted for through the modelling technique. Likewise the computed truss deflection also includes its web contribution.

Floor vibration due to occupancy

Composite truss (or joist) floor systems provide strong and stiff floors in most instances. When such a system is used to support a large open area, free of partitions or other natural damping features, special consideration should be given to susceptibility to walking vibration to ensure that vibration characteristics are acceptable for the intended use and occupancy. The reader is referred to Appendix G of S16.1 for design information. A sample calculation of a composite truss floor vibration evaluation is demonstrated in Section 7.6.

Deflections due to creep and shrinkage

The latter two serviceability design considerations are often considered to be not critical in the design of composite joists and trusses for the following reasons:

- The composite joist and trusses covered by this chapter are generally deep, with the out-to-out steel member depths falling within the range of 1/17 to 1/11 of span. With such steel member depths, shrinkage and creep deflections of composite assemblies tend to be insignificant compared with the values one would find with a similar deck-slab on a shallower solid web steel beam.
- The composite joist or truss floors described herein are normally used for office occupancy. In such cases, the amount of sustained live load does not represent a significant portion of the total specified design live load. Hence, creep deflections also tend to be not critical.

5.6 TYPICAL CONNECTIONS AND DETAILS FOR TRUSSES

The key to economical production of trusses lies in the selection of compatible web and chord members to permit simple and direct connection details. Without the use of loose connection materials or gusset plates, members are produced on a mass production basis using a jig. When using a jig, the chord members are positioned with the desired camber to permit placement of one half of the web members on one side of the truss. Then, the entire assembly is turned over 180 degrees for placement of the other half of the web members.

In the case of trusses with WT chords and single angle web members, the process of web installation may be simplified, permitting a moderate saving in fabrication costs. For example, all web members may be placed on one side, then the truss "flipped" to add end diagonals, thus avoiding undue eccentricity at the truss end connection. Alternatively, the web system may be staggered. For example, in a Pratt configuration, all verticals may be placed on one side with diagonals on the other (but usually paired at the end) allowing easier fit-up and welding.

Although it is desirable that component members, connected at a joint, have their centroidal axes intersecting at a point, it is frequently unavoidable during shop-detailing. The joint eccentricity which is thus introduced, permits the production of an inexpensive joint fabrication detail. Such details could consist of web members free of angular clipped ends along with an easy-to-fabricate welding detail. Additional structural analysis and design are required to assure the adequacy of member and connection resistances, including the effects of joint eccentricity in the analysis. The joint eccentricity illustrated in Fig. 5.3 could be more readily tolerated at mid span of a compositely designed truss, whereas, use of a similar detail near the ends of the truss, could create greater concern of a shear failure.

The design of double angle web compression members often includes the consideration of whether battens or spacers are required. Heavy unbattened members or lighter battened members may be considered in selecting the most economical solution.

Truss types 1 and 2 (shown in Table 5.2) utilizing HSS top and bottom chords with double angle webs or WT top and bottom chords with single and/or double angle webs are generally considered to be best suited to the fabrication criteria as described in the last few paragraphs. Nevertheless, trusses fabricated with double angle top and bottom chords and an HSS web system continue to be strongly recommended by some fabricators from an economic standpoint.

Some typical web-to-chord connection details are shown in Figure 5.8. Typical end-support details for flange-top connections and web-framed connections are shown in Figure 5.9. To illustrate the capability of cantilever extensions using a compositely designed truss, the reader is referred to Figure 5.10. In Figure 5.11, typical details at a vierendeel opening are also provided.

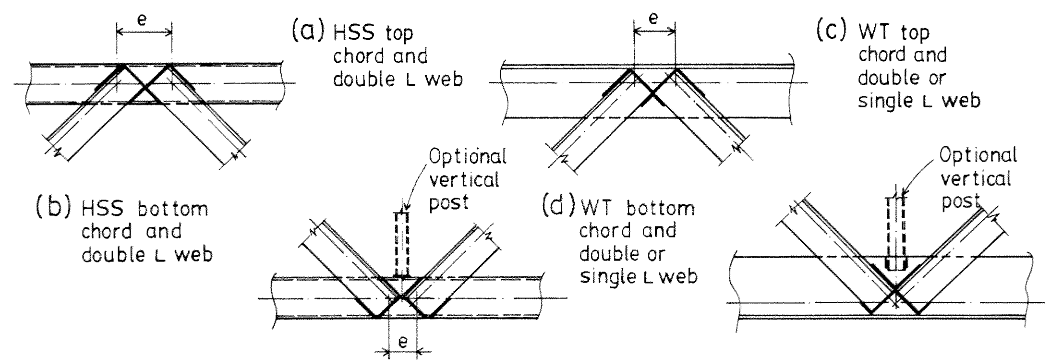


Figure 5.8
Typical Web-to-Chord Connection Details

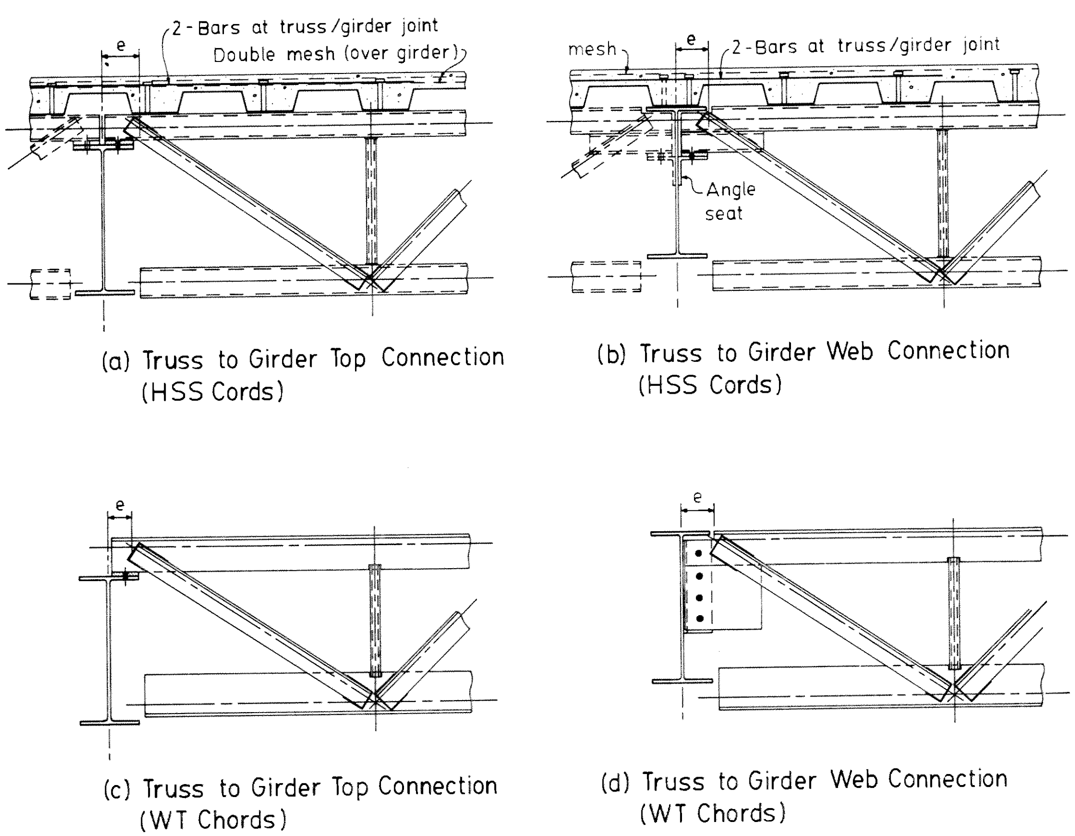


Figure 5.9
Typical Truss-to-Girder Connections

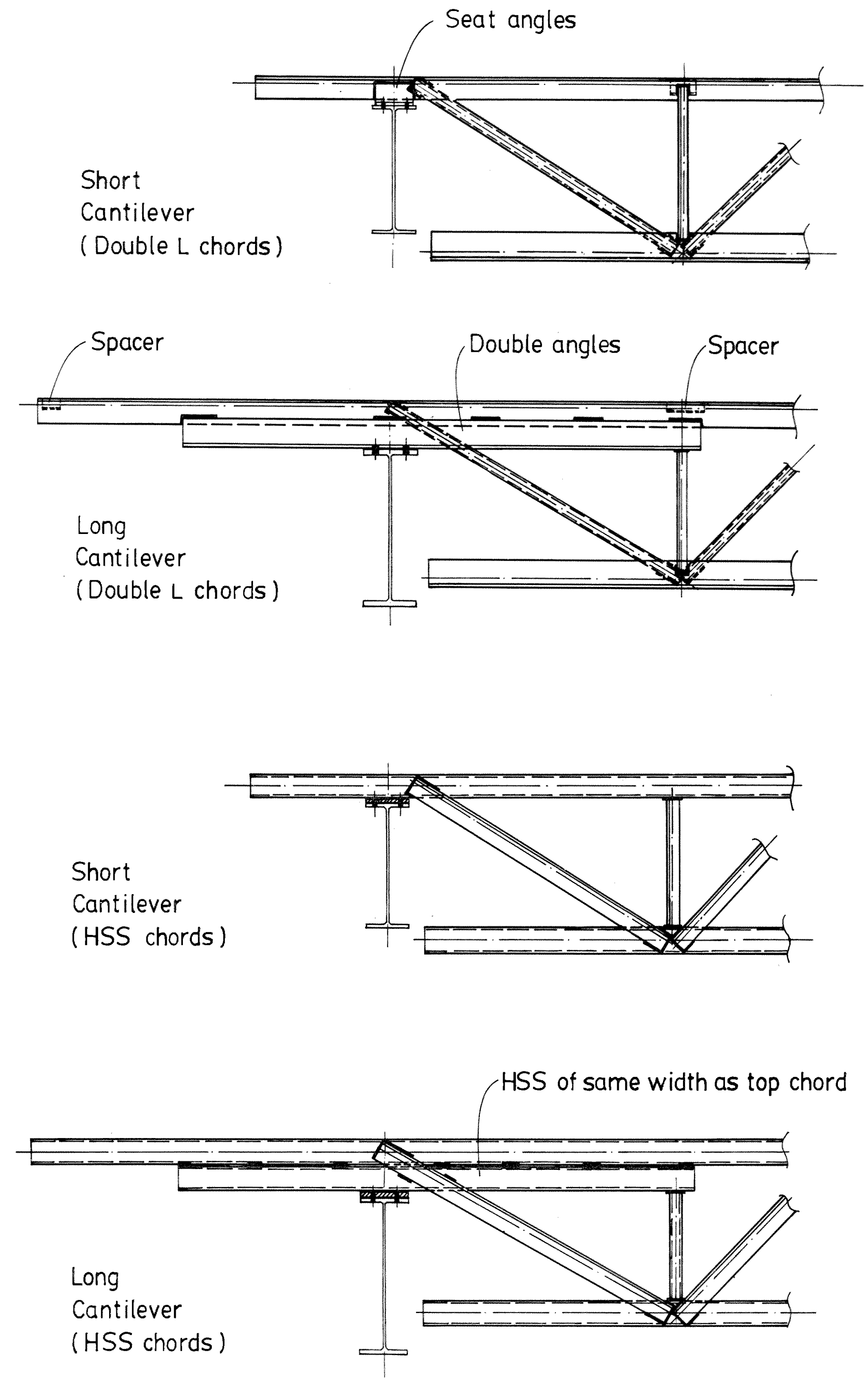


Figure 5.10
Typical Cantilever End Details
for Composite Truss Design

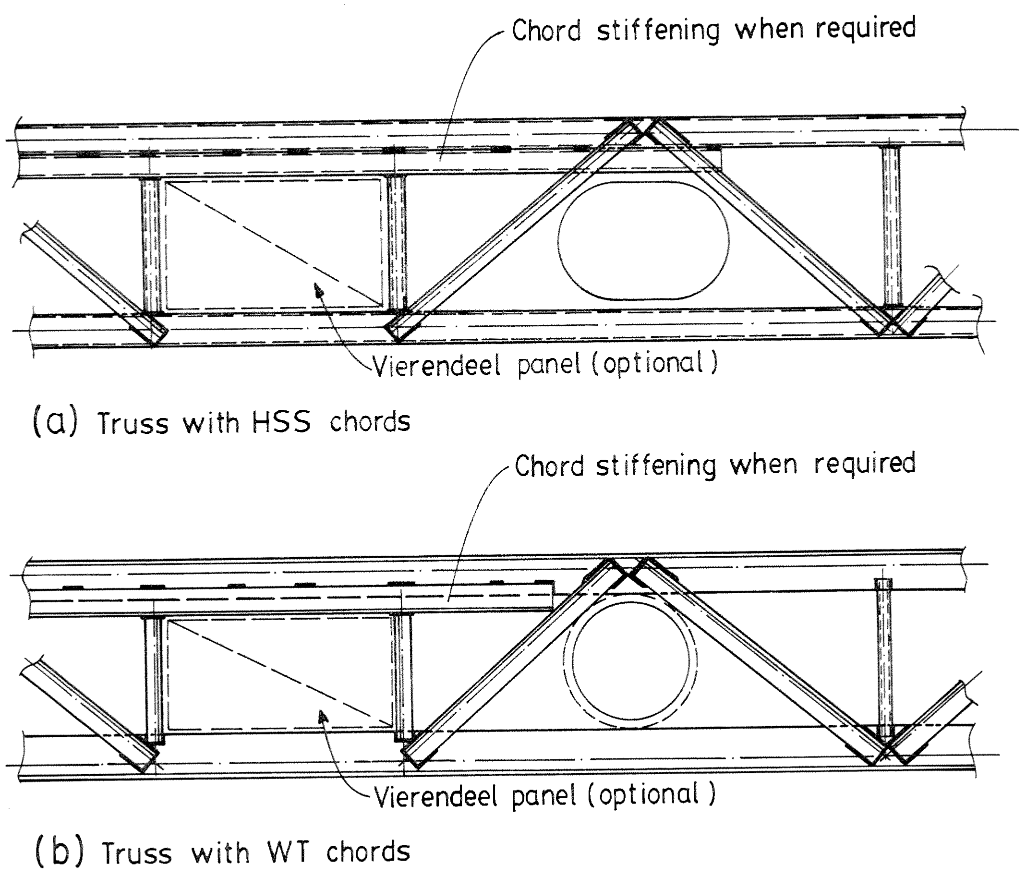


Figure 5.11
Typical Vierendeel Opening Details

5.7 COMPOSITE TRUSS MEMBERS – TRIAL SELECTION TABLES.

To facilitate the trial selection of truss component members, the following design tables are provided:

Table 5.3 HSS Bottom Chords
Table 5.4 WT Bottom Chords
Table 5.5 Class C HSS Top Chords
Table 5.6 Class H HSS Top Chords
Table 5.7 WT Top Chords
Table 5.8 Double Angle Tension Members
Table 5.9 Single Angle Tension Members
Table 5.10 Single Angle Struts* - with one leg welded to chords
Table 5.11 Double Angle Struts* - interconnected at mid length
Table 5.12 Class C HSS Warren Posts
Table 5.13 Class H HSS Warren Posts
Table 5.14 I_s/h^2 values – HSS chords
Table 5.15 I_s/h^2 values – WT chords

*Note: Angles in compression included in design tables belong to section classification up to 3 only.

The following is a detailed explanation of symbols which are used in the above mentioned design tables.

b_1 = Effective width of concrete slab (Clause 17.3.2 of S16.1) used in computing values of M_{rc} and I_g, in millimetres.
C_r = Factored axial compressive resistance of a concentrically loaded web member, in kilonewtons. (Resistance to torsional-flexural instability of a double angle strut has been computed by means of an equivalent radius of gyration method.)
C_{re} = Factored axial compressive resistance of a single angle web member with one leg welded to chords, in kilonewtons (force resultant assumed to act at centroid of attached leg but half the moment caused by such eccentricity is included in the computation, $K = 0.9$) (for explanation, see 5.8 Floor Design Example).
D = Overall depth of steel truss, in millimetres.
h = Effective depth of steel truss (vertical distance between centroids of steel chords), in millimetres.
I_s = Moment of inertia of steel truss, 15% reduction due to open web included, in 10^6 mm^4.
I_g = Gross moment of inertia of composite truss, neglecting effects due to open web, deck profile and concrete creep, in 10^6 mm^4.
L = Truss span, in millimetres.
L' = Laterally unsupported length of steel top chord while placing deck, in millimetres.
M_{rc} = Factored moment resistance of composite truss, in kilonewton metres (refer to Clause 17.4.3(a) of S16.1).
M_{rx} = Factored moment resistance about the x-x axis (as identified in Table 5.11), in kilonewton metres.
p = Top chord panel width defined as the horizontal distance between panel points (where top chord to 'Warren Post' intersection is also a panel point), in millimetres.
r_y = Radius of gyration about the y-y axis (as identified in the diagrams), in millimetres.
T_r = Factored axial tensile resistance, $T_r = \phi A F_y$, in kilonewtons.
V_h = Total horizonal shear to be resisted by shear connectors distributed between point of maximum moment and each adjacent point of zero moment, for full shear connection, in kilonewtons ($V_h = \phi A_b F_y$, where A_b is the bottom chord area).
V_r = Factored shear resistance, in kilonewtons ($V_r = 0.66 F_y \phi A_w$, where A_w = shear area).
w_{r1} = Factored top chord resistance to total u.d.l. while placing steel deck, in kilonewtons per metre.
w_{r2} = Factored top chord resistance to total u.d.l. while placing concrete, in kilonewtons per metre.

5.8 FLOOR DESIGN EXAMPLE

The designs of two composite truss configurations are illustrated in this section to represent a typical floor truss (T1 as indicated in the typical floor plan), shown in Figure 5.E1. To best illustrate the mechanics of the design process, the loadings and criteria are assumed to be the same as described in Section 4.14 except as noted below:

Storey heights given
- floor to floor height = 3 640 mm
- floor to ceiling height = 2 590 mm

- plenum depth = 1 050 mm
- maximum horizontal duct depth = 390 mm
  (after deducting depths of chord members and stiffening member and sprayed fire protection material, Fig. 5.E1)
- Out to out depth of steel truss, D = 730 mm

Solution:

Deck-slab system selected:

76 mm wide-rib profile deck manufactured by a different deck producer than the deck shown in Example 4.14. A normal density (20 MPa) cover slab of 65 mm thick is assumed.

$q_d = 0.1$ kPa $\qquad I_d = 1.25 \times 10^6$ mm^4 $\qquad q = 2.4$ kPa

$$
\begin{aligned}
w &= (1 + 0.2\, w_c s^4 / I_d)\, s\, q \qquad \text{(see Table 3.1)}\\
&= [1 + 0.2\,(2\,300)\,(3)^4 / (1.25 \times 10^6)]\, s\, q\\
&= 1.03\,(3)\,(2.4)\\
&= 7.42 \text{ kN/m}
\end{aligned}
$$

$W_c = (7.42 + 0.6)\,(11.5) = 92.2$ kN $\quad$ (assumed steel member = 0.6 kN/m)
$W_L = 68.7$ kN ; see section 4.14
$W_p = 41.4$ kN ; see section 4.14
$W_{OD} = 24.2$ kN ; see section 4.14

$$
\begin{aligned}
W_f &= \alpha_L W_L + \alpha_D (W_c + W_p + W_{OD})\\
&= 1.5(68.7) + 1.25\,(92.2 + 41.4 + 24.2)\\
&= 300 \text{ kN}\\
M_f &= W_f L/8 = 300\,(11.5)/8 = 431 \text{ kN·m}\\
V_f &= W_f/2 = 150 \text{ kN}
\end{aligned}
$$

Composite Truss with HSS Chords and Angle Webs

(a) Trial Section:

Bottom chord (HSS system)
Compute effective slab width (assuming width of top chord = 76 mm)

$L/4$	$= 11\,500/4$	= 2 875 mm
$16t_o + b$	$= 16\,(141) + 76$	= 2 332 mm (governs)
beam spacing		= 3 000 mm

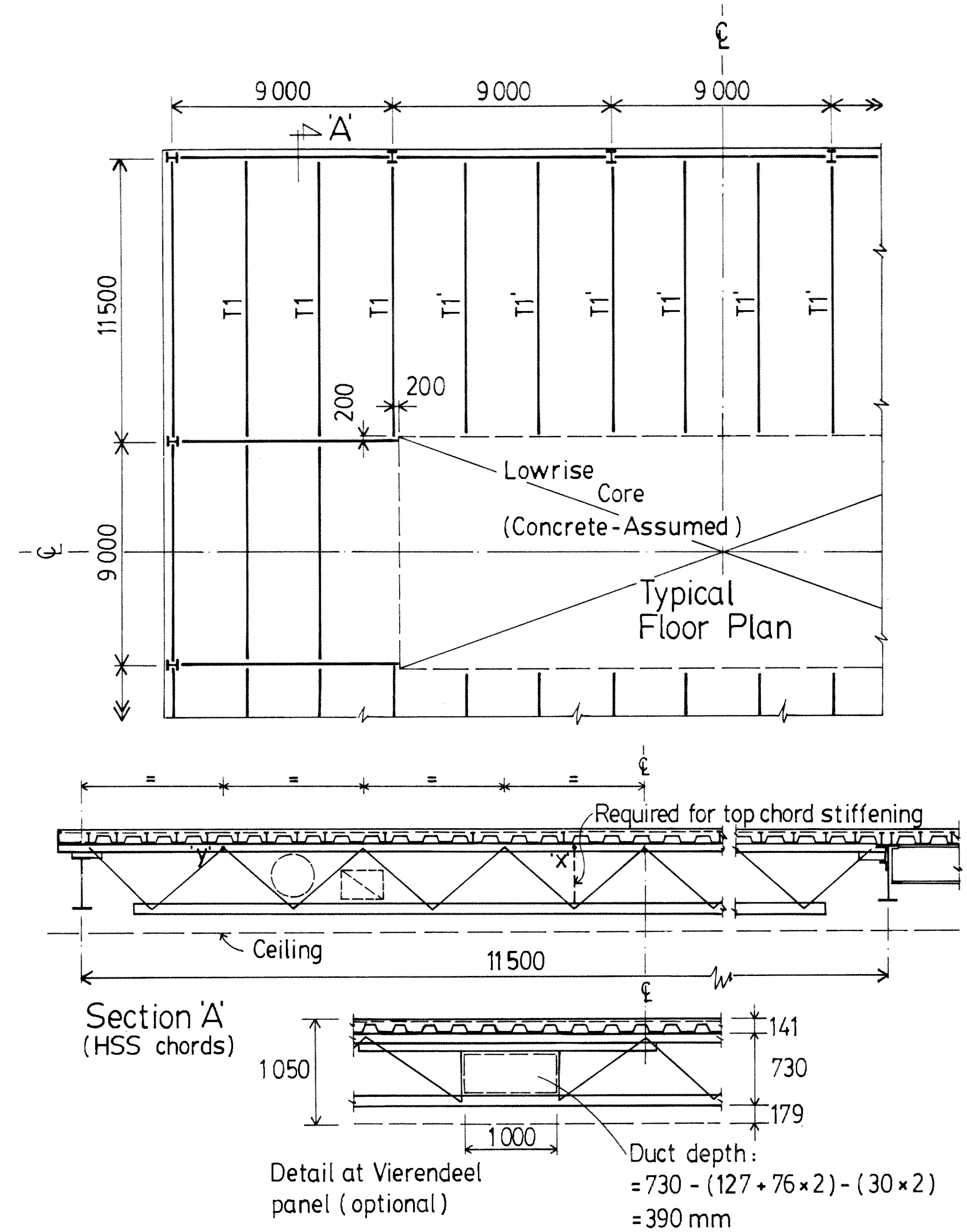


Figure 5.E1
Floor Design Example Key Plan
(Composite Truss Floor)

Therefore effective slab width, b_l = 2 332 mm

Using Table 5.3 – Composite Truss Bottom Chord
– trial selection table (HSS Bottom Chords),

for D = 730 mm, t_d = 76 mm, t_c = 65 mm
b_l = 2 330 mm and using **HSS127×76.2×4.78,**

and since tables do not provide a value for D = 730,

$$M_{rc} \text{ may be interpolated as } 460 - \frac{(460-404)(750-730)}{(750-650)} = 449 \text{ kN·m}$$

$$> 431 \text{ kN·m} \qquad \text{OK}$$

V_h = 564 kN

$$I_g \text{ may be interpolated as } \left[1\,020 - \frac{(1\,020-784)(750-730)}{(750-650)} \right] \times 10^6$$

or $I_g = 973 \times 10^6$ mm^4

Top chord (HSS system)
A diagonal angle of approximately 45 degrees is desirable
11 500/(730×2) = 7.9 – use 8 panel points
Select top chord panel width at 11 500/8 = 1 437.5 mm

– Construction Stage 1 – Deck placement

A = 11.5 × 3 = 34.5 m^2
27 m^2 < A < 54 m^2
q_L' = 0.7 – A/135 (Table 3.2 requires q_L' to be linearly
= 0.44 kPa varied from 0.5 to 0.3 kPa, when
54 > A > 27)

Dead load (deck + truss steel) = 0.3 + 0.6 = 0.9 kN/m
Factored total load at deck placement,
w_{f1} = 1.25 (0.9) + 1.5 (0.44)(3) = 3.11 kN/m

Using linear interpolation from Table 5.5, for trial top chord section **HSS76.2×76.2×6.35**, of truss span 11.5 m, lateral support of top chord at 1/3 span, panel width (p) = 1 438 mm, top to bottom chord centroidal distance (h) = 730–0.5(127+76) = 628.5 mm, p/h = 2.29, the values of w_{r1} and w_{r2} can be obtained as 5.15 and 12.9 kN/m respectively. See computation below:

The following calculation is intended to show how to obtain w_{r2} value from Table 5.5 by interpolation, when span = 11.5 m, h = 628.5 mm, p/h = 2.29

Span = 11 , h = 550 , p/h = 1 , w_{r2} = 18 (from table)
Span = 13 , h = 550 , p/h = 1 , w_{r2} = 13 (from table)

Interpolating, when span = 11.5 , h = 550 , p/h = 1 ,
then, w_{r2} = 18 – (18–13)(11.5–11)/(13–11) = 16.75 kN/m
Similarly, when span = 11.5 , h = 700 , p/h = 1 ,
w_{r2} = 21.6 – (21.6–15.8)(11.5–11)/(13–11) = 20.15 kN/m
and, when span = 11.5 , h = 550 , p/h = 2.5 ,
w_{r2} = 13 – (13–9.8)(11.5–11)/(13–11) = 12.2 kN/m

and, when span = 11.5 , h = 700 , p/h = 2.5 ,
w_{r2} = 12.5 – (12.5–9.8)(11.5–11)/(13–11) = 11.83 kN/m

Interpolating using values obtained above,
when span = 11.5 , h = 550 , p/h = 2.29
w_{r2} = 16.75 – (16.75–12.2)(2.29–1)/(2.5–1) = 12.8 kN/m
and when span = 11.5 , h = 700 , p/h = 2.29
w_{r2} = 20.15 – (20.15–11.83)(2.29–1)/(2.5–1) = 13 kN/m

again, interpolating for value of w_{r2}, when
span = 11.5 , h = 628.5 , p/h 2.29

w_{r2} = 12.8 + (13–12.8)(628.5–550)/(700–550) = 12.9 kN/m

Since maximum top chord force is calculated at location 'x', Fig. 5.E1, instead of at the mid-span, the values of w_{r1} and w_{r2} may be multiplied by the ratio (moment at mid-span / moment at location 'x'). Thus,

Factored u.d.l. for deck placement = w_{r1} = 5.22 kN/m, and
Factored u.d.l. for slab placement = w_{r2} = 13.1 kN/m.

Since w_{r1} (= 5.22 kN/m) is greater than w_{f1} (= 3.11 kN/m) (calculated above), the top chord is satisfactory for use under the deck placement loading.

– Construction Stage 2 – concrete placement

$q_L = q_L'$ = 0.88 kPa
Dead load (truss steel + deck + concrete) = 0.6 + 7.42 = 8.02 kN/m

Factored total load at concrete placement,
w_{f2} = 1.25(8.02) + 1.5(0.88)(3) = 14.0 kN/m > (w_{r2} = 13.1)

Since top chord panel width may be modified by web framing changes, or connection details, HSS76.2×76.2×6.35 is kept for more detailed analysis, as shown later.

Web members (Double angle struts without spacers or battens)
Using tables 5.8 and 5.10, the web members (shown in Fig. 5.E2) are selected for the factored axial loads and are tabulated in the table below:

Web Member Reference	Factored Axial Force (kN) (–ve = tension)	Factored axial Resistance (kN) (C_r or T_r)	Section Selected using Tables 5.8 and 5.10 (Theoretical length 955 mm)
A	– 199	T_r = 224	2L45×30×6
B	+ 199	C_r = 201	2L55×55×10
C	– 143	T_r = 175	2L35×35×5
D	+ 143	C_r = 167	2L55×55×8
E	– 85.6	T_r = 121	2L25×25×5
F	+ 85.6	C_r = 112	2L55×55×5
G	– 28.6*	T_r = 121, C_r = 18.9	2L25×25×5
H	+ 28.6	C_r = 44	2L35×35×5

*Note: Possibility of force reversal during erection (6 kN), from results of unbalanced load analysis.

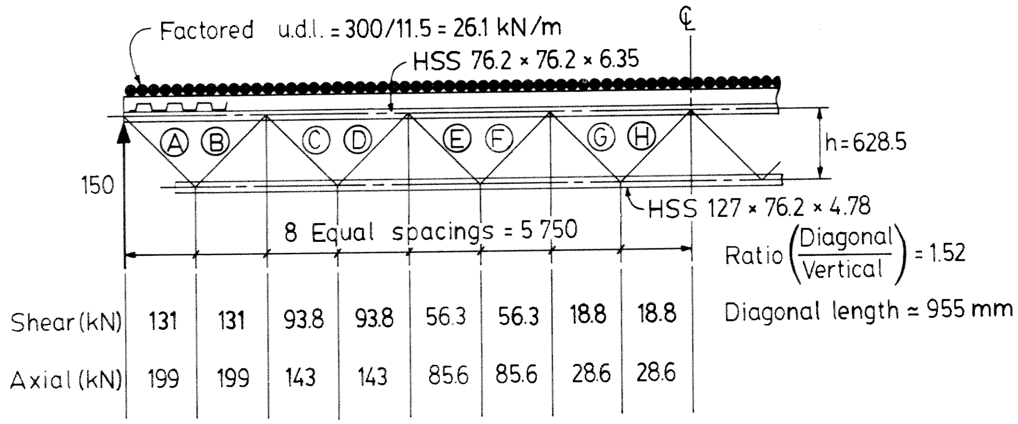


Figure 5.E2
Computation of Factored Web Forces for Preliminary Design (HSS chords)

(b) Truss Framing Layout and Truss Modelling:

Following the trial selection, a scaled layout of truss members is constructed (as shown in Fig. 5.E3) to ensure simplicity in connection details and to evaluate the amount of connection eccentricity at each web to chord joint.

With the scaled truss layout, structural analysis models may be constructed and analysed using a computer. Two basic truss models are constructed and analysed, as shown in Fig. 5.E4(a) and Fig. 5.E4(b):

- Non-composite steel truss model (see Fig. 5.E4a)
  (to be analysed under construction factored loads caused by deck placement and concrete placement)
- Composite truss model (see Fig. 5.E4b)
  (to be analysed under occupancy factored load and specified floor live load with superimposed dead loads) (Also see Fig. 5.E5 for exaggerated deflected truss shape)

(c) Detailed Member Design (bottom chord)

$$\begin{aligned}\text{Out to out composite truss depth} &= t_o + D\\ &= 141 + 730\\ &= 871 \text{ mm}\end{aligned}$$

Using bottom chord section HSS127×76.2×4.78 (A_s = 1 790 mm²), and checking moment resistance of composite section (see formulas 5.1 and 5.2, Fig. 5.1),

$$a = \frac{\phi\, A_s\, F_y}{0.85\, \phi_c\, f_c'\, b_1} = \frac{0.9(1\ 790)(350)}{0.85\ (0.60)(20)(2\ 330)} = 23.7 \text{ mm} < t_c$$

$$\begin{aligned}M_{rc} &= e'\, \phi\, A_s\, F_y\\ &= [871 - (127+23.7)/2](0.9)(1\ 790)(350)\ 10^{-6}\\ &= 449 \text{ kN·m} > (M_f = 431 \text{ kN·m})\\ V_h &= \phi\, A_s\, F_y = 564 \text{ kN} \qquad \text{(same as before)}\end{aligned}$$

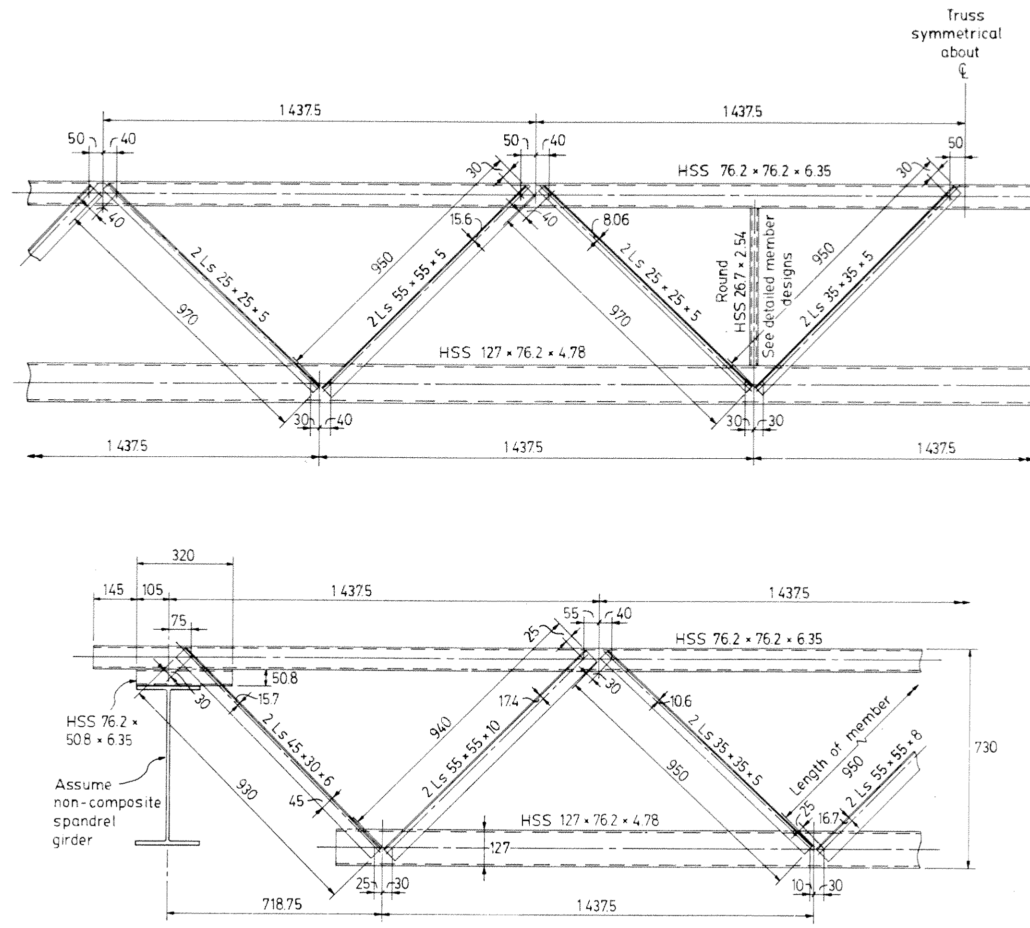


Figure 5.E3
Truss Framing Layout (Truss T1)
(HSS Chords)

(d) Detailed Member Design (top chord)

Five load cases are included in the computer analysis of the non-composite truss model:

Case 1 : factored total u.d.l. = 3.11 kN/m

Case 2 : factored dead u.d.l. = 1.13 kN/m, plus
factored live load = 6 kN, acting at fourth panel of top chord from the left support and distributed for a length of 0.3 metre.

Case 3 : factored total u.d.l. = 14.0 kN/m

Case 4 : factored dead u.d.l. = 10.0 kN/m
factored live load = 6 kN, acting at fourth panel of top chord from the left support and distributed for a length of 0.3 metre.

Case 5 : specified dead load (at concrete placement) = 8.02 kN/m
(results will be discussed in (f))

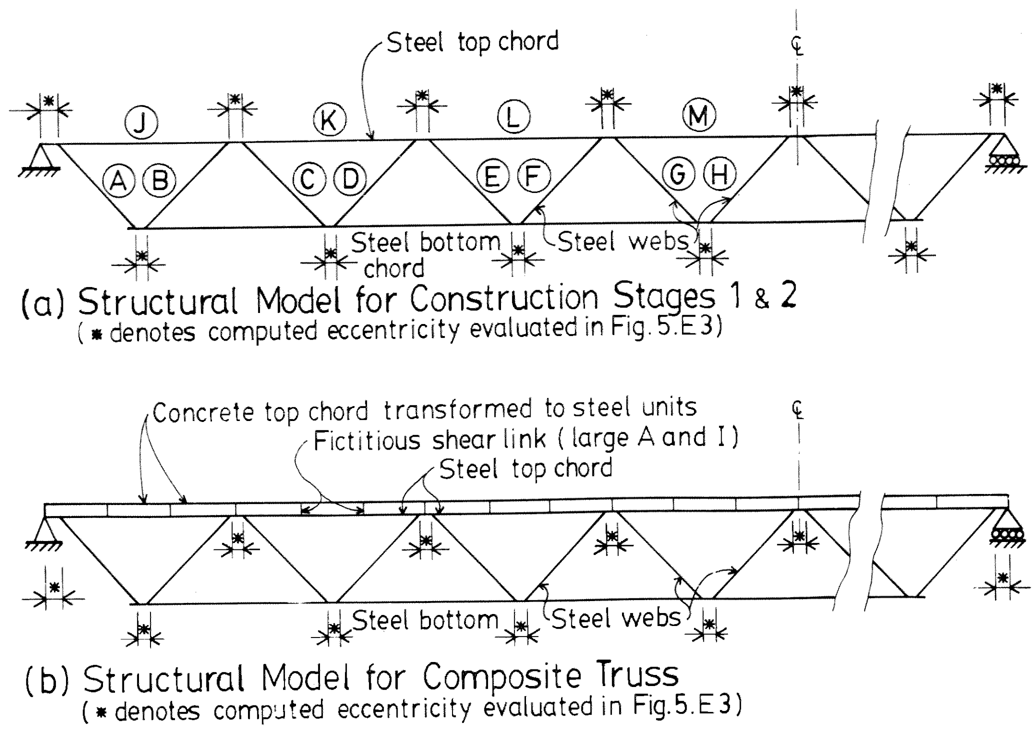


Figure 5.E4
Structural Modelling for Truss T1

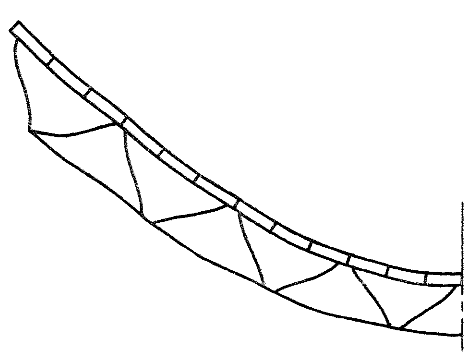


Figure 5.E5
Deflected Shape of Composite Truss
(Exaggerated to Show Member Curvature)

Member forces at critical top chord panels under load cases 1 to 4 are tabulated as follows:

Load Case	Load Description	Critical top chord member	Member Design Forces: Factored Axial (kN) C_f	Factored Moment (kN·m) at mid panel M_{fm}	Factored Moment (kN·m) (max) at support M_{fs}
1	deck placement	M J	78.8 17.8	0.494 1.06	0.29 1.15
2	deck placement	M	52.5	1.49	0.58
3	concrete placement	M J	365 82.4	2.29 4.89	1.34 5.34
4	concrete placement	M	287	2.96	1.45

Design checks for top chord panel M (HSS76.2×76.2×6.35)

For load Cases 1 and 2:

– panel width (conservatively assumed) $= \dfrac{11\,500}{8} = 1\,438$ mm

– lateral support (provided by bridging or by direct deck-to-top chord connection) = 11 500/3 = 3 833 mm (governs)

effective length $= Kl_y = (1.0)(3\,833) = 3\,833$ mm

$$\frac{Kl_y}{r_y} = \frac{3\,833}{28} = 137$$

For 350W steel, $\dfrac{C_r}{A} = 83.1$ MPa, (Handbook, Table 4-3).

$C_{r(kl)} = 83.1\ A = (83.1)(1\,670)\ 10^{-3} = 139$ kN
$M_r = 13.5$ kN·m (Handbook, Page 4-61)
$C_{ro} = 526$ kN

effective length $Kl_x = (0.9)(1\,438) = 1\,294$ mm (K = 0.9, interior panel)

$$\frac{Kl_x}{r_x} = \frac{1\,294}{28} = 46.2\ , \quad \frac{C_e}{A} = 923\ , \quad C_e = 923\ A = 1\,541 \text{ kN}$$

$$\frac{C_f}{C_e} = \frac{78.8}{1\,541} = 0.05\ , \quad U = 1.05$$ (Handbook, Table 4-9) (for Case 1 loading)

Utilization of combined axial and bending resistance (for Case 1 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{78.8}{139} + \frac{0.494\ (1.0)(1.05)}{13.5} = 0.61 < 1.0$$

$$\frac{C_f}{C_e} = \frac{52.5}{1\ 541} = 0.034 \quad , \quad U = 1.04 \qquad \text{(for Case 2 loading)}$$

Utilization of combined axial and bending resistance (for Case 2 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{52.5}{139} + \frac{1.49\ (1.0)(1.04)}{13.5} = 0.49 < 1.0$$

For load Cases 3 and 4: (top chord is laterally supported by deck to chord connections.)

For 350W steel, $\frac{C_r}{A} = 260$ MPa (for $Kl_x/r_x = 46.2$)

$C_{r(kl)} = 260\ A = (260)(1\ 670)\ 10^{-3} = 434$ kN

$C_{ro} = 526$ kN , $M_r = 13.5$ kN·m

$$\frac{C_f}{C_e} = \frac{365}{1\ 541} = 0.24 \quad , \quad U = 1.32 \qquad \text{(for Case 3 loading)}$$

Utilization of combined axial and bending resistance (for Case 3 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{365}{434} + \frac{2.29(1.0)(1.32)}{13.5} = 1.06 > 1.0 \text{ fail}$$

$$\frac{C_f}{C_e} = \frac{287}{1\ 541} = 0.19 \quad , \quad U = 1.23 \qquad \text{(for Case 4 loading)}$$

Utilization of combined axial and bending resistance (for Case 4 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{287}{434} + \frac{2.96\ (1.0)(1.23)}{13.5} = 0.93 < 1.0$$

Top chord failed code check under load Case 3 (slab placing under the total u.d.l. loading). The designer may chose one of several options to continue the truss design:

- increase the size of top chord to HSS76.2×76.2×7.95
- introduce vertical post to support top chord panel M at location 'x' (Fig. 5.E1)
- rearrange web framing to reduce width of panel M

Let us assume the use of vertical post to modify the Warren framing under the panel M.

Tributary floor area for the computation of maximum axial force in the vertical post may be approximated as $(11.5)(3)/16 = 2.2\ m^2$.

Total factored axial force

$$= (2.2)\frac{W_c + W_p + W_{OD}}{(11.5)(3)}(1.25) + (9)(1.5)$$

$$= 12.6 + 13.5 = 26.1 \text{ kN}$$

Note that NBC minimum specified concentrated load of 9.0 kN, applied over any area of 0.75 m × 0.75 m has been used as the second term in the above equation. (NBCC Table 4.1.6.B)

Using Table 5.12, $Kl = (730 - 76.2 - 127) \simeq 530$ mm

Round Hollow Section of 26.7 OD is OK for use as a support post. See Fig. 5.E3 for final detail.

It can be shown by further analysis and design check that the utilization of combined axial and bending resistance of top chord member M is reduced to less than 1.0, when the vertical post is introduced.

Similar design checks may be made to top chord panel L (Fig. 5.E4a), without the use of a vertical post. The utilization of combined axial and bending resistance of the top chord member, under the critical load Case 3, can be shown as follows:

$$\frac{C_f}{C_e} = \frac{318}{1\ 541} = 0.21 \quad , \quad U = 1.27$$

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{318}{434} + \frac{2.3\ (1.0)(1.27)}{13.5} = 0.95 < 1.0$$

Design checks for top chord panel J

For load Case 1:
- panel width (conservatively assumed) = 1 438 − 75 = 1 363 mm
- lateral support (same as for top chord panel M) = 3 833 mm

Therefore, $C_{r(kl)} = 139$ kN (same as for top chord panel M)
$M_r = 13.5$ kN $C_{ro} = 526$

$$\frac{Kl_x}{r_x} = \frac{(1.0)(1\ 363)}{28} = 48.7 \qquad K = 1.0 \text{ (end panel)}$$

$$\frac{C_e}{A} = 830 \quad , \quad C_e = 830(1\ 670)10^{-3} = 1\ 386 \text{ kN}$$

$$C_f = \frac{17.8}{1\ 386} = 0.013 \quad , \quad U = 1.01$$

Utilization of combined axial and bending resistance (for Case 1 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{17.8}{139} + \frac{1.15\ (1.0)(1.01)}{13.5} = 0.21 < 1.0$$

For load Case 3:

$$\frac{C_r}{A} = 255 \quad , \quad C_{r(kl)} = 255(1\ 670)\ 10^{-3} = 426 \text{ kN}$$

$$\frac{C_f}{C_e} = \frac{82.4}{1\ 386} = 0.06 \quad , \quad U = 1.06$$

Utilization of combined axial and bending resistance (for Case 3 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\ \omega\ U}{M_r} = \frac{82.4}{426} + \frac{5.34\ (1.0)(1.06)}{13.5} = 0.61 < 1.0$$

Check maximum end shear at truss supports

V_f = 150 kN , as shown before
$V_r = \phi\ (0.66)\ F_y\ C_{rt}$
where C_{rt} = Shear constant (mm^2) See Handbook
(or effective shear area)
$V_r = 0.9\ (0.66)(350)(645)(10^{-3})$
= 134 kN ($< V_f$) (assuming top chord section only)
See shoe detail provided in Fig. 5.E3 (Therefore $V_r > V_f$).

Check maximum shear at location 'y' (See Fig. 5.E1)

$$V_f \simeq 150\left(\frac{3}{4}\right) = 113\ \text{kN}$$

V_r = 134 kN OK

The design checks provided up to this stage show that the top chord is satisfactory for construction load conditions (Cases 1 to 4) as well as for occupancy load condition (with respect to ultimate shear resistance).

With the selected top chord member of HSS76.2×76.2×6.35 the maximum diameter of shear studs permitted by Clause 17.3.5.5 of S16.1 can be calculated as 2.5t, or **stud diameter** = 15.9 mm

The number of studs (based on single stud per flute) per truss = $2V_h/q_r$ = (2)(564/51.6) = **22 studs** (q_r value obtained from Table 2.1) (See Fig. 5.E1).

(e) Detailed Member Designs (Web members)

S16.1 Clause 17.3.1.2 states that web steel members of composite joist (or truss) shall be proportioned to carry the total vertical shear, V_f. In addition, section 5.4d of this publication has identified the types of local moments to be included in the design of web members in compression. However, during the design of web members in tension, such local moments are neglected, as noted in the text.

In the following example calculations, two typical web members are design checked. Diagonal A consists of a pair of single angles in tension, and diagonal B consists of a pair of single angles in compression. In either case, the angles are end-connected on one leg by welding, but are not battened nor connected by spacers between supports. The design of fillet weld web-to-chord connections is also included in order to provide a complete picture of connected web members. All design procedures, assumptions and details are illustrated and are assumed to be appropriate only to this type of steel web members.

– **Diagonal 'A'** (see Fig. 5.E6)

T_f = 199 kN (as shown previously)
M_{fx} = 0.66 kN·m (from stiffness analysis, composite model)
(under factored occupancy loading)
M_{fy} due to out of plane eccentricity (ignored as explained)

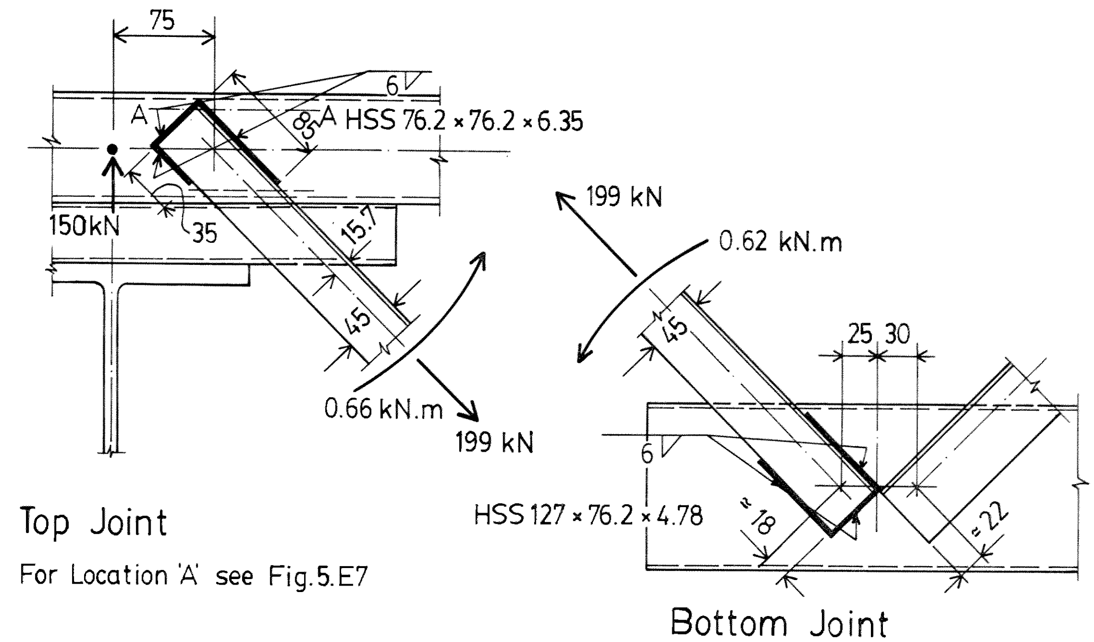


Figure 5.E6
Detail at Diagonal 'A'

Section previously selected consists of 2 angles L45×30×6
A_s = 828 mm² $S_x = 5.58 \times 10^3$ mm³
$T_r = 0.9\ A_s\ F_y = 0.9(828)(0.300)$ = 224 kN
$M_{rx} = 0.9\ S_x\ F_y = 0.9(5.58)(0.300)$ = 1.51 kN·m

Utilization of factored member resistance

$$\frac{T_f}{T_r} = \frac{199}{224} = 0.89 < 1.0$$

Since stiffness analysis of the composite model has been performed based on rigid-joint basis whilst the axial tension of 199 kN was obtained using a pin-jointed model, the utilization calculation may be computed based on tensile values only provided that the top chord alone is able to resist the total bending moment caused by joint eccentricity ($M_f = 150 \times 0.075 = 11.25$ kN·m, M_r for HSS76.2×76.2×6.35 = 13.5 kN·m, see Fig. 5.E6).

– Welding at ends of diagonal 'A' (see Fig. 5.E6)
The following calculation is provided to illustrate the design-check-methodology of weldment using a simple and conservative approach.

Upper joint of diagonal 'A'

Figure 5.E6 provides assumed detail of fillet weld for the joint along with the applied member forces. For the assessment of weld resistance, portion of weld material above the assumed line AA is neglected for effects of corner radius of the HSS top chord; see Fig.5.E7.

C.G. of weld group about 'x-x' may be computed as,

$$\frac{(40)^2/2+(70)(45)}{40+70+35} = 27.2 \text{ mm}$$

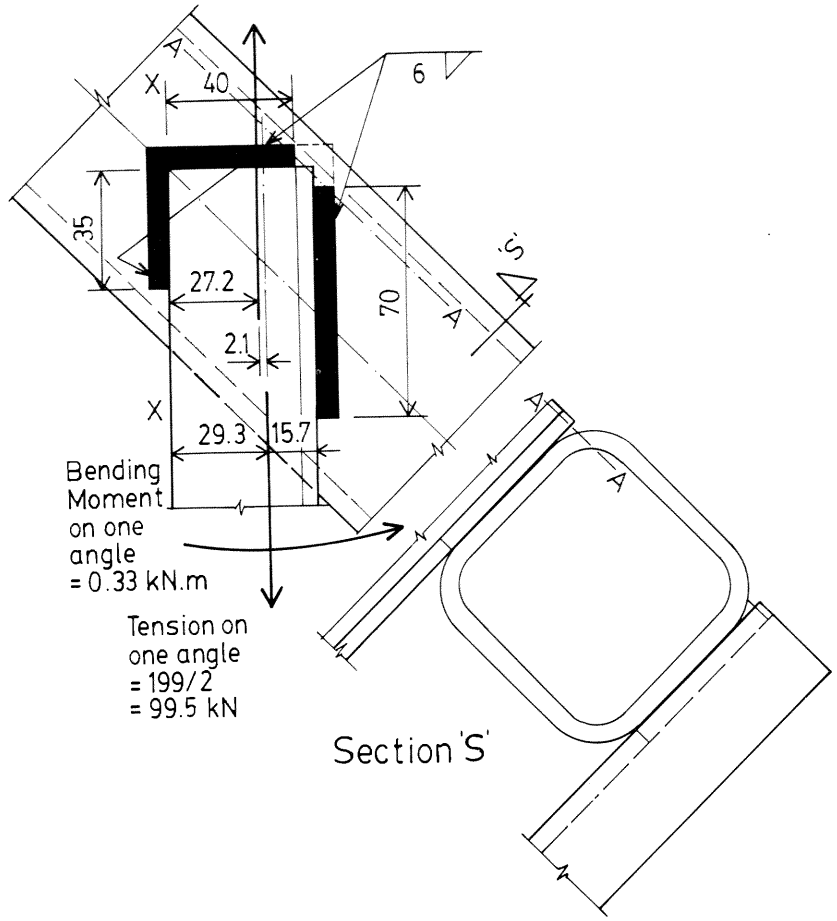


Figure 5.E7
Design of Fillet Welds for Diagonal 'A' (Upper Joint)

To estimate maximum factored moment on weld group, let us consider the following two assumptions:

Case A Bending moment acting on the weld group (without the consideration of local moment) caused by the eccentricity of the weldment,

$$M_{f1} \text{ (on one angle)} = \frac{199}{2}(2.1)\ 10^{-3} = 0.209 \text{ kN·m}$$

Case B Bending moment acting on the weld group caused by both the eccentricity of the weldment and the effect of local moment (from stiffness analysis of composite model assuming rigid-end joints),

$$M_{f2} \text{ (on one angle)} = \frac{0.66}{2} - \frac{199}{2}(2.1)\ 10^{-3} = 0.12 \text{ kN·m}$$

Let us conservatively assume that the maximum factored moment acting on the weld group is equal to the larger of the values computed above; $M_f = 0.209$ kN·m.

$$\text{Factored tensile force on one angle} = \frac{199}{2} = 99.5 \text{ kN}$$

Using 6 mm fillet weld, the factored shear resistance per millimetre of weld length can be found from the Handbook of Steel Construction, Table 3-24, as 0.918 kN.

Hence total factored shear resistance of weld group is computed as,
$(40+70+35)(0.918) = 133$ kN

Considering only 35 mm of fillet weld on each side of steel angle (angle leg width = 45 mm), total factored moment resistance of the assumed weld group may be approximated as,

$$0.918(35)(45)\ 10^{-3} = 1.45 \text{ kN·m}$$

Utilization of weld resistance under the combined shear and moment may be computed as,

$$\frac{99.5}{133} + \frac{0.209}{1.45} = 0.89 < 1.0 \qquad \text{OK}$$

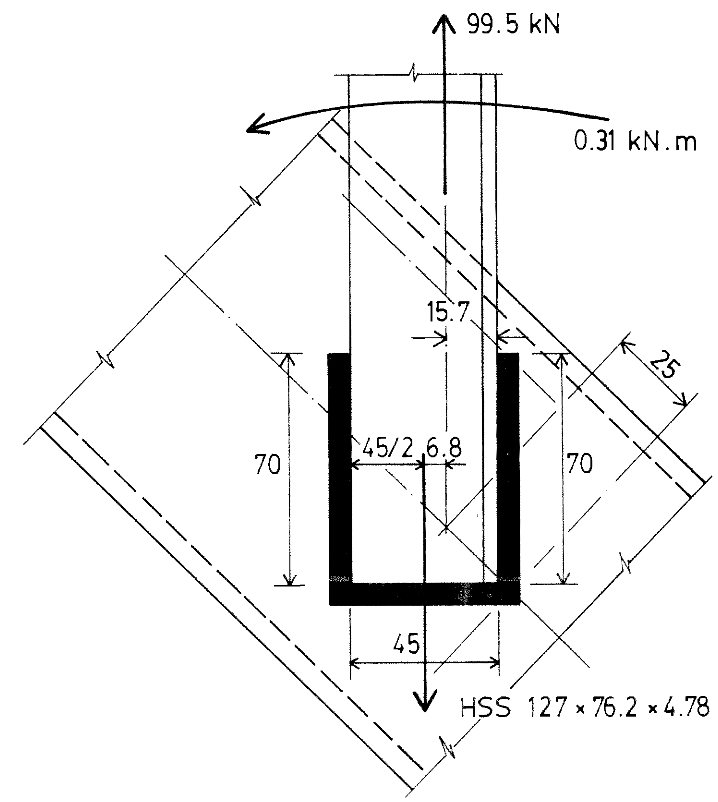


Figure 5.E8
Design of Fillet Welds for Diagonal 'A' (Lower Joint)

Lower joint of diagonal 'A'

Figure 5.E8 provides assumed detail of fillet weld along with applied member forces for one angle diagonal. To estimate maximum factored moment on the assumed weld group, let us consider the following two assumptions:

Case A Bending moment acting on the weld group caused by the eccentricity of weldment (without the consideration of local moment)

M_{f1} (on one angle) $= 99.5\ (6.8)\ 10^{-3} = 0.677$ kN·m

Case B Bending moment acting on the weld group caused by both the eccentricity of weldment and the effect of local moment (from stiffness analysis of composite model assuming rigid-end joints)

M_{f2} (on one angle) $= 0.677 + 0.31 = 0.987$ kN·m

Let us conservatively assume that the maximum factored moment acting on the weld group is equal to the larger of the values computed above; $M_f = 0.987$ kN·m

Factored tensile force on one angle = 99.5 kN

Using 6 mm fillet weld, total factored shear resistance of weld group can be computed as,

$$\left[(70)(2) + 45\right](0.918) = 170 \text{ kN}$$

Estimated factored moment resistance of two lines of 6 mm weld of length 70 mm each, spaced 45 mm apart may be computed as

$$(70)(0.918)(45)\ 10^{-3} = 2.89 \text{ kN·m}$$

Utilization of weld resistance under the combined shear and moment may be computed as,

$$\frac{99.5}{170} + \frac{0.987}{2.89} = 0.93 < 1.0 \qquad \text{OK}$$

Note that for the design of weld groups, the effect of out-of-plane bending (due to end connection to one leg of angle) may be neglected for long members. See section 5.9 of Reference (5.16)

– **Diagonal 'B'** (see Fig. 5.E9)

Section previously selected consists of two angles L55×55×10 (batten or spacer not used). Factored forces, acting at ends of one angle of diagonal 'B', are shown in Fig. 5.E10; and they are as summarized below:

$C_f = 99.5$ kN
$M_{fs(top)} = 1.84$ kN·m $\qquad M_{fs(bottom)} = 1.13$ kN·m
$M_{fz(top)} = 0.257$ kN·m $\qquad M_{fz(bottom)} = -\ 0.449$ kN·m
Member curvature about s-s axis = single
Member curvature about z-z axis = double

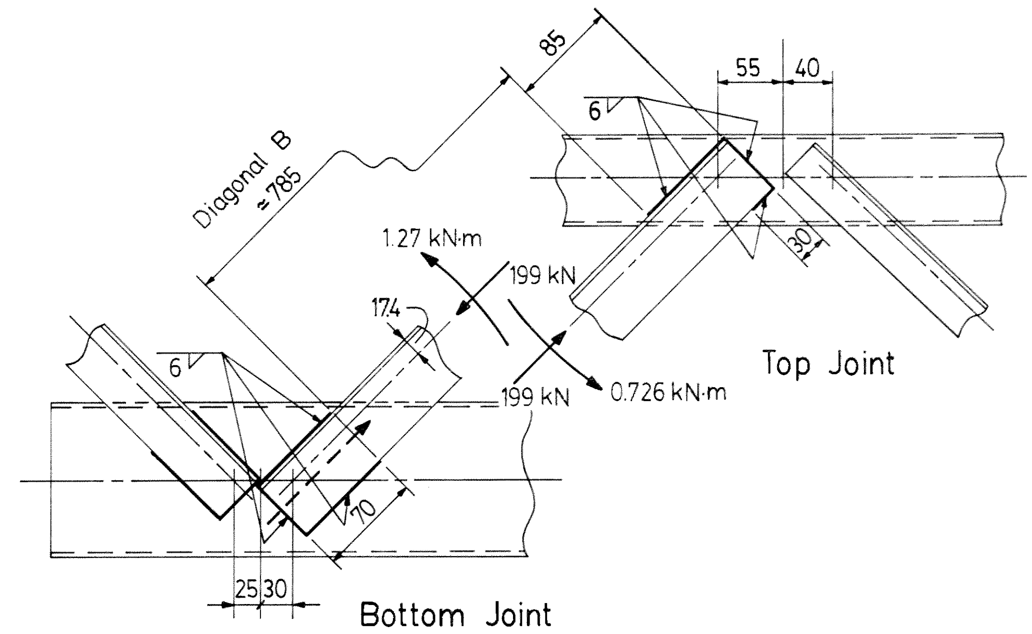


Figure 5.E9
Detail at Diagonal 'B'

Member properties: L55×55×10 (not a class 4 section, see 5.7)

Kl	= clear length	= 785 (say)		
A_s	$= 1\,000$ mm²	$r_x = r_y = 16.4$ mm		$r_z = 10.7$ mm
y	= 17.4 mm			
r_s^2	$= 2\,r_x^2 - r_z^2$	= 423	(for equal leg angles)	
I_s	$= r_2^2\,A_s$	$= 0.423\times10^6$ mm⁴		
S_s	$= I_s/38.9$	$= 10.9\times10^3$ mm³	(see Fig. 5.E10)	
I_z	$= r_z^2\,A_s$	$= 10.7^2\,(A_s) = 0.114\times10^6$ mm⁴		
S_z	$= I_z/14.3$	$= 7.97\times10^3$ mm³ (with respect to point 'A')		

Factored resistances calculations:

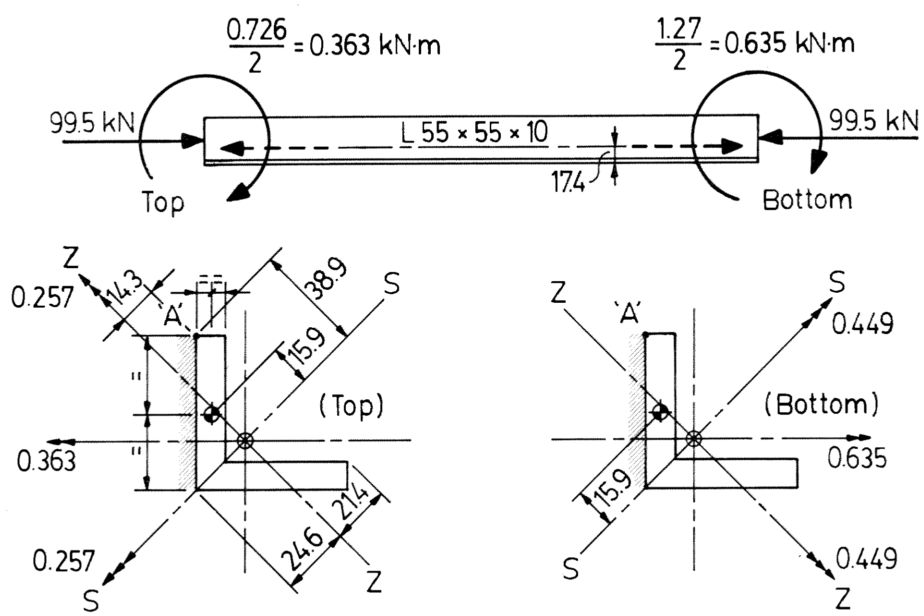
$$\frac{Kl}{r_z} = \frac{785}{10.7} = 73 \quad , \quad C_{ez} = 370A = 370 \text{ kN}$$
(Table 4-8 of Handbook)

$C_{ro} = 0.9\,(0.3)(1\,000) = 270$ kN
$C_{rz} = 182$ kN $\qquad$ (Use Table 4-3 of Handbook)
$M_{rs} = 0.9(0.3)(S_s) = 2.94$ kN·m $\qquad$ with respect to point 'A'
$M_{rz} = 0.9(0.3)(S_z) = 2.15$ kN·m

$$\frac{Kl}{r_s} = \frac{785}{20.6} = 38 \quad , \quad C_{es} = 1\,364 \text{ kN}$$

$$\frac{C_f}{C_{ez}} = \frac{99.5}{370} = 0.27 \quad , \quad U_z = (1-0.27)^{-1} = 1.37$$

$$\frac{C_f}{C_{es}} = \frac{99.5}{1\,364} = 0.07 \quad , \quad U_s = (1-0.07)^{-1} = 1.08$$



Moment about SS axis
$= 0.257 + 99.5(15.9)10^{-3}$
$= 1.84$ kN·m

Moment about ZZ axis
$= 0.257$ kN·m

Moment about SS axis
$= 99.5(15.9)10^{-3} - 0.449$
$= 1.13$ kN·m

Moment about ZZ axis
$= -0.449$ kN·m

Note: +ve moment produces compression at location 'A'

Figure 5.E10
Factored Forces on One Angle of Diagonal 'B'

$$\omega_s = 0.6 + 0.4\frac{1.13}{1.84} = 0.85 \qquad \text{(Single curvature)}$$

$$\omega_z = 0.6 - 0.4\frac{0.257}{0.449} = 0.37 \text{ (i.e 0.4)} \qquad \text{(Double curvature)}$$

Check Strength*:

$$\frac{99.5}{270} + \left(\frac{1.84}{2.94} + \frac{0.257}{2.15}\right)/2 = 0.74 < 1.0 \qquad \text{OK (top joint)}$$

$$\frac{99.5}{270} + \left(\frac{1.13}{2.94} - \frac{0.449}{2.15}\right)/2 = 0.46 < 1.0 \qquad \text{OK (bottom joint)}$$

Check Stability*:

$$\frac{C_f}{C_{rz}} + \left(\frac{\omega_s M_{fs} U_s}{M_{rs}} + \frac{\omega_z M_{fz} U_z}{M_{rz}}\right)/2$$

$$= \frac{99.5}{182} + \left[\frac{0.85(1.84)(1.08)}{2.94} + \frac{0.4(0.449)(1.37)}{2.15}\right]/2$$

$$= 0.89 < 1.0 \qquad \text{OK}$$

*The sum of utilization of moment resistances as shown above has been arbitrarily halved due to the following reasons:

- Although the axial compression is calculated based on pin-jointed members, the local bending at ends of diagonal member are obtained assuming that all joints are rigid. This is too conversative.
- Test results(5.17) on single angle struts of similar slenderness have shown that it is too conservative to assume full end moment (caused by connecting one leg of the angle strut).

Upper and Lower joints of diagonal 'B'

Same procedure shall be used for the sizing of weldments for diagonal 'B' as that given for diagonal 'A'. For finished detail, see Fig. 5.E9.

(f) Truss Deflection Estimate

Top chord	HSS76.2×76.2×6.35	$I_x = 1.31 \times 10^6$ mm^4
Bottom chord	HSS127×76.2×4.78	$I_x = 3.78 \times 10^6$ mm^4

Web member	A	2L45×30×6
	B	2L55×55×10
	C	2L35×35×5
	D	2L55×55×8
	E	2L25×25×5
	F	2L55×55×5
	G	2L25×25×5
	H	2L35×35×5

- Camber requirement

Using approximate calculation method and Table 5.14, steel truss moment of inertia I_s† may be calculated as,

$$I_s = 743\,(h^2) = 734\,[D-(76.2+127)/2]^2$$
$$= 289.8 \times 10^6 \text{ mm}^4$$

Total effective truss moment of inertia,

$$I = (289.8+1.31+3.78)\,10^6 = 295 \times 10^6 \text{ mm}^4$$

†Note, I_s values in Table 5.14 include a 15% reduction to allow for web deflection due to strain.

Specified dead load at concrete placement is estimated as 8.02 kN/m (see item (d))

$$\Delta_c = \frac{5}{384}\,\frac{8.02(11.5)^4}{200\,(295)}\,10^3 = 31 \text{ mm}$$

Using stiffness analysis computer run, based on rigid jointed steel truss members, Δ_c can be shown as 30.7 mm.

Therefore truss T1 is to be **cambered** at mid span for about **30 mm.**

– Deflection due to all specified superimposed loads including long term effects.

Using approximate calculation method

Step 1. Compute composite truss moment of inertia

I_g, from item (a), is given as 973×10^6 mm^4

Assume I loss for web deflection due to strain,
$I_{wr} \simeq$ 15% of I of truss chords
$= (0.15/0.85)\ 289.8\ (10^6)$
$= 51 \times 10^6$ mm^4

Therefore moment of inertia of composite truss,

$I_t \simeq (973-51) \times 10^6 = 922 \times 10^6$ mm^4

Step 2. Composite deflection due to all superimposed loads

$$= \frac{5}{384}\ \frac{(W_L+W_p+W_{OD})\ L^3}{E\ I_t}\ (1.0 + 15\% \text{ for slip} + 15\% \text{ for creep})$$

$$= \frac{5}{384}\ \frac{(68.7+41.4+24.2)\ 11.5^3}{200\ (922)}\ (10^3)\ 1.30$$

$$= 18.7 \text{ mm} \quad < \quad \frac{L}{300} = 38 \quad \text{OK}$$

Computer stiffness analysis of rigid joint truss model gives 18.5 mm

Composite Truss with WT Chords and Angle Webs

(a) Trial Selection

Bottom Chord (WT system)

Compute effective slab width (assuming width of top chord = 150 mm)

b_l is governed by $16t_o + b = 16(141)+150 = 2\ 406$ mm in this truss design, (see previous design).

Using Table 5.4 – Composite Truss Bottom Chord
– trial selection table (WT Bottom Chord)

for $D = 730$ mm, $t_d = 76$ mm, $t_c = 65$ mm.
$b_l = 2\ 410$ mm and using **WT125×16.5**

By interpolation, $M_{rc} = 468$ kN·m $> (M_f = 431)$ OK
$V_h = 562$ kN $\quad V_r = 140$ kN $> V_f = (W_f/2)-W_f/16 = 131$ kN OK

$y = 27.3$ mm

By interpolation, $I_g = 1\ 224 \times 10^6$ mm^4

Top Chord (WT system)

As computed before:
for deck placement, $w_{f1} = 3.11$ kN/m
for concrete placement, $w_{f2} = 14.0$ kN/m

Using Table 5.7 and **WT125×19.5** as top chord,
Truss span = 11 500 mm   lateral support spacing = span/2
panel width = 1 438 mm

Centroidal distance, $h = 730 - (27.3 + 26.7)$
$= 676$ mm

$p/h = 2.13$

w_{r1} may be interpolated as 5.3 kN/m (> 3.11) OK
w_{r2} may be interpolated as 15.4 kN/m (> 14.0) OK
$V_r = 154$ kN (> $W_f/2$ OK)

Web members (Double angle struts, interconnected at mid length)
(Double angle tension members)

Factored axial forces (under occupancy loading) in web members are presented in Fig. 5.E11 and are tabulated (with the selected web members) in the table below:

Web Member Reference	Factored Axial Force (kN) (−ve = tension)	Factored Axial Resistance (kN) (C_r or T_r)	Section Selected using Tables 5.8 and 5.11 (Theoretical length 990 mm)
A	− 191	$T_r = 207$	2L35×35×6
B	+ 191	$C_r = 214$	2L55×55×5
C	− 137	$T_r = 175$	2L35×35×5
D	+ 137	$C_r = 169$	2L45×45×5
E	− 82.2	$T_r = 121$	2L25×25×5
F	+ 82.2	$C_r = 102$	2L35×35×5
G	− 27.4*	$T_r = 60.8$	
		$C_r = 9$	L25×25×5
H	+ 27.4	$C_r = 44$	2L25×25×5

*Note: Possibility of force reversal during erection (6 kN)

(b) Truss Framing Layout and Truss Modelling

A scaled layout of truss members is provided as shown in Fig. 5.E12, to ensure simplicity in connection detailing and to evaluate amount of eccentricity at each web to chord joint.

(c) Detailed Member Design Checks (all chord and web members)

Bottom Chord: Use method as shown for HSS truss T1.

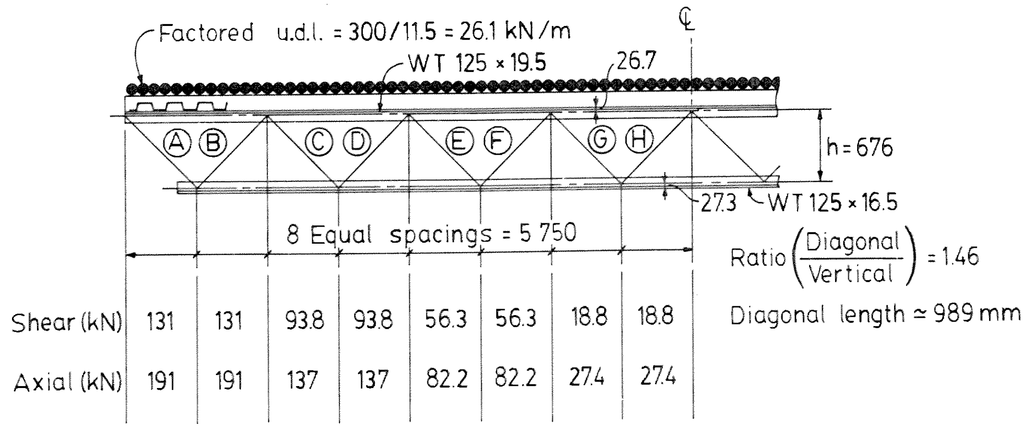


Figure 5.E11
Computation of Factored Web Forces for Preliminary Design (WT Chords)

Top Chord: Five load cases as shown for HSS truss T1 shall be investigated. Note that WT chord selected is a "Class 4" section. Values of sectional resistance are computed based on properties of a reduced 'tee' section whose d/w ratio satisfies "Class 3" limit.

Web members (in compression): Double angle struts interconnected at mid length are assumed. Torsional-flexural instability of each web member in compression is to be investigated using an equivalent radius of gyration method, see Reference (5.18).

Web members (in tension): Use method as shown for HSS truss T1.

(d) Compute Total Number of Shear Studs

With the selected top chord member of WT125×19.5, **stud diameter** of 19 mm is found to be satisfactory.

2.5 times flange thickness of WT = 2.5(11.2) = 28 > 19 OK

q_r for 19 mm studs in 20 MPa, 2 300 kg/m^3 concrete may be obtained from Table 2.1 as 74.3 kN.
V_h is shown as 562 kN; see item (a) of composite truss with WT chords.

Total number of 19 mm diameter studs (based on single stud per flute)
$= 2\ V_h/q_r$
$= (2)(562)/74.3$
= **16 studs** per truss.

(e) Truss Deflection Estimate

Top Chord	WT 125×19.5	$I_x = 3.26 \times 10^6$ mm^4
Bottom Chord	WT 125×16.5	$I_x = 2.85 \times 10^6$ mm^4

– Camber requirement (use Table 5.15)
$I_s = 958\ (h^2) = 958(676)^2 = 438 \times 10^6$ mm^4

Total effective truss moment of inertia
$I = (438 + 3.26 + 2.85)\ 10^6 = 444 \times 10^6$ mm^4

$$\Delta_c = \frac{5}{384}\ \frac{8.02\ (11.5)^4}{200\ (444)}\ 10^3 = 21 \text{ mm}$$

20 mm camber required

– Deflection due to all specified superimposed loads including long term effects.

$I_g = 1\ 224 \times 10^6$ mm^4 $\quad L = 11.5$ m
$I_{wr} = (0.15/0.85)\ 438 \times 10^6$
$= 77 \times 10^6$ mm^4
$I_t = (1\ 224 - 77) \times 10^6$
$= 1\ 147 \times 10^6$ mm^4

$$\Delta = \frac{5}{384}\ \frac{(68.7+41.4+24.2)L^3}{200\ (1\ 147)}\ 10^3 \times 1.3$$

$$= 15.1 \text{ mm} < \frac{L}{300} = 38 \quad \text{OK}$$

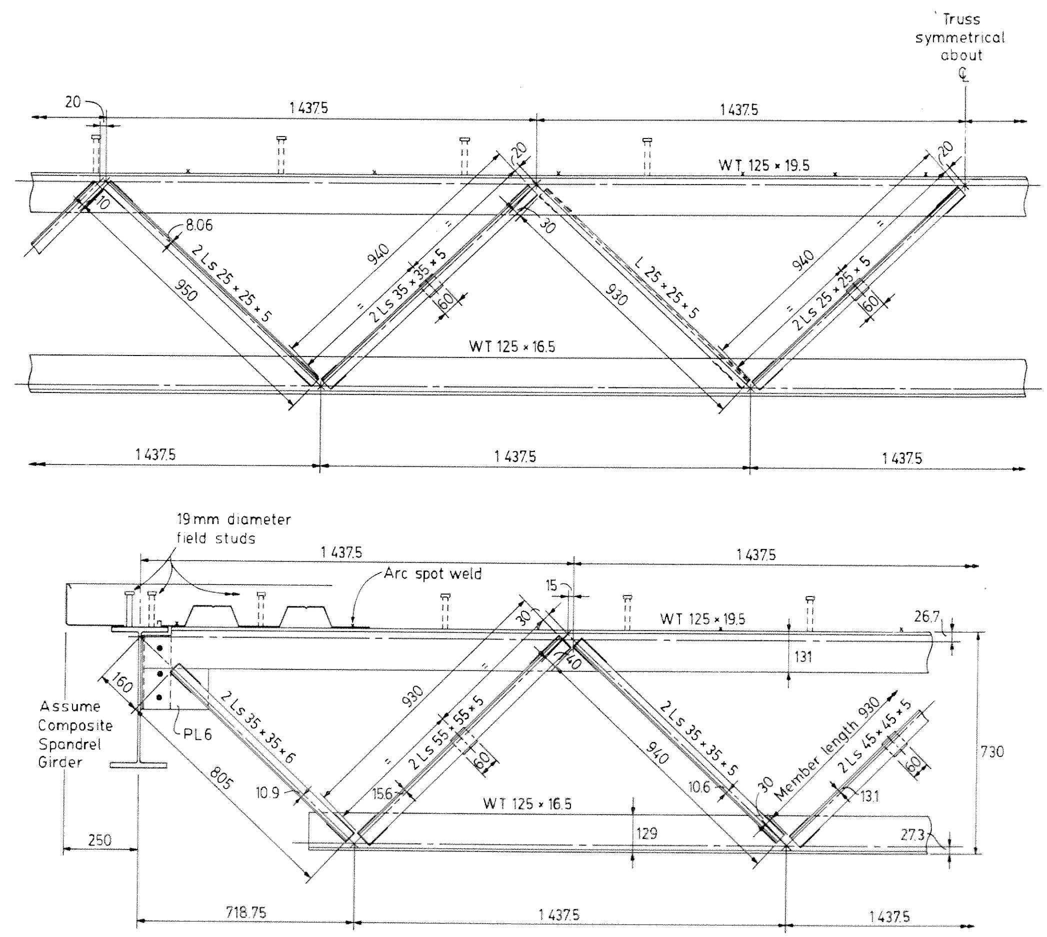


Figure 5.E12
Truss Framing Layout (Truss T1) (WT Chords)

Summary of Truss Mass Takeoff

Truss Chord Type	Mass per Truss (kg)	
HSS Chords	427	(or 37.1 kg/m)
WT Chords	487	(or 42.4 kg/m)

From reference (15): cost factor for trusses with HSS chords is 1.8, and cost factor for trusses with WT chords is 1.5.

Therefore factored truss mass ratio for costing purposes (HSS versus WT truss)
$= 1.8\ (427)/(1.5(487)) \simeq 1.0$. Say about the same cost.

REFERENCES

5.1 "Steel Joist Facts", Second Edition, Canadian Institute of Steel Construction, May 1980.

5.2 "Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders", Steel Joist Institute, 1981.

5.3 Lembeck, Jr., H.G., "Composite Design of Open Web Steel Joists", M.Sc. Thesis, Washington University, St. Louis, Mo., 1965.

5.4 Wang, P.C., and Kaley, D.J., "Composite Action of Concrete Slab and Open Web Joist (without the use of shear connectors)", AISC Engineering Journal, January 1967.

5.5 Tide, R.H.R., and Galambos, T.V., "Composite Open-Web Steel Joists", AISC Engineering Journal, January 1970.

5.6 Cran, J.A., "Design and Testing Composite Open Web Steel Joists", Technical Bulletin 11, Stelco, January 1972.

5.7 Atkinson, A.H., and Cran, J.A., "The Design and Economics of Composite Open-Web Steel Joists", Canadian Structural Engineering Conference, 1972.

5.8 Azmi, M.H., "Composite Open-Web Trusses with Metal Cellular Floor", a Master of Engineering Thesis, McMaster University, April 1972.

5.9 Fahmy, E.H.A., "Inelastic Analysis of Composite Open-Web Steel Joists", a Master of Engineering Thesis, McMaster University, 1974.

5.10 Robinson, H., "Composite Open-Web Joists with Formed Steel Deck", CSEC, 1978.

5.11 Robinson, H., Fahmy, E.H.A., "The Design of Partially Connected Composite Open-Web Joists", CJCE, 1978.

5.12 Iyengar, S.H., and Zils, J.J., "Composite Floor System for Sears Tower", AISC Engineering Journal, Third Quarter, 1973.

5.13 Bjorhovde, R., "Full Scale Test of a Composite Truss", Structural Engineering Report No. 97, Department of Civil Engineering, University of Alberta.

5.14 Ritchie, J.K., "Steel Building Structures in Canada – an Overview", Proceedings – SAISC Second Engineering Symposium, 1980.

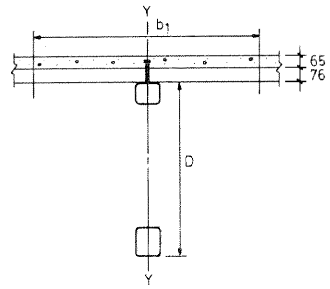
5.15 "A Project Analysis Approach to Building Costs", Canadian Institute of Steel Construction and Canadian Steel Construction Council, 1983.

5.16 Blodgett, O.W., "Design of Welded Structures", The J.F. Lincoln Arc Welding Foundation, 1976.

5.17 Madugula, M.K.S., and Ray, S.K., "Ultimate Strength of Eccentrically Loaded Cold-Formed Angles", CSCE Annual Conference Proceedings, 1983.

5.18 Gaylord, E.H., and Gaylord, C.N., "Design of Steel Structures", McGraw-Hill, 1957.

Table 5.3
COMPOSITE TRUSS BOTTOM CHORD
Trial Selection Table



65 mm Concrete Cover on 76 mm Deck
20 MPa Normal Density+ Concrete

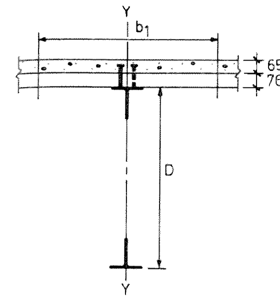
HSS BOTTOM CHORDS (G40.21-M 350W)

Bottom Chord Section		Composite Truss								
Designation and Design Data	Mass kg/m		Factored Moment Resistances, M_{rc} (kN·m)				Gross Moment of Inertia, I_g (10^6 mm^4)			
		D(mm) / b_1(mm)	650	750	850	950	650	750	850	950
HSS152.4×101.6×11.13	38.0	2360	1040	1190	1340	1490	1740	2290	2910	3610
V_h = 1520 kN		2330	1040	1190	1340	1490	1740	2280	2900	3590
r_y = 38.4 mm		2290	1040	1190	1340	1490	1730	2270	2890	3580
HSS152.4×101.6×9.53	33.3	2360	921	1050	1190	1320	1570	2060	2620	3250
V_h = 1340 kN		2190	918	1050	1190	1320	1550	2030	2580	3200
r_y = 39.1 mm		2020	914	1050	1180	1320	1520	1990	2540	3140
HSS152.4×101.6×7.95	28.4	2360	788	902	1020	1130	1380	1820	2310	2860
V_h = 1140 kN		2040	784	898	1010	1130	1350	1770	2250	2780
r_y = 39.9 mm		1720	778	892	1010	1120	1300	1700	2160	2680
HSS152.4×101.6×6.35	23.2	2360	648	741	835	928	1170	1540	1960	2420
V_h = 932 kN		1880	644	737	830	923	1130	1480	1880	2330
r_y = 40.6 mm		1410	636	729	822	916	1060	1390	1770	2190
HSS101.6×101.6×9.53	25.7	2360	740	843	946	1050	1370	1790	2250	2780
V_h = 1030 kN		1960	736	839	942	1040	1330	1730	2180	2680
r_y = 36.8 mm		1550	729	832	935	1040	1260	1640	2070	2550
HSS101.6×101.6×7.95	22.1	2360	641	730	819	907	1210	1570	1990	2450
V_h = 888 kN		1850	636	725	814	903	1160	1510	1910	2350
r_y = 37.6 mm		1340	628	717	806	895	1090	1410	1780	2200
HSS127.0×76.2×7.95	22.1	2330	629	718	807	896	1160	1520	1930	2380
V_h = 888 kN		1840	625	714	803	891	1120	1460	1850	2290
r_y = 29.4 mm		1340	617	706	795	884	1050	1370	1730	2140
HSS127.0×76.2×6.35	18.2	2330	521	594	667	740	985	1290	1630	2010
V_h = 731 kN		1720	517	590	663	736	941	1230	1560	1930
r_y = 30.1 mm		1100	508	581	654	727	861	1130	1430	1760
HSS127.0×76.2×4.78	14.1	2330	404	460	516	573	784	1020	1300	1600
V_h = 564 kN		1670	401	457	514	570	752	983	1240	1540
r_y = 30.8 mm		1000	395	451	508	564	689	901	1140	1410
HSS101.6×76.2×9.53	21.9	2330	634	722	810	898	1200	1560	1960	2420
V_h = 879 kN		1830	630	718	806	894	1150	1490	1890	2320
r_y = 27.7 mm		1330	622	710	798	886	1070	1400	1770	2180
HSS101.6×76.2×7.95	18.9	2330	550	626	701	777	1060	1370	1730	2130
V_h = 759 kN		1740	546	621	697	773	1010	1310	1660	2040
r_y = 28.5 mm		1140	537	613	689	765	926	1210	1520	1880
HSS101.6×76.2×6.35	15.6	2330	456	519	581	644	892	1160	1460	1800
V_h = 627 kN		1670	453	515	578	641	854	1110	1400	1730
r_y = 29.3 mm		1000	445	508	570	633	776	1010	1270	1570

No. of studs per truss = $2(V_h/q_r)$

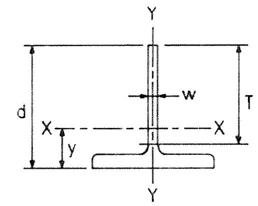
+ Concrete density = 2300 kg/m^3

Table 5.4
COMPOSITE TRUSS BOTTOM CHORD
Trial Selection Table



65 mm Concrete Cover on 76 mm Deck
20 MPa Normal Density+ Concrete

WT BOTTOM CHORDS (G40.21-M 300W)

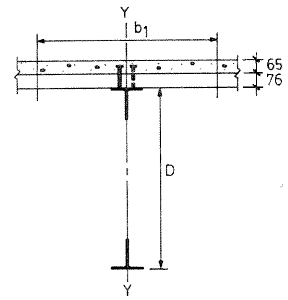


Bottom Chord Section and Design Data			Composite Truss								
				Factored Moment Resistances M_{rc} (kN·m)				Gross Moment of Inertia, I_g (10^6 mm^4)			
			D(mm) / b_1(mm)	600	750	900	1050	600	750	900	1050
WT180×39.5	r_y=48.9 mm	d=177 mm	2460	923	1130	1330	1540	1770	2640	3690	4920
V_h=1360 kN	y =35.0 mm	T=141 mm	2250	920	1120	1330	1530	1730	2590	3620	4810
V_r= 296 kN	w= 9.4 mm		2050	916	1120	1320	1530	1690	2530	3530	4700
WT180×36	r_y=48.5 mm	d=175 mm	2460	839	1020	1210	1390	1640	2440	3410	4540
V_h=1230 kN	y =34.2 mm	T=141 mm	2160	835	1020	1200	1390	1590	2370	3310	4410
V_r= 268 kN	w= 8.6 mm		1860	829	1010	1200	1380	1530	2290	3190	4250
WT180×32	r_y=48.1 mm	d=174 mm	2460	754	919	1080	1250	1500	2240	3130	4160
V_h=1100 kN	y =33.2 mm	T=141 mm	2060	750	915	1080	1240	1450	2160	3010	4010
V_r= 239 kN	w= 7.7 mm		1660	743	908	1070	1240	1370	2050	2860	3810
WT180×28.5	r_y=39.2 mm	d=179 mm	2430	665	811	958	1100	1340	2000	2800	3730
V_h=975 kN	y =39.2 mm	T=149 mm	1950	660	807	953	1100	1280	1910	2680	3570
V_r=252 kN	w= 7.9 mm		1470	653	799	945	1090	1200	1790	2500	3340
WT180×25.5	r_y=38.8 mm	d=178 mm	2430	595	725	856	986	1220	1820	2550	3390
V_h=869 kN	y =38.8 mm	T=149 mm	1870	590	721	851	981	1160	1740	2430	3230
V_r=228 kN	w= 7.2 mm		1310	582	712	843	973	1070	1600	2240	2980
WT180×22.5	r_y=37.8 mm	d=176 mm	2430	531	647	764	880	1100	1650	2300	3070
V_h=775 kN	y =40.2 mm	T=149 mm	1800	527	643	759	876	1050	1570	2190	2920
V_r=216 kN	w= 6.9 mm		1170	518	634	750	867	948	1420	1990	2650
WT155×43	r_y=63.7 mm	d=155 mm	2460	1020	1240	1470	1690	1940	2880	4010	5330
V_h=1490 kN	y =26.1 mm	T=122 mm	2350	1020	1240	1470	1690	1920	2850	3970	5270
V_r= 251 kN	w= 9.1 mm		2250	1020	1240	1460	1690	1900	2820	3920	5210
WT155×39.5	r_y=63.1 mm	d=153 mm	2460	935	1140	1340	1550	1810	2690	3750	4980
V_h=1360 kN	y =26.1 mm	T=121 mm	2250	932	1140	1340	1540	1770	2640	3670	4880
V_r= 240 kN	w= 8.8 mm		2050	928	1130	1340	1540	1730	2580	3590	4760
WT155×37	r_y=49.7 mm	d=155 mm	2460	878	1070	1260	1450	1710	2550	3560	4730
V_h=1280 kN	y =29.7 mm	T=122 mm	2190	874	1070	1260	1450	1670	2480	3460	4600
V_r= 260 kN	w= 9.4 mm		1930	869	1060	1250	1440	1610	2410	3350	4460
WT155×33.5	r_y=49.2 mm	d=153 mm	2460	793	965	1140	1310	1580	2350	3270	4340
V_h=1150 kN	y =28.7 mm	T=121 mm	2100	788	961	1130	1310	1520	2270	3160	4200
V_r= 232 kN	w= 8.5 mm		1730	782	954	1130	1300	1450	2160	3010	4000
WT155×30	r_y=49.0 mm	d=152 mm	2460	714	868	1020	1180	1450	2150	2990	3980
V_h=1030 kN	y =27.5 mm	T=122 mm	2000	709	863	1020	1170	1390	2060	2870	3820
V_r= 203 kN	w= 7.5 mm		1550	701	856	1010	1160	1300	1940	2700	3590

No. of studs per truss = $2(V_h/q_r)$

+ Concrete density = 2 300 kg/m^3

Table 5.4 (continued)
COMPOSITE TRUSS BOTTOM CHORD
Trial Selection Table



65 mm Concrete Cover on 76 mm Deck
20 MPa Normal Density+ Concrete

WT BOTTOM CHORDS (G40.21-M 300W)

Bottom Chord Section and Design Data			Composite Truss	Factored Moment Resistances M_{rc} (kN·m)				Gross Moment of Inertia, I_g (10^6 mm^4)			
			D(mm) / b_1(mm)	600	750	900	1050	600	750	900	1050
WT155×26	r_y=39.2 mm	d=158 mm	2430	620	755	890	1020	1270	1900	2650	3530
V_h=899 kN	y =32.9 mm	T=128 mm	1890	616	750	885	1020	1220	1810	2530	3360
V_r=214 kN	w= 7.6 mm		1360	607	742	877	1010	1120	1680	2340	3120
WT155×22.5	r_y=38.8 mm	d=156 mm	2430	532	647	762	877	1120	1660	2320	3090
V_h=767 kN	y =32.1 mm	T=128 mm	1790	528	643	758	873	1060	1580	2210	2930
V_r=183 kN	w= 6.6 mm		1160	519	634	749	864	962	1430	2000	2660
WT155×19.5	r_y=38.3 mm	d=155 mm	2410	464	564	664	764	992	1480	2060	2740
V_h=677 kN	y =31.4 mm	T=128 mm	1710	461	561	661	761	940	1400	1950	2600
V_r=160 kN	w= 5.8 mm		1010	452	552	652	752	838	1250	1740	2320
WT125×29	r_y=50.4 mm	d=126 mm	2430	699	849	999	1150	1440	2130	2960	3930
V_h=1000 kN	y =22.2 mm	T= 95 mm	1970	694	844	994	1140	1380	2040	2840	3770
V_r= 180 kN	w= 8.0 mm		1510	686	836	986	1140	1290	1920	2660	3540
WT125×24.5	r_y=49.2 mm	d=124 mm	2430	591	717	844	970	1250	1850	2560	3400
V_h=842 kN	y =22.1 mm	T= 96 mm	1850	587	713	839	965	1190	1760	2440	3240
V_r=164 kN	w= 7.4 mm		1270	578	704	831	957	1090	1610	2240	2970
WT125×22.5	r_y=35.1 mm	d=133 mm	2410	538	654	770	886	1140	1690	2350	3130
V_h=772 kN	y =27.8 mm	T=110 mm	1790	534	650	766	882	1080	1610	2240	2970
V_r=180 kN	w= 7.6 mm		1160	525	641	757	873	979	1460	2030	2700
WT125×19.5	r_y=34.7 mm	d=131 mm	2410	465	565	665	764	1000	1490	2070	2750
V_h=664 kN	y =26.7 mm	T=110 mm	1700	462	561	661	760	949	1410	1960	2610
V_r=154 kN	w= 6.6 mm		1000	453	552	652	751	845	1260	1750	2320
WT125×16.5	r_y=33.7 mm	d=129 mm	2410	395	479	563	648	864	1280	1790	2370
V_h=562 kN	y =27.3 mm	T=110 mm	1700	392	476	561	645	824	1220	1700	2260
V_r=140 kN	w= 6.1 mm		1000	386	470	554	639	744	1110	1540	2050
WT100×18	r_y=40.8 mm	d=100 mm	2410	439	532	625	717	967	1430	1980	2630
V_h=618 kN	y =17.5 mm	T= 77 mm	1700	436	529	622	714	919	1360	1890	2500
V_r=110 kN	w= 6.2 mm		1000	428	521	614	707	823	1220	1690	2240
WT100×15.5	r_y=32.0 mm	d=105 mm	2390	383	464	545	626	849	1260	1740	2310
V_h=540 kN	y =21.1 mm	T= 85 mm	1690	380	461	542	623	811	1200	1670	2210
V_r=120 kN	w= 6.4 mm		1000	374	455	536	617	735	1090	1510	2010
WT100×13.5	r_y=31.2 mm	d=104 mm	2390	324	392	461	529	730	1080	1500	1990
V_h=456 kN	y =21.1 mm	T= 86 mm	1690	322	391	459	527	701	1040	1440	1910
V_r=107 kN	w= 5.8 mm		1000	318	386	455	523	644	954	1330	1760

No. of studs per truss = $2(V_h/q_r)$

+ Concrete density = 2 300 kg/m^3

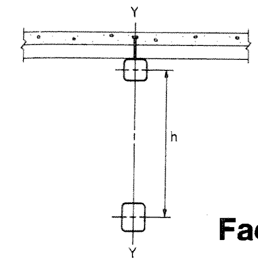
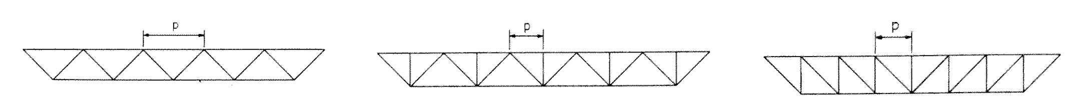


Table 5.5
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table

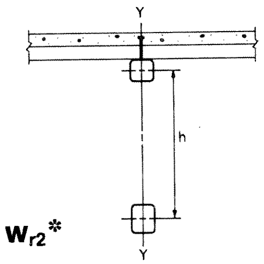
Class C HSS Top Chords (G40.21-M 350W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

Top chord section and design data	Truss Span L(mm)	h(mm) / L′ / p/h	W_{r1}+ (kN/m) 550 L/2	550 L/3	700 L/2	700 L/3	850 L/2	850 L/3	W_{r2} (kN/m) 550	700	850
HSS101.6×101.6×9.53	15 000	1.0		5.4		6.8		8.2	19.7	24.4	28.5
Mass = 25.7 kg/m		2.5		5.3		6.5		7.6	16.6	18.1	18.2
r_y = 36.8 mm	17 000	1.0		3.4		4.3		5.3	15.4	19.1	22.4
2.5t = 23.8 mm		2.5		3.4		4.2		5.0	13.2	14.6	14.9
	19 000	1.0		2.2		2.8		3.4	12.4	15.4	18.1
		2.5		2.2		2.8		3.3	10.7	12.0	12.5
HSS101.6×101.6×7.95	14 000	1.0	3.0	6.1	3.8	7.7	4.6	9.3	19.5	24.0	28.0
Mass = 22.1 kg/m		2.5	3.0	5.9	3.7	7.3	4.4	8.3	16.3	17.7	17.6
r_y = 37.6 mm	16 000	1.0		3.8		4.8		5.8	15.0	18.5	21.7
2.5t = 19.9 mm		2.5		3.7		4.7		5.5	12.7	14.1	14.3
	18 000	1.0		2.5		3.1		3.8	11.9	14.7	17.3
		2.5		2.4		3.0		3.6	10.2	11.5	11.9
HSS101.6×101.6×6.35	13 000	1.0	3.5	6.6	4.4	8.4	5.3	10.1	18.6	22.8	26.5
Mass = 18.2 kg/m		2.5	3.4	6.4	4.2	7.8	4.9	8.8	15.4	16.6	16.4
r_y = 38.4 mm	15 000	1.0	2.0	4.1	2.5	5.1	3.0	6.2	14.0	17.3	20.3
2.5t = 15.9 mm		2.5	2.0	4.0	2.5	4.9	2.9	5.7	11.9	13.1	13.2
	17 000	1.0		2.6		3.3		4.0	10.9	13.6	15.9
		2.5		2.6		3.2		3.8	9.4	10.6	10.9
HSS101.6×76.2×6.35	10 000	1.0	4.9	9.5	6.2	11.9	7.5	14.2	26.4	32.0	36.4
Mass = 15.6 kg/m		2.5	4.8	8.9	5.8	10.5	6.7	11.4	20.4	20.6	19.1
r_y = 29.3 mm	12 000	1.0		5.1		6.4		7.7	18.6	22.7	26.2
2.5t = 15.9 mm		2.5		4.9		6.0		6.8	15.0	15.7	15.1
	14 000	1.0		2.9		3.6		4.4	13.7	16.9	19.6
		2.5		2.8		3.5		4.1	11.4	12.3	12.2
HSS76.2×76.2×7.95	10 000	1.0	4.3	8.6	5.4	10.7	6.5	12.7	25.9	30.8	34.1
Mass = 15.8 kg/m		2.5	4.1	7.9	5.0	9.1	5.7	9.6	17.9	16.8	14.6
r_y = 27.2 mm	12 000	1.0		4.5		5.7		6.9	18.2	22.0	24.8
2.5t = 19.9 mm		2.5		4.4		5.3		5.9	13.3	13.0	11.6
	14 000	1.0		2.5		3.2		3.8	13.5	16.4	18.7
		2.5		2.5		3.0		3.5	10.2	10.3	9.4
HSS76.2×76.2×6.35	9 000	1.0	5.7	10.6	7.1	13.2	8.5	15.5	26.4	31.2	34.2
Mass = 13.1 kg/m		2.5	5.4	9.5	6.3	10.5	6.9	10.6	17.8	16.4	14.2
r_y = 28.0 mm	11 000	1.0	2.6	5.4	3.3	6.7	4.0	8.1	18.0	21.6	24.2
2.5t = 15.9 mm		2.5	2.5	5.1	3.1	6.0	3.6	6.5	13.0	12.5	11.2
	13 000	1.0		2.9		3.7		4.5	13.0	15.8	18.0
		2.5		2.9		3.5		4.0	9.8	9.8	9.0



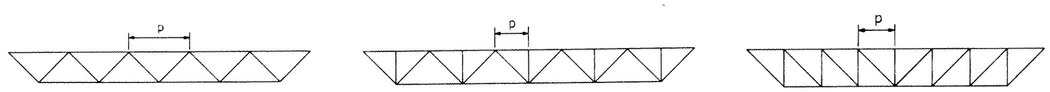
* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.
\+ Steel truss should have an axis of symmetry in its plane.

Table 5.6
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table



Class H HSS Top Chords (G40.21-M 350W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

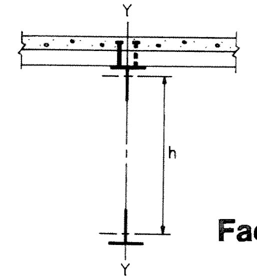
Top chord section and design data	Truss Span L(mm)	h(mm) / p/h (L′)	$W_{r1}$$^{+}$(kN/m) 550 L/2	550 L/3	700 L/2	700 L/3	850 L/2	850 L/3	W_{r2}(kN/m) 550	700	850
HSS101.6×101.6×9.53 Mass = 25.7 kg/m r_y = 36.8 mm 2.5t = 23.8 mm	15 000	1.0		6.0		7.6		9.1	19.9	24.9	29.4
		2.5		5.8		7.2		8.3	17.7	19.7	19.8
	17 000	1.0		3.6		4.6		5.6	15.5	19.5	23.2
		2.5		3.6		4.5		5.3	14.1	15.9	16.4
	19 000	1.0		2.4		3.0		3.6	12.5	15.7	18.7
		2.5		2.3		2.9		3.5	11.5	13.2	13.7
HSS101.6×101.6×7.95 Mass = 22.1 kg/m r_y = 37.6 mm 2.5t = 19.9 mm	14 000	1.0	3.2	6.8	4.1	8.7	4.9	10.4	19.6	24.5	28.8
		2.5	3.2	6.6	4.0	8.1	4.7	9.2	17.4	19.1	19.1
	16 000	1.0		4.1		5.3		6.4	15.1	18.9	22.4
		2.5		4.1		5.1		5.9	13.6	15.3	15.6
	18 000	1.0		2.6		3.3		4.0	11.9	15.0	17.9
		2.5		2.6		3.2		3.8	10.9	12.5	13.0
HSS101.6×101.6×6.35 Mass = 18.2 kg/m r_y = 38.4 mm 2.5t = 15.9 mm	13 000	1.0	3.7	7.7	4.7	9.7	5.6	11.6	18.7	23.3	27.3
		2.5	3.6	7.4	4.5	8.9	5.2	9.9	16.4	17.9	17.7
	15 000	1.0	2.1	4.5	2.7	5.7	3.2	6.9	14.1	17.6	20.9
		2.5	2.1	4.4	2.6	5.4	3.1	6.3	12.7	14.2	14.4
	17 000	1.0		2.8		3.6		4.3	11.0	13.8	16.4
		2.5		2.8		3.4		4.1	10.1	11.5	11.9
HSS101.6×76.2×6.35 Mass = 15.6 kg/m r_y = 29.3 mm 2.5t = 15.9 mm	10 000	1.0	5.2	10.9	6.6	13.7	7.9	16.3	26.7	32.7	37.5
		2.5	5.1	10.1	6.2	11.8	7.0	12.6	21.6	22.0	20.4
	12 000	1.0		5.6		7.1		8.5	18.7	23.2	27.0
		2.5		5.4		6.5		7.4	15.9	16.9	16.3
	14 000	1.0		3.1		3.9		4.7	13.8	17.3	20.3
		2.5		3.0		3.7		4.3	12.2	13.3	13.2
HSS76.2×76.2×7.95 Mass = 15.8 kg/m r_y = 27.2 mm 2.5t = 19.9 mm	10 000	1.0	4.6	9.7	5.8	12.1	6.9	14.3	26.6	32.0	35.8
		2.5	4.4	8.8	5.3	10.1	6.0	10.5	19.3	18.2	15.8
	12 000	1.0		4.9		6.2		7.4	18.7	22.9	26.1
		2.5		4.7		5.6		6.2	14.5	14.2	12.7
	14 000	1.0		2.7		3.4		4.1	13.9	17.1	19.8
		2.5		2.6		3.2		3.7	11.2	11.3	10.4
HSS76.2×76.2×6.35 Mass = 13.1 kg/m r_y = 28.0 mm 2.5t = 15.9 mm	9 000	1.0	6.1	12.4	7.6	15.4	9.1	18.0	27.0	32.3	35.8
		2.5	5.7	10.9	6.7	11.8	7.3	11.7	19.1	17.7	15.1
	11 000	1.0	2.8	6.0	3.5	7.5	4.2	8.9	18.4	22.4	25.4
		2.5	2.7	5.6	3.3	6.6	3.8	7.1	14.1	13.6	12.1
	13 000	1.0		3.1		4.0		4.8	13.3	16.4	18.9
		2.5		3.0		3.7		4.2	10.7	10.7	9.8



* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.

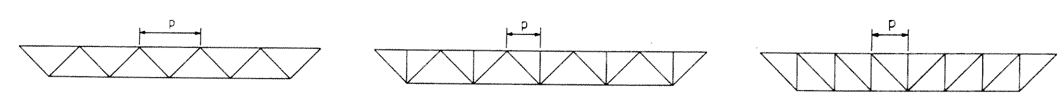
+ Steel truss should have an axis of symmetry in its plane.

Table 5.7
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table



WT Top Chords (G40.21-M 300W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

Top chord section and design data	Truss Span L(mm)	h(mm) / p/h (L′)	$W_{r1}$$^{+}$(kN/m) 650 L/2	650 L/3	800 L/2	800 L/3	950 L/2	950 L/3	W_{r2}(kN/m) 650	800	950
WT180×39.5 V_r = 296 kN, w = 9.4 mm b = 205 mm, y = 35.0 mm t = 16.8 mm, T = 141 mm d = 177 mm	20 000	1.0		5.2		6.3		7.5	17.2	20.5	23.3
		2.5		5.0		5.8		6.5	13.9	14.5	14.2
	21 500	1.0		4.0		4.9		5.8	15.0	17.9	20.4
		2.5		3.9		4.6		5.2	12.3	13.0	12.9
	23 000	1.0		3.2		3.9		4.6	13.1	15.7	18.0
		2.5		3.1		3.7		4.2	10.9	11.7	11.7
WT180×36‡ V_r = 268 kN, w = 8.6 mm b = 204 mm, y = 34.2 mm t = 15.1 mm, T = 141 mm d = 175 mm	18 000	1.0	3.4	6.5	4.2	7.9	4.9	9.3	18.7	22.1	24.9
		2.5	3.3	6.1	3.9	7.0	4.4	7.6	14.5	14.7	14.0
	19 500	1.0		5.0		6.1		7.1	16.0	19.0	21.5
		2.5		4.7		5.5		6.1	12.7	13.1	12.6
	21 000	1.0		3.9		4.7		5.5	13.9	16.5	18.8
		2.5		3.7		4.4		4.9	11.2	11.7	11.5
WT180×28.5‡ V_r = 252 kN, w = 7.9 mm b = 172 mm, y = 39.2 mm t = 13.1 mm, T = 149 mm d = 179 mm	15 000	1.0	3.5	6.8	4.3	8.2	5.0	9.6	20.2	23.5	26.0
		2.5	3.3	6.2	3.9	7.0	4.4	7.4	14.5	14.2	12.9
	16 500	1.0		4.9		6.0		7.0	16.8	19.7	22.0
		2.5		4.6		5.3		5.8	12.6	12.5	11.6
	18 000	1.0		3.6		4.4		5.2	14.2	16.8	18.9
		2.5		3.5		4.1		4.5	10.9	11.1	10.5
WT155×33.5 V_r = 232 kN, w = 8.5 mm b = 204 mm, y = 28.7 mm t = 14.6 mm, T = 121 mm d = 153 mm	17 000	1.0	4.1	7.6	5.0	9.2	5.9	10.7	19.5	22.8	25.3
		2.5	3.9	6.9	4.6	7.7	5.0	8.1	14.2	13.8	12.7
	18 500	1.0	3.0	5.7	3.6	7.0	4.3	8.2	16.6	19.5	21.8
		2.5	2.9	5.3	3.4	6.1	3.8	6.6	12.4	12.4	11.5
	20 000	1.0		4.4		5.4		6.3	14.3	16.9	19.0
		2.5		4.2		4.8		5.3	10.9	11.1	10.5
WT155×30‡ V_r = 203 kN, w = 7.5 mm b = 203 mm, y = 27.5 mm t = 13.1 mm, T = 122 mm d = 152 mm	15 000	1.0	5.7	10.1	7.0	12.1	8.1	13.9	21.7	25.0	27.2
		2.5	5.3	8.6	5.9	9.2	6.3	9.1	14.6	13.7	12.1
	16 500	1.0	4.0	7.4	4.9	8.9	5.8	10.3	18.2	21.1	23.2
		2.5	3.8	6.6	4.4	7.3	4.8	7.5	12.7	12.2	11.0
	18 000	1.0	2.9	5.5	3.5	6.7	4.2	7.8	15.4	18.0	19.9
		2.5	2.8	5.1	3.3	5.7	3.6	6.1	11.1	10.9	10.0
WT155×26‡ V_r = 214 kN, w = 7.6 mm b = 167 mm, y = 32.9 mm t = 13.2 mm, T = 128 mm d = 158 mm	14 000	1.0	4.3	8.1	5.3	9.8	6.2	11.3	21.8	25.2	27.5
		2.5	4.1	7.2	4.7	8.0	5.1	8.2	15.0	14.2	12.7
	15 500	1.0	2.9	5.8	3.6	7.0	4.2	8.2	18.0	20.9	23.1
		2.5	2.8	5.3	3.3	6.0	3.7	6.4	12.9	12.5	11.4
	17 000	1.0		4.2		5.2		6.0	15.1	17.7	19.7
		2.5		4.0		4.6		5.0	11.2	11.1	10.3

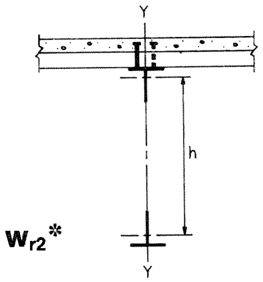


* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.

‡ Class 4 section due to stem slenderness. w_{r1} and w_{r2} values have been computed based on properties of a reduced tee section whose d/w ratio satisfies class 3 limit.

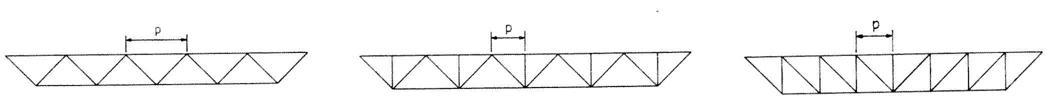
+ Steel truss should have an axis of symmetry in its plane.

Table 5.7 (continued) COMPOSITE TRUSS STEEL TOP CHORD Trial Selection Table



WT Top Chords (G40.21-M 300W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

Top chord section and design data	Truss Span L(mm)	h(mm) → / L′, p/h ↓	W_{r1}^{+} (kN/m) 650 L/2	650 L/3	800 L/2	800 L/3	950 L/2	950 L/3	W_{r2} (kN/m) 650	800	950
WT155×22.5‡ V_r = 183 kN, w = 6.6 mm b = 166 mm, y = 32.1 mm t = 11.2 mm, T = 128 mm d = 156 mm	12 500	1.0	9.4	9.5	6.5	11.2	7.5	12.7	21.4	24.1	25.5
		2.5	4.8	7.7	5.3	8.0	5.4	7.6	12.9	11.5	9.8
	13 500	1.0	4.0	7.4	4.8	8.8	5.6	10.1	18.6	21.1	22.6
		2.5	3.7	6.3	4.1	6.7	4.4	6.6	11.7	10.6	9.2
	14 500	1.0	3.0	5.8	3.7	7.0	4.3	8.1	16.3	18.7	20.1
		2.5	2.8	5.1	3.3	5.6	3.6	5.7	10.6	9.8	8.6
WT125×22.5 V_r = 180 kN, w = 7.6 mm b = 148 mm, y = 27.8 mm t = 13.0 mm, T = 110 mm d = 133 mm	12 500	1.0	4.8	8.9	5.8	10.7	6.7	12.2	23.2	26.2	27.9
		2.5	4.4	7.6	4.9	8.0	5.2	7.9	14.4	13.0	11.2
	13 500	1.0	3.6	6.9	4.3	8.3	5.1	9.6	20.1	23.0	24.7
		2.5	3.3	6.1	3.8	6.6	4.2	6.7	13.0	12.0	10.4
	14 500	1.0		5.4		6.6		7.6	17.6	20.3	22.0
		2.5		4.9		5.5		5.7	11.8	11.0	9.7
WT125×19.5‡ V_r = 154 kN, w = 6.6 mm b = 147 mm, y = 26.7 mm t = 11.2 mm, T = 110 mm d = 131 mm	11 500	1.0	5.5	9.8	6.6	11.7	7.7	13.2	23.0	25.7	26.9
		2.5	4.9	7.9	5.3	8.1	5.5	7.7	13.5	11.9	10.0
	12 500	1.0	4.0	7.5	4.9	9.0	5.6	10.2	19.8	22.3	23.7
		2.5	3.7	6.3	4.1	6.7	4.4	6.6	12.2	10.9	9.4
	13 500	1.0	3.0	5.8	3.6	7.0	4.2	8.1	17.2	19.6	21.0
		2.5	2.8	5.1	3.2	5.5	3.5	5.6	11.0	10.0	8.7
WT125×16.5‡ V_r = 140 kN, w = 6.1 mm b = 146 mm, y = 27.3 mm t = 9.1 mm, T = 110 mm d = 129 mm	10 000	1.0	7.1	12.0	8.4	13.9	9.5	15.3	24.1	26.2	26.6
		2.5	5.8	8.7	6.0	8.4	5.8	7.5	12.7	10.7	8.8
	11 000	1.0	5.1	8.9	6.1	10.5	7.0	11.8	20.4	22.5	23.3
		2.5	4.4	7.0	4.7	7.0	4.8	6.5	11.4	9.8	8.2
	12 000	1.0	3.6	6.7	4.4	8.0	5.1	9.1	17.5	19.5	20.5
		2.5	3.3	5.6	3.7	5.8	3.8	5.6	10.2	9.0	7.6
WT100×15.5 V_r = 120 kN, w = 6.4 mm b = 134 mm, y = 21.1 mm t = 10.2 mm, T = 85 mm d = 105 mm	9 000	1.0	9.2	15.3	10.7	17.3	11.8	18.2	27.2	28.2	27.3
		2.5	6.8	9.5	6.6	8.4	5.9	7.1	12.0	9.6	7.6
	10 000	1.0	6.5	11.1	7.7	12.9	8.6	14.0	22.8	24.2	24.0
		2.5	5.2	7.7	5.3	7.2	5.0	6.3	10.8	8.9	7.1
	11 000	1.0	4.6	8.3	5.5	9.7	6.2	10.8	19.4	21.0	21.2
		2.5	3.9	6.2	4.1	6.1	4.1	5.5	9.8	8.2	6.7
WT100×13.5 V_r = 107 kN, w = 5.8 mm b = 133 mm, y = 21.1 mm t = 8.4 mm, T = 86 mm d = 104 mm	8 000	1.0	10.9	17.5	12.5	19.2	13.5	19.7	28.0	28.5	27.0
		2.5	7.4	9.8	6.8	8.4	6.0	6.9	11.6	9.1	7.1
	9 000	1.0	7.5	12.5	8.7	14.2	9.7	15.1	23.2	24.2	23.6
		2.5	5.6	8.0	5.5	7.2	5.1	6.1	10.5	8.4	6.7
	10 000	1.0	5.2	9.1	6.2	10.5	7.1	11.5	19.5	20.8	20.7
		2.5	4.3	6.5	4.4	6.1	4.2	5.4	9.4	7.7	6.2



* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.

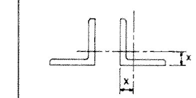
‡ Class 4 section due to stem slenderness. w_{r1} and w_{r2} values have been computed based on properties of a reduced tee section whose d/w ratio satisfies class 3 limit.

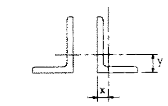
⁺ Steel truss should have an axis of symmetry in its plane.

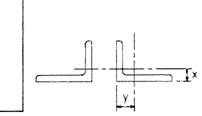
Factored Axial Tensile Resistances, T_r in kN

Table 5.8 Double Angle Tension Members

	Double Angles*	Mass kg/m	x mm	y mm	T_r kN
Equal Leg Angles	2L75×75×10	22.0	22.4		756
	2L65×65×10	18.8	19.9		648
	2L55×55×10	15.7	17.4		540
	2L55×55×8	12.8	16.7		440
	2L45×45×8	10.3	14.2		354
	2L35×35×6	6.03	10.9		207
	2L35×35×5	5.10	10.6		175
	2L25×25×5	3.53	8.06		121
Unequal Leg Angles	2L90×65×10	22.8	17.3	29.8	783
	2L80×60×10	20.4	16.5	26.5	702
	2L80×60×8	16.6	15.8	25.8	570
	2L75×50×8	14.7	13.0	25.5	505
	2L65×50×8	13.4	13.8	21.3	462
	2L65×50×5	8.63	12.7	20.2	258
	2L55×35×6	7.93	9.04	19.0	273
	2L45×30×6	6.50	8.22	15.7	224



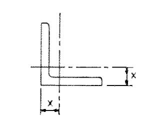


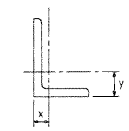


* G40.21-M 300W Steel

Table 5.9 Single Angle Tension Members

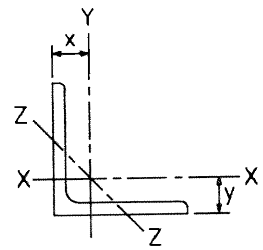
	Single Angles*	Mass kg/m	x mm	y mm	T_r kN
Equal Leg Angles	L100×100×13	19.1	29.8	29.8	656
	L90×90×13	17.0	27.2	27.2	586
	L75×75×13	14.0	23.5	23.5	481
	L65×65×10	9.42	19.9	19.9	324
	L55×55×10	7.85	17.4	17.4	270
	L55×55×8	6.41	16.7	16.7	220
	L45×45×8	5.15	14.2	14.2	177
	L45×45×6	3.96	13.4	13.4	136
	L35×35×6	3.01	10.9	10.9	104
	L35×35×5	2.55	10.6	10.6	87.8
	L25×25×5	1.77	8.06	8.06	60.8
Unequal Leg Angles	L125×90×13	20.6	23.7	41.2	710
	L100×90×13	18.1	26.1	31.1	621
	L100×75×13	16.5	20.9	33.4	570
	L90×75×13	15.5	21.8	29.3	535
	L90×75×10	12.2	20.7	28.2	419
	L90×65×10	11.4	17.3	29.8	392
	L80×60×10	10.2	16.5	26.5	351
	L80×60×8	8.29	15.8	25.8	286





* G40.21-M 300W Steel

Table 5.10
SINGLE ANGLE WEB STRUTS*
(with one leg welded to chords)



Factored Compressive Resistances, $C_{re}{}^{\#}$, in kN

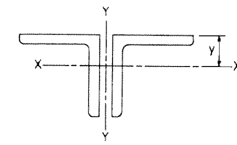
G40.21-M 300W

Single Angle Struts	Mass kg/m	x or y mm	Strut lengths between intersections of axes of strut and chords, L, (mm)									
			600	700	800	900	1000	1100	1200	1300	1400	1500
L100×100×16	23.1	30.8	393	388	381	375	368	360	352	344	335	325
L100×100×13	19.1	29.8	331	327	322	316	310	304	297	290	282	274
L100×100×10	14.9	28.7	264	260	256	252	247	242	237	231	225	218
L90×90×13	17.0	27.2	291	286	280	275	268	262	254	247	238	230
L90×90×10	13.3	26.2	233	229	224	220	215	209	203	197	191	183
L90×90×8	10.8	25.5	191	188	184	180	176	172	167	162	157	151
L75×75×13	14.0	23.5	229	224	218	212	205	197	189	180	170	159
L75×75×10	11.0	22.4	185	181	176	171	165	159	153	145	137	128
L75×75×8	8.92	21.7	153	149	146	141	137	132	126	120	113	106
L65×65×10	9.42	19.9	153	149	144	138	132	125	117	109	102	95
L65×65×8	7.66	19.2	127	123	119	114	109	103	97	90	84	78
L65×65×6	5.84	18.5	99	96	93	89	85	81	75	70	65	61
L55×55×10	7.85	17.4	121	116	111	105	97	89	82	76	71	66
L55×55×8	6.41	16.7	101	97	92	87	81	74	68	63	58	54
L55×55×6	4.90	16.0	79	76	72	68	63	58	54	49	46	42
L55×55×5	4.12	15.6	68	65	61	58	54	49	45	42	39	36
L45×45×8	5.15	14.2	75	70	65	59	53	48	44	40	37	34
L45×45×6	3.96	13.4	60	56	52	46	42	38	35	32	29	27
L45×45×5	3.34	13.1	51	48	44	40	36	32	30	27	25	23
L35×35×6	3.01	10.9	39	35	30	27	24	22	19	17	15	14
L35×35×5	2.55	10.6	34	30	26	23	21	18	17	15	13	12
L25×25×5	1.77	8.1	17	14	12	10	9					

\# Force resultant assumed to act through centroid of attached leg and half the moment caused by such eccentricity distributed to the strut. K = 0.9

* Multiply C_{re} values by 2 **for angle struts in pairs.**

Table 5.11
DOUBLE ANGLE STRUTS
(interconnected at mid length)



Factored Axial Compressive Resistances, $C_r{}^{+}$, in kN
Legs 6 mm Back to Back*

G40.21-M 300W

Double angle Struts	Mass kg/m	x or y mm	M_{rx} kN•m	Strut lengths between intersections of axes of strut and chords, L= (mm)								
				700	800	900	1000	1100	1200	1300	1400	1500
2L90×90×10	26.7	26.2	10.9	788	785	782	778	773	767	759	750	739
2L90×90×8	21.6	25.5	8.91	589	588	586	584	581	577	573	568	562
2L75×75×10	22.0	22.4	7.45	673	669	662	654	643	631	618	601	582
2L75×75×8	17.8	21.7	6.10	518	515	512	507	501	494	486	476	464
2L65×65×10	18.8	19.9	5.48	585	576	566	551	534	516	497	477	457
2L65×65×8	15.3	19.2	4.51	458	454	448	440	430	419	406	390	374
2L65×65×6	11.7	18.5	3.48	322	320	317	313	309	303	296	288	279
2L55×55×10	15.7	17.4	3.83	481	465	448	431	412	392	371	349	325
2L55×55×8	12.8	16.7	3.16	388	380	367	353	338	322	305	287	269
2L55×55×6	9.80	16.0	2.45	282	278	273	266	258	249	237	224	210
2L55×55×5	8.24	15.6	2.08	224	221	218	214	208	202	194	186	177
2L45×45×8	10.3	14.2	2.07	299	285	270	253	236	218	197	178	162
2L45×45×6	7.91	13.4	1.61	231	221	210	198	185	172	156	142	129
2L45×45×5	6.67	13.1	1.37	189	183	177	168	157	146	133	120	110
2L35×35×6	6.03	10.9	0.94	159	147	133	118	104	92	83	75	68
2L35×35×5	5.10	10.6	0.81	135	125	114	101	89	79	71	64	58
2L25×25×5	3.53	8.10	0.39	70	58	50	43	37	33	29	25	22

* If angles are connected to WT-chords with stems thicker than 6 mm, C_r values err no more than 1% on the conservative side.

= C_r values computed based on $K_x = 0.9$; $K_y = 1$.

\+ Concentrically loaded. Resistance to torsional-flexural instability computed by means of an equivalent radius of gyration method.

Table 5.12
HSS WARREN POSTS (CLASS C)

Factored Compressive Resistances, Cr, in kN
G40.21-M 350W

CLASS C HSS		Mass kg/m	Post lengths between intersections of axes of post and chords, L† in millimetres					
			600	700	800	900	1000	1100
SQUARE HSS	38.1×38.1×3.81	3.81	129	121	114	105	96	86
	38.1×38.1×3.18	3.28	112	106	99	92	85	76
	38.1×38.1×2.54	2.71	93	88	83	77	71	65
	31.8×31.8×2.54	2.20	70	65	59	53	46	40
	25.4×25.4×3.18	2.01	54	46	38	33	28	24
	25.4×25.4×2.54	1.69	47	41	34	29	25	22
ROUND HSS	48.3×3.18	3.54	125	120	114	108	101	94
	42.2×3.18	3.06	104	98	92	85	78	70
	42.2×2.54	2.48	85	80	75	70	64	58
	33.4×2.54	1.93	60	54	49	42	36	32
	26.7×3.18	1.84	48	39	33	28	24	21
	26.7×2.54	1.51	40	33	28	24	20	18

†K = 1.0

Table 5.13
HSS WARREN POSTS (CLASS H)

Factored Compressive Resistances, Cr, in kN
G40.21-M 350W

CLASS H HSS		Mass kg/m	Post lengths between intersections of axes of post and chords, L† in millimetres					
			600	700	800	900	1000	1100
SQUARE HSS	38.1×38.1×3.81	3.81	143	138	132	125	116	107
	38.1×38.1×3.18	3.28	124	120	115	109	102	94
	38.1×38.1×2.54	2.71	103	99	95	91	85	79
	31.8×31.8×2.54	2.20	80	76	71	65	58	50
	25.4×25.4×3.18	2.01	65	57	48	39	32	28
	25.4×25.4×2.54	1.69	56	50	43	35	29	25
ROUND HSS	48.3×3.18	3.54	136	133	129	124	119	113
	42.2×3.18	3.06	115	111	106	101	94	87
	42.2×2.54	2.48	94	91	87	83	77	72
	33.4×2.54	1.93	68	64	59	53	46	38
	26.7×3.18	1.84	57	50	41	33	27	23
	26.7×2.54	1.51	48	42	35	28	23	20

†K = 1.0

Table 5.14
I_s/h^2 Values in mm^2

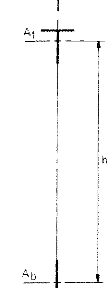
HSS CHORDS

Top Chord / Bottom Chord	Size (mm)	101.6×101.6			101.6×76.2	76.2×76.2	
	Thickness (mm)	9.53	7.95	6.35	6.35	7.95	6.35
	Mass (kg/m)	25.7	22.1	18.2	15.6	15.8	13.1
152.4×101.6×11.13	38.0	1660	1510				
152.4×101.6×9.53	33.3	1570	1440	1270			
152.4×101.6×7.95	28.4	1460	1350	1200			
152.4×101.6×6.35	23.2	1320	1230	1110			
101.6×101.6×9.53	25.7	1390	1290	1160			
101.6×101.6×7.95	22.1		1200	1080			
127.0×76.2×7.95	22.1				992	998	
127.0×76.2×6.35	18.2				911	915	825
127.0×76.2×4.78	14.1				801	805	734
101.6×76.2×9.53	21.9				987	993	888
101.6×76.2×7.95	18.9				926	932	838
101.6×76.2×6.35	15.6				846	850	772

Table 5.15
I_s/h^2 Values in mm^2

WT CHORDS

Bottom Chord / Top Chord	WT180			WT155				WT125			WT100	
Mass kg/m	39.5	36	28.5	33.5	30	26	22.5	22.5	19.5	16.5	15.5	13.5
WT180×39.5	2150	2030	1790	1960	1840							
WT180×36		1930	1710	1870	1760	1630						
WT180×32			1630	1770	1670	1560	1420					
WT180×28.5			1530	1660	1570	1470	1350	1360				
WT180×25.5					1480	1390	1280	1290	1190			
WT180×22.5						1310	1210	1220	1130	1030		
WT155×43	2240	2120		2040	1910							
WT155×39.5	2140	2030	1790	1960	1840							
WT155×37		1970	1740	1900	1790	1660						
WT155×33.5			1660	1810	1710	1590	1450					
WT155×30			1570	1710	1620	1510	1380	1390				
WT155×26					1510	1420	1300	1310	1200			
WT155×22.5						1300	1210	1210	1120	1020		
WT155×19.5							1120	1130	1050	960	939	
WT125×29						1490	1370	1370				
WT125×24.5						1370	1260	1270	1170	1060		
WT125×22.5							1210	1220	1120	1020	1000	
WT125×19.5								1120	1050	958	938	852
WT125×16.5									958	884	867	793
WT100×18										926	907	827
WT100×15.5										867	850	779
WT100×13.5											779	718



Steel truss moment of inertia* (mm^4), $I_s = (I_s/h^2)$ values tabulated multiplied by h^2

$$I_s = 0.85\left[\left(\frac{A_b}{A_t + A_b}\right)^2 A_t + \left(1 - \frac{A_b}{A_t + A_b}\right)^2 A_b\right] h^2$$

h = vertical distance between centroids of chords in mm and 15% reduction due to open web effect included.

CHAPTER 6

6.0 STUB-GIRDER FLOOR CONSTRUCTION

6.1 INTRODUCTION

About 1970, a special composite girder was introduced in the U.S. construction market. This unique steel-concrete composite structural system was dubbed the "stub-girder" by its principal proponent, Colaco[6.1]. The structural arrangement, Fig. 6.1, offered mechanical-structural integration through "natural" openings, and a saving in steel mass due to both the efficiency of the composite girder and the combination cantilever (or Gerber) design and composite design of the beams. The stub-girder system has since seen use by other consultants in the U.S.. A limited number of project oriented load tests were also carried out in the U.S. to supplement theoretical analyses.

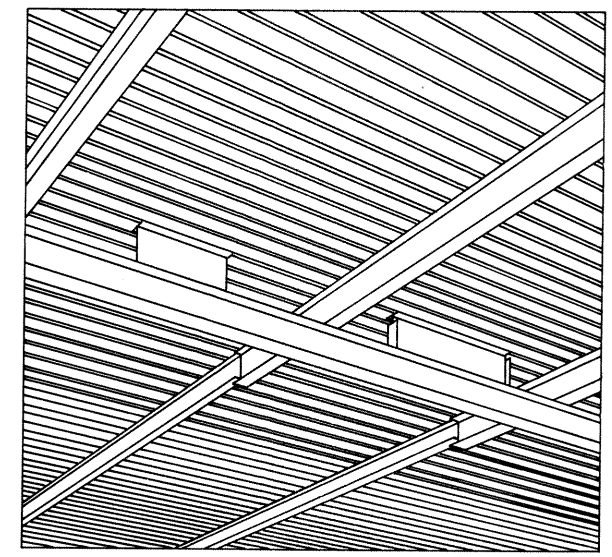


Figure 6.1
Stub-Girder Floor System

In Canada, this structural system has been the subject of several research projects, including comprehensive full-scale tests, and the Canadian research has resulted in several significant changes to the original concept. These changes include steel girder section depths, stiffening of stubs, and slab reinforcement, all of which will be discussed in this chapter. Several projects have now been designed by Canadian consulting engineers and built incorporating some of the results from this Canadian research.

The Stub-Girder System

The stub-girder floor system is basically a gravity-load-carrying floor framing system, although some adaptations to include provision for lateral forces will be discussed later. Stub-girders are vierendeel-girder type assemblies, consisting of a steel W-shape bottom flange or chord, and a

TABLE 6.1 A PARTIAL LIST OF STUB-GIRDER STRUCTURES IN NORTH AMERICA

Project Name	No. of Storeys	Approximate Floor Area (m^2)	Ref. No.
First International Building Dallas	51	176 000	6.8
Mercantile Center St. Louis	35	70 000	6.9
101 Marietta Building Atlanta	35		6.9
Town Center Tower Southfield	32	48 000	6.9
Cullen Center, Dresser Tower Houston	40	93 000	6.1
One Allen Center Building Houston	34	88 000	6.10
Pennzoil Place (twin towers) Houston	37	167 000	6.11
Georgia Power Company Headquarters Atlanta	24	71 000	6.16
One Houston Center Houston	48	100 000	6.12
First City Bank Building Houston	50	130 000	6.13
Nova Corporation Head Office Calgary	37	70 000	6.7
101 California Building San Francisco	48		6.14
ManuLife Place Edmonton	33	112 000	6.15
NRC Building Boucherville, Quebec	2	7 800	
401 West Georgia Vancouver	22	28 000	6.20

concrete deck-slab top flange or chord, with intermittent connections consisting of short lengths of W-shape (known as stubs) connected to both chords to transfer the shear between the two elements. Secondary framing passes through the vierendeel openings and these members are also connected to both top and bottom chords. Ideally, stub-girders span about 12 metres (usually core to exterior wall in a conventional office building) with the secondary framing or floor beams spanning about 9 metres. The system is very versatile, particularly with respect to secondary framing spans with beam depths being adjusted to the required structural configuration and mechanical requirements. Overall girder depths vary only slightly, by virtue of the beam depth (thus stub depth) variation. Traditionally, Canadian research and construction have concentrated on a 310 mm deep bottom chord while most U.S. projects have used 360 mm sections. For a span of greater than 13.5 metres, stub-girders tend to become impractical, with the slab design becoming critical. Girder spans down to 8 metres are worthy of consideration if openings are necessary, or if continuous beam spans are beneficial.

The floor beams, ranging from about 310 to 460 mm in depth, are placed over girders between stubs at about 2.5 to 3.5 metre centres depending upon the structural module and the spanning capability of the selected deck-slab system. The deck-slab system usually incorporates a composite wide-rib profile deck, 51 to 76 mm deep, covered by approximately 85 mm of semi-low density concrete or 65 to 75 mm of normal density concrete, usually no less than 25 MPa in strength.

This somewhat unique floor framing system has generally been used for office floors with live loads ranging from 2.4 kPa to 4.8 kPa, plus partitions. It has also seen use in "special purpose" buildings, such as laboratories, and has received serious consideration for hospital construction.

Structurally, the floor beams are designed based on cantilever and suspended span construction (also known as Gerber construction) where the floor beams are continuous over girders and cut off near the points of inflection to pick up the drop-in suspended spans. The positive moment regions of the floor beams are usually designed compositely with the deck-slab system, to produce savings in structural steel as well as to provide stiffness. The cantilever segments are bolted to the top flange of the steel bottom chord of the stub-girder, while two shear studs are usually specified on each floor beam, over the beam-girder connection, for anchorage to the deck-slab system. The drop-in or suspended span segments are frequently shallower in depth, permitting additional plenum service depth in those areas. Composite design of the drop-in segments is also often found to be beneficial.

The stub-girder is analysed as a vierendeel girder, with the deck-slab acting as a compression top-chord, the full length steel girder as a tensile bottom-chord and the steel stubs as vertical web members or shear panels. It is important to note that, using stub-girder construction, shoring is required for the fresh concrete condition since the top chord has virtually no strength at this stage. Retention of the shoring for a specific period of time is critical and will be discussed later.

This structural system permits the use of steel and concrete at their optimum strengths, resulting in overall efficiency. The natural openings between vierendeel "posts" allow structural-mechanical-sprinkler integration in two directions, permitting storey-height reduction when compared with some other structural framing systems. See Fig. 6.2.

6.2 PROPOSED DESIGN CRITERIA

Since the early seventies more than forty stub-girder framed buildings have been built in North America, totalling more than two million square metres, some of which are listed in Table 6.1. Yet, understanding of an appropriate design technique has not been widespread in the structural engineering community, due primarily to the involvement of a very small number of consulting firms mainly in the United States, and more recently in Canada.

This chapter is intended to provide an overview of the system, along with practical design criteria, for layout, analysis, structural design, detailing and construction techniques for the stub-girder floor system.

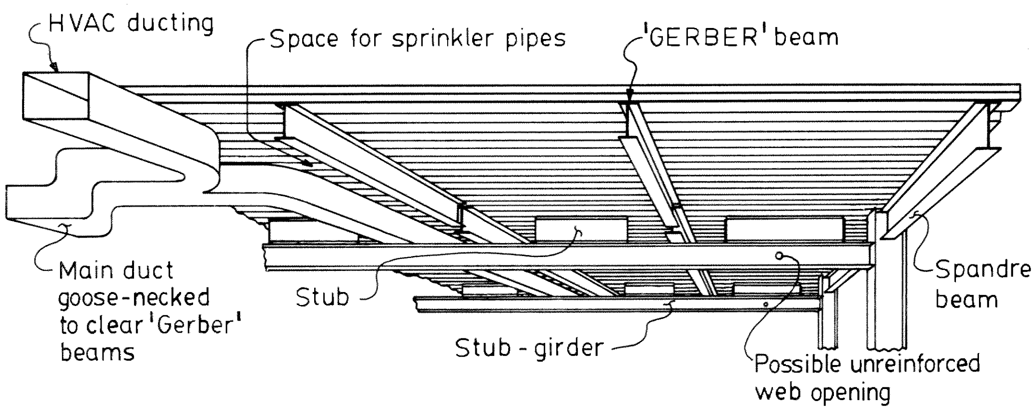


Figure 6.2
Structural-Mechanical-Sprinkler Integration of a Typical Stub-Girder Floor

The following proposed design criteria have been prepared by the authors based on a detailed study of this structural system over a number of years. This study has included in-depth discussions with designers of the bulk of North American projects, viewing of a good number of projects during construction, and participation in both initial design and laboratory testing of several Canadian full-scale girder tests. Further information has been gleaned from some U.S. full-scale tests, and technical journals.

The proposed guidelines for the design and construction of stub-girder floor system cover the following topics:

- Deck-slab considerations
- Stub and beam layout
- Cantilever and suspended span beams (Gerber Beams)
- Depth control and design checks for Gerber Beams
- Structural properties of reinforced concrete top chord (deck-slab)
- Structural modelling of stub-girders for preliminary manual analysis
- Structural modelling of stub-girders for detailed computer analysis
- Stub-girder member strength checks
- Design of transverse slab reinforcement
- Stud shear connection design
- Shear capacity of stubs and stub stiffener design
- Stub-girder deflection checks
- Floor vibration checks
- Shoring checks for stub-girders
- Special design and construction considerations.

6.3 DECK-SLAB CONSIDERATIONS FOR STUB-GIRDER FLOOR SYSTEM

Selection of a suitable deck-slab system for a stub-girder framed floor involves more than just consideration of the load carrying capacity of the deck-slab. The following important points should be noted early in the selection process.

a) A large deck-slab total depth is preferred to produce a large effective slab design width. Steel decks 50 mm or greater in depth are suitable. A stiffer top chord can provide more efficient vierendeel action, which in turn enables more economical sizing of the steel bottom chord.

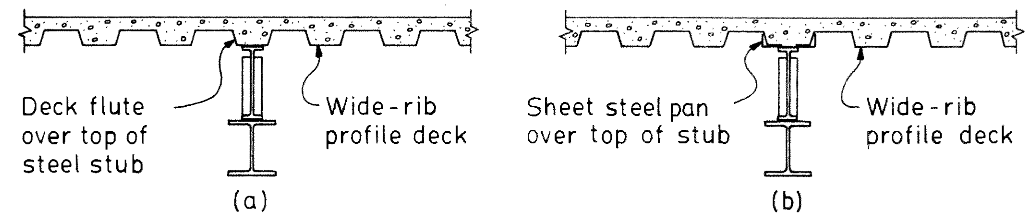


Figure 6.3
Steel Deck Arrangement in Stub-Girder Floors

b) An 85 mm thick semi-low density concrete cover slab on a steel deck with underside unsprayed is often chosen to provide a two-hour fire resistance rating.

c) In locales where semi-low density concrete aggregates are either not normally available or not economical, normal density concrete cover slabs 65 to 75 mm thick may be used. Deck-slab systems of this type have been tested using full-scale assemblies, as will be noted later. If a fire resistance rating is required in such instances, sprayed-on fire protective materials must be applied to the underside of deck.

d) Since the slab will be subjected to both high compression and high shear stresses, concrete strengths are generally recommended to be not less than 25 MPa.

e) A wide-rib profile deck is chosen for the deck-slab to provide adequate concrete area directly above the stub locations, and sufficient width of concrete rib to admit stud connectors. See Fig. 6.3a and Table 1.1.

f) The location of deck flutes in stub-girder floor bays must be planned, so that a concrete rib coincides with a girder module, and it may be necessary to create a special rib coinciding with the

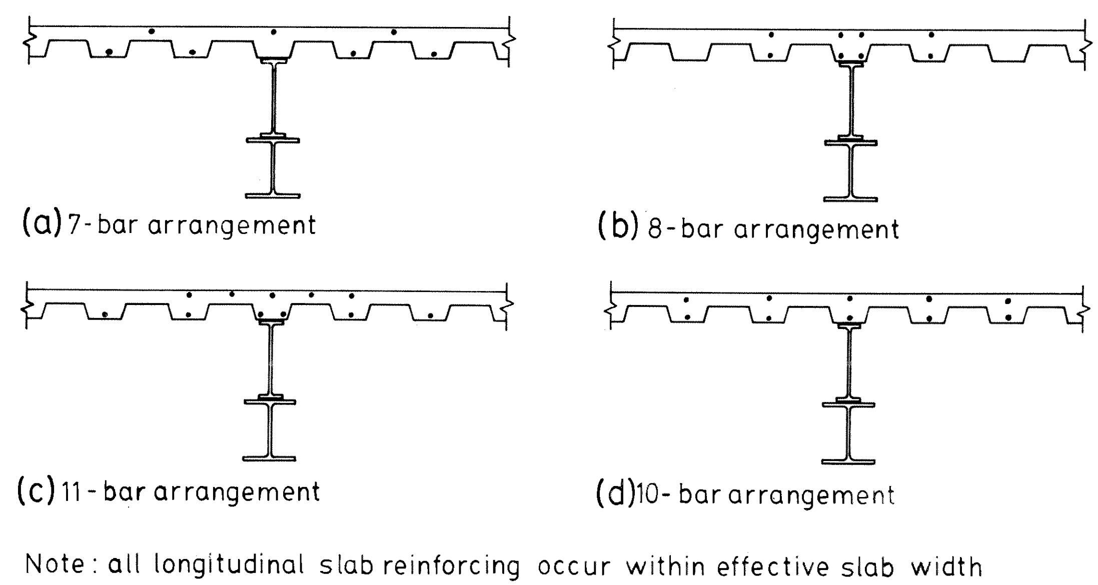
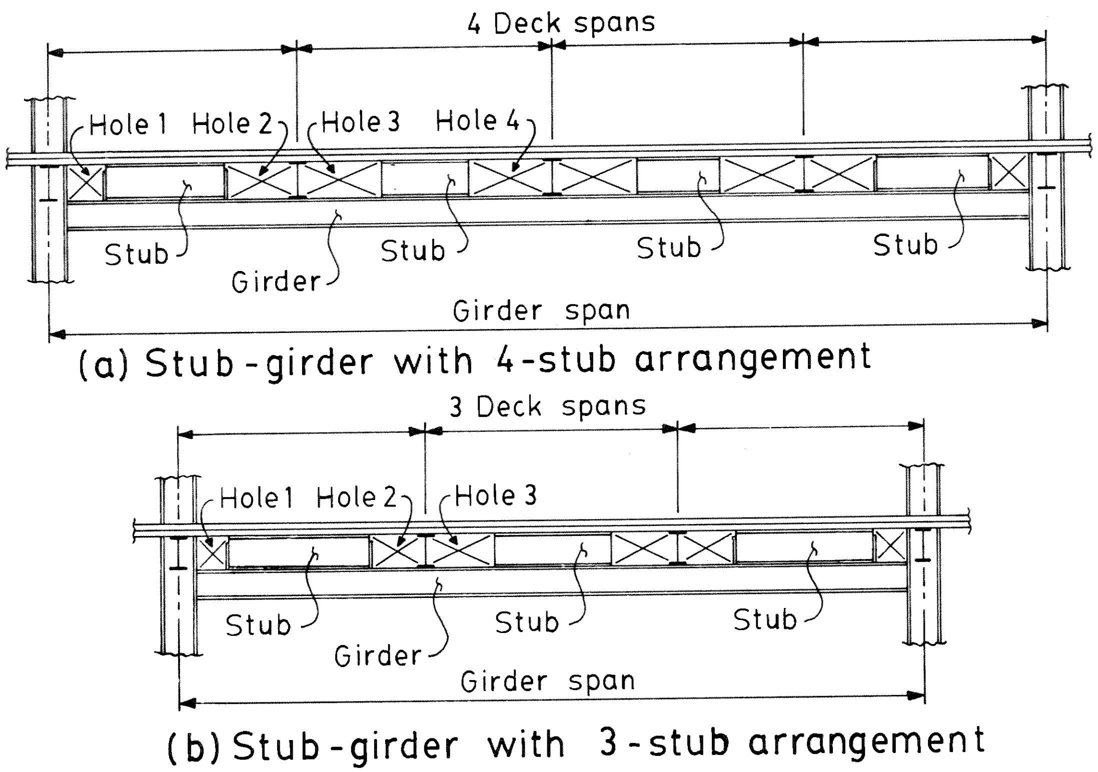


Figure 6.4
Continuous Longitudinal Reinforcing in Deck-Slabs atop Stub-Girders



WEB openings suitable for preliminary manual design of typical stub-girders (as shown above):

Opening Reference Number	Width of opening as percent of girder span	
	Stub-girder with 3-stub arrangement	Stub-girder with 4-stub arrangement
1	*	*
2	7.5 to 8.5	6 to 7
3	9.5	8
4	—	8

* Computed based on bottom chord bending resistance and stub length required for the development of longitudinal slab shear and stub web shear resistance.

Figure 6.5
Typical Stub-Girders
and Proposed Width of Web Openings
Suitable for Preliminary Manual Design

girder using a continuous sheet steel pan. See Fig. 6.3b. The continuous sheet steel pans may only be required on some stub-girder modules, depending on the module of the steel deck selected.

g) Continuous full span longitudinal slab reinforcing, arranged within the effective slab design width, is required at both top and bottom of the deck-slab. This system of slab reinforcement provides added flexural strength, shear strength and ductility to the top chord of the stub-girder. See Fig. 6.4.

6.4 STUB AND BEAM LAYOUT

The majority of stub-girders built to date are in the span range of 11.5 to 13.5 metres, with four deck spans, and thus three intermediate beams, and four stubs between girder end supports. Occasionally, 3-deck spans with 3-stub arrangements or 5-deck spans with 5-stub arrangements have been used. Recommendations for width and position of web openings of typical stub-girders, illustrated in Fig. 6.5 for preliminary design purposes, assume 'optimum' use of structural material, whilst maintaining maximum natural web openings for mechanical ducting and sprinkler pipe layouts.

6.5 CANTILEVER AND SUSPENDED SPAN BEAMS (GERBER BEAMS)

To achieve more efficient distribution of moments, and to enable more economical use of structural materials both at centre span and near the supports, the century old Gerber construction, coupled with the relatively new hollow-composite construction, is used for the design of floor beams in the stub-girder floor system. See Fig. 6.6. The cantilever segments are designed on the assumption that negative moment regions at supports (i.e. at a beam-stub-girder intersection) are non-composite, and that positive moment regions are either composite or non-composite, depending upon beam span, member size and floor loading. The drop-in (or suspended) segments are designed as simply supported beams of either composite or non-composite construction.

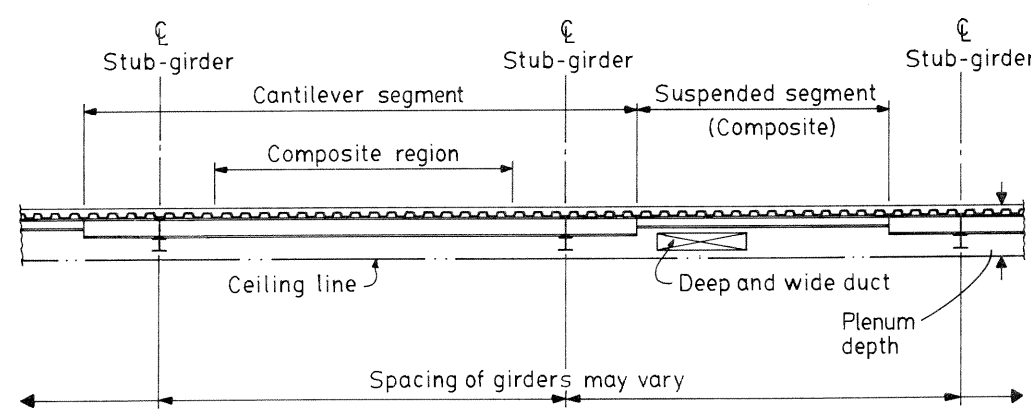


Figure 6.6
Cantilever and Suspended Span Beams (Gerber)
Construction with Optional Composite Design at
Positive Moment Regions

Considerations when selecting the depth of cantilever segments should include strength of stub-girder, depth of duct openings, storey height, total floor steel mass, etc. Since the tensile component of the stub-girder is separated from its top chord by the depth of floor beams forming the cantilever segments, a change in the depth of these beams can significantly influence the governing design forces at several critical design locations for both the deck-slab top chord and the steel bottom chord.

Suspended segment locations of the beam system are often selected to coincide with the occurrence of main air supply ducts feeding out of a building core. See Fig. 6.6. Using shallow beam sections at these locations, large supply ducts can easily be accommodated within a relatively shallow plenum.

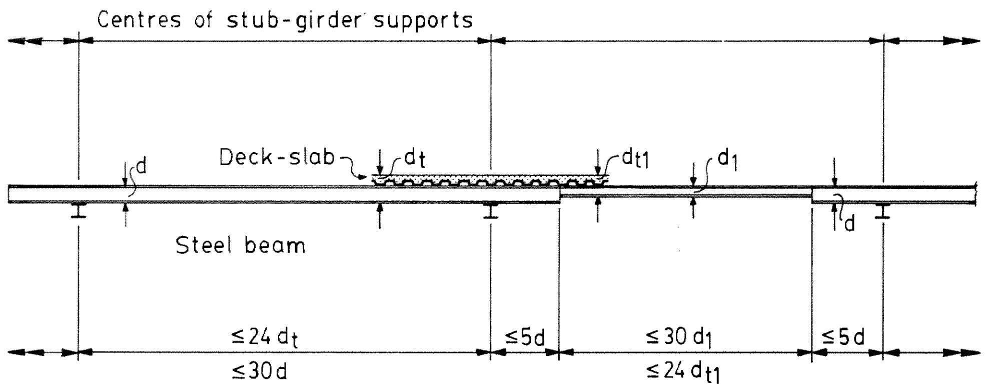


Figure 6.7
Depth Control for
Cantilever and Suspended Span Beams

6.6 DEPTH CONTROL AND DESIGN CHECKS FOR GERBER BEAMS

During preliminary design of a Gerber beam, it is convenient to know how to proportion the beam in relation to its span and where and what design checks are required. The following is a list of proposed design considerations for Gerber member selection.

– Cantilever segments (See Fig. 6.7)

a) Clear span/depth of steel shape to be ≤ 30.
   Clear span/depth of composite section to be ≤ 24.
b) Cantilever arm length/depth of steel shape to be ≤ 5.
   (See Fig. 6.8 for shear stud connection at end of cantilever arm).
c) Cantilever arm length ≤ unsupported length, L_u, for maximum M_r of steel section.

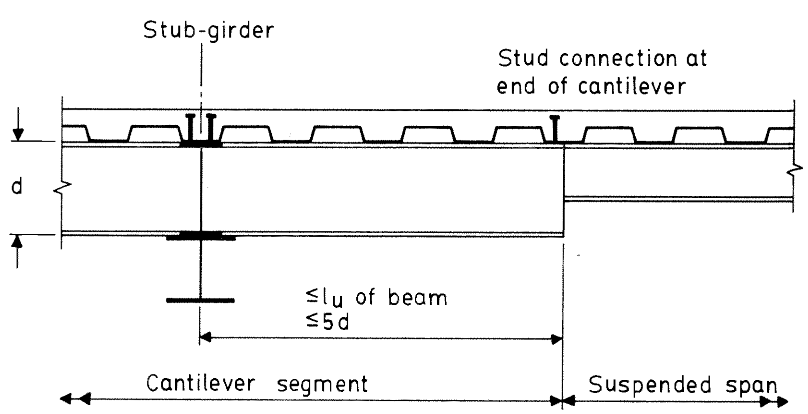


Figure 6.8
Cantilever Arm Proportioning

d) Check maximum negative bending and shear at the support of cantilevers.
e) Check mid-span positive bending (composite section) with zero live load on both cantilever arms and adjacent suspended spans.
f) When designed compositely, possible yielding of bottom steel fibre at positive moment area should be checked for maximum combined fresh-concrete condition loading and superimposed dead and live loads when the beam is unshored.
g) Check deflections at mid-span and at ends of cantilever arms.
h) Determine whether cambering for wet concrete loads is required.

– Suspended segments (See Fig. 6.7)

a) Suspended span/depth of steel shape to be ≤ 30.
   Suspended span/depth of composite section to be ≤ 24.
b) Check bending at mid-span and shear at supports.
c) When designed compositely, check possible yielding of bottom steel fibre under maximum combined fresh-concrete condition loading and superimposed dead and live loads, if unshored.
d) Check deflections at mid-span.
e) Determine whether cambering for wet concrete loads is required.

6.7 STRUCTURAL PROPERTIES OF REINFORCED CONCRETE TOP CHORD (DECK-SLAB)

The width of reinforced concrete deck-slab, deemed to be effective for stub-girder analysis and design, may be calculated using S16.1 rules, as illustrated in Section 1.4 and Figure 1.13. However, the effective concrete compression area of the top chord of a stub-girder also includes the cross-sectional area of the concrete ribs below the cover slab thickness. In addition to the concrete area, cross sectional areas provided by longitudinal reinforcing bars and deck steel should also be included during the computation of top chord structural properties. For more details see calculations in the worked example 6.19.

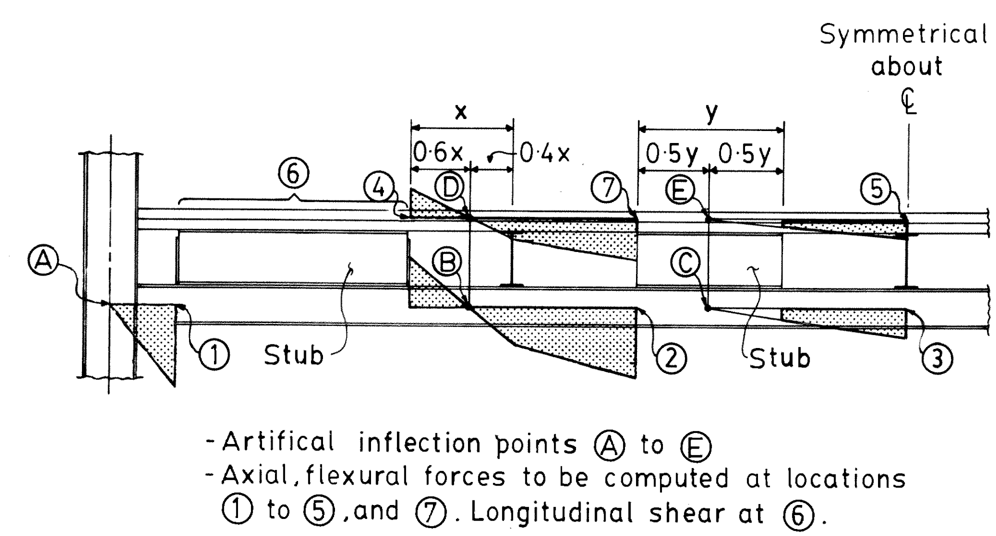


Figure 6.9
Simplified Stub-Girder Analysis Model
(Four-Stub Arrangement)

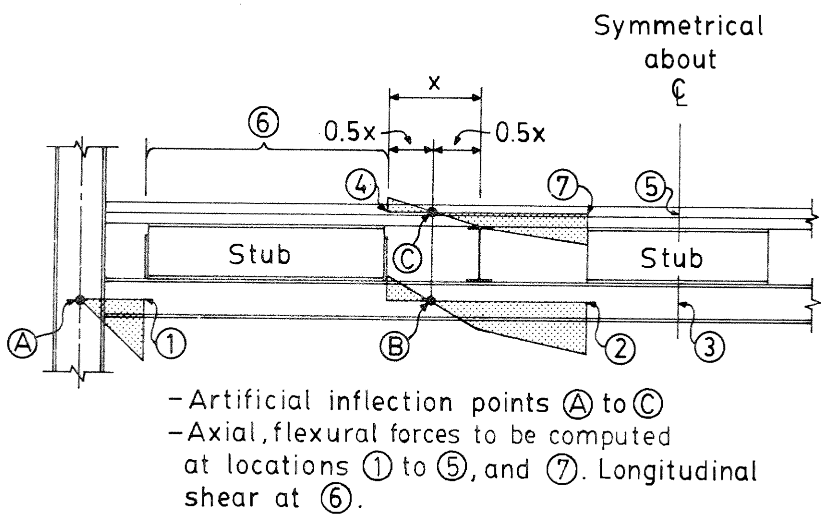


Figure 6.10
Simplified Stub-Girder Analysis Model
(Three-Stub Arrangement)

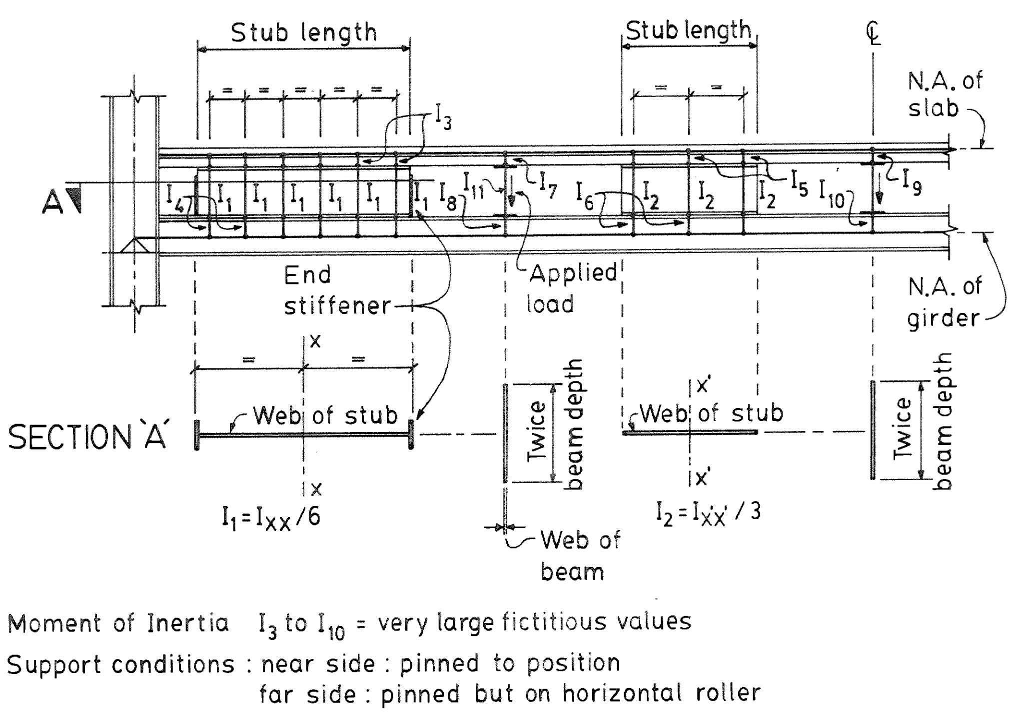


Figure 6.11
Structural Modelling of a Typical Stub-Girder
for Detailed Analysis Using a Computer

6.8 STRUCTURAL MODELLING OF STUB-GIRDERS FOR PRELIMINARY MANUAL ANALYSIS

For preliminary analysis, a stub-girder can be modelled as a symmetrical "Vierendeel" girder with artificial "hinges" as indicated in Figures 6.9 and 6.10 for four-stub and three-stub arrangements respectively. Introduction of the hinges makes the structural model statically determinate, and greatly simplifies the preliminary design. Shear forces at hinge locations in top and bottom chords are proportioned according to the calculated member stiffness of the top chord and the stiffness of a trial bottom chord section. Girder deflection can be calculated by summing the flexural deflections of all subcomponents and the computed girder deflection due to axial deformation of top and bottom chord members. For detailed manual calculations and comparison of their accuracy with computer analysis results, see worked example 6.19.

6.9 STRUCTURAL MODELLING OF STUB-GIRDER FOR COMPUTER ANALYSIS

Computerized structural analysis of stub-girders, through finite element modelling and vierendeel-girder modelling, was illustrated by Colaco[6.1] in 1972. Both structural models were found to provide varying degrees of success in the prediction of mid-span deflection, concrete stresses, and steel stresses, when compared to test results of a full-scale girder specimen. The finite element model was not found to be suitable for practical design use due to its excessive demand on computer and human resources. However, it did serve its purpose for research studies by verifying the suitability of the vierendeel-girder model.

A vierendeel-girder model of a typical four-stub arrangement stub-girder is shown in Fig. 6.11. Note that the stub pieces between top and bottom chords have been 'transformed' into a series of vertical posts. A 'plane frame' type of stiffness analysis computer program can be used to obtain member forces. This analysis method was used to provide design forces for five full-scale stub-girder test specimens which were tested at two Canadian universities[6.2,6.3,6.4]. Correlation between calculated forces and girder performances was found to be excellent. With the aid of a micro-computer, a designer can economically analyse and design such a girder. A step by step illustration of this modelling technique is included in the worked example in Section 6.19.

6.10 STUB-GIRDER MEMBER STRENGTH CHECKS

During both preliminary and detailed structural analysis, a designer should check strengths of top and bottom chord members at several critical locations. For a typical three- or four-stub arrangement stub-girder (see Figs. 6.9 and 6.10), up to 7 locations are generally considered critical. The appropriate design checks required for each of the locations shown are specified below:

Location 1	– check bending
	– check shear
Location 2	– check combined bending and axial tension
Location 3	– check combined bending and axial tension
Location 4, 5, 7	– check axial compression plus flexural compression or tension (for full effective slab width)
Location 6	– check local compression and longitudinal slab shear adjacent to stubs (see Section 6.11 below)

6.11 DESIGN OF TRANSVERSE SLAB REINFORCEMENT

Failure to provide adequate longitudinal shear resistance may cause a stub-girder to exhibit non-ductile behaviour at ultimate failure load. Figure 6.12 illustrates a cut-away view of a stub-girder at the end stub location. The total shear force delivered by the end stub to the deck-slab can be represented by two planes of longitudinal slab shear plus an area of local concrete

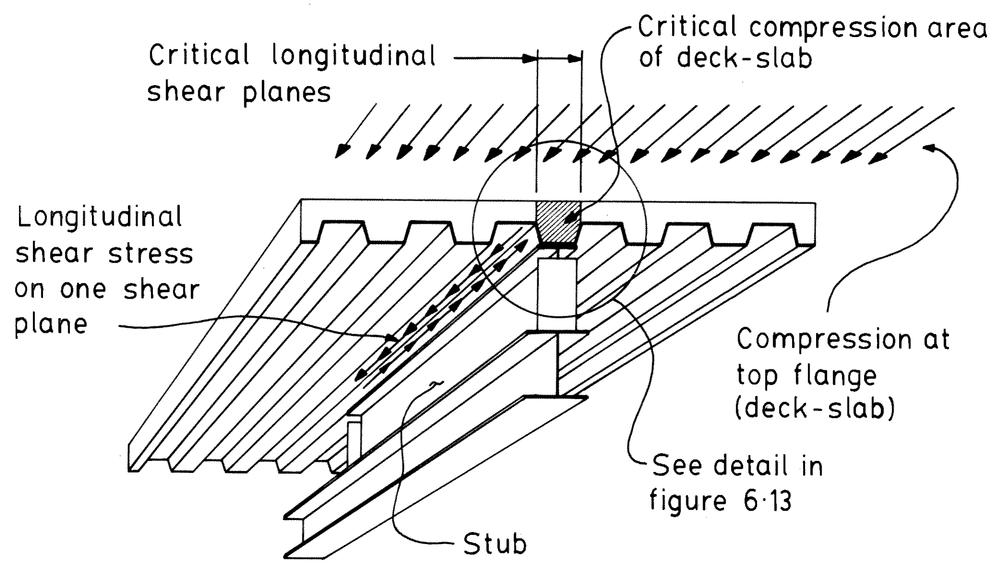


Figure 6.12
Cut-Away View of a Stub-Girder (at End-Stub Location)

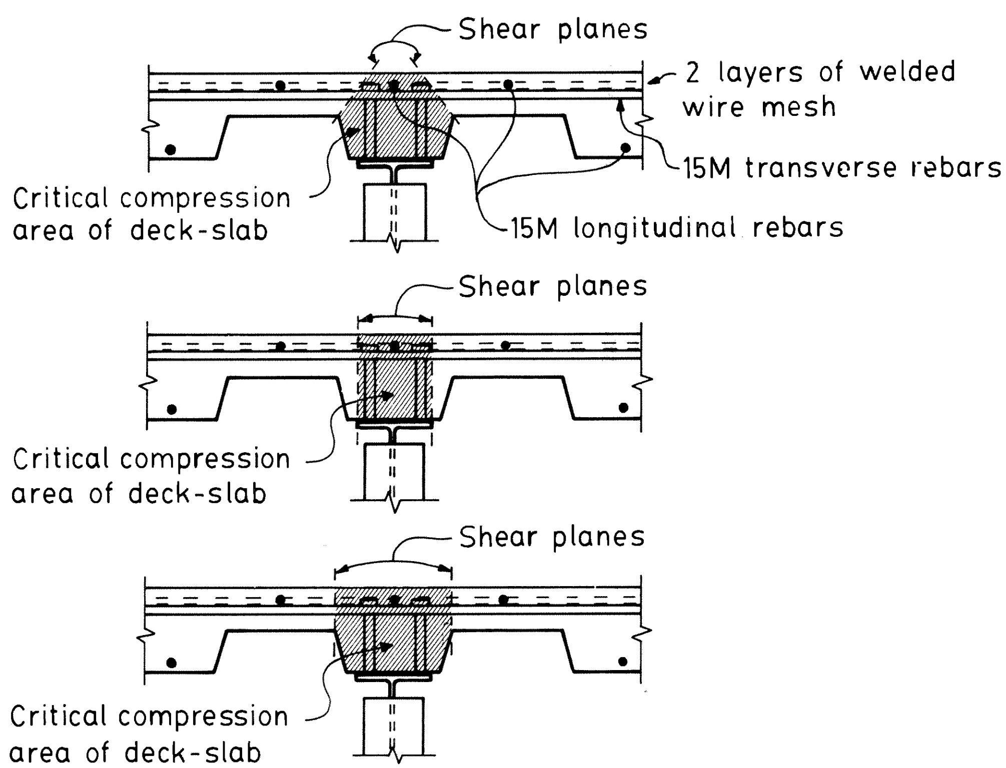


Figure 6.13
Idealized Failure Modes under Longitudinal Slab Shear and End-Stub Slab Compression

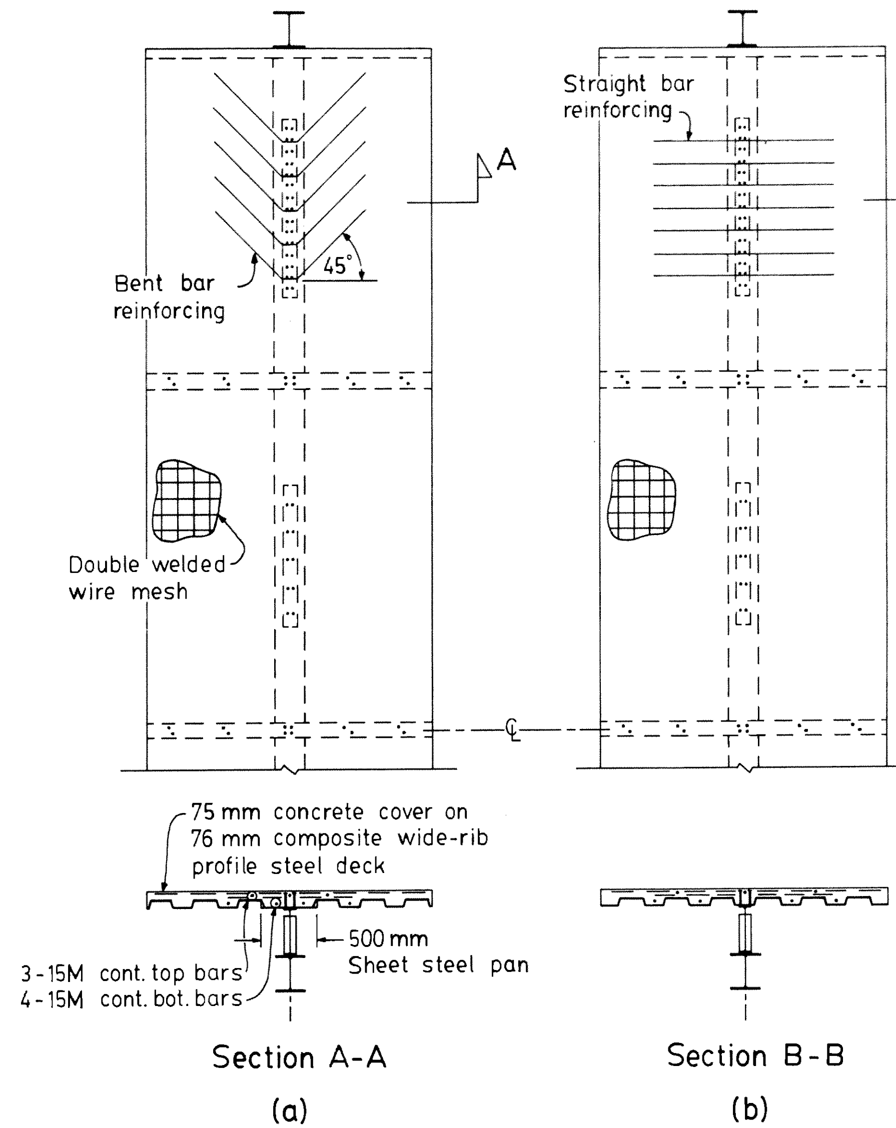


Figure 6.14
University of Saskatchewan Test Specimens

compression at the concrete rib directly in front of the end stub. (See typical detail in Fig. 6.13). A rational design method permitting evaluation of the longitudinal shear capacity of deck-slabs, with or without transverse reinforcement, was proposed by Buckner, et al[6.5]. The details of this design method are presented in Section 4.9.

Three full-scale lab-tests were conducted at the University of Saskatchewan[6.4] on stub-girders spanning 12 metres. Three different deck-slab transverse reinforcing details were chosen for the same stub-girder steel and stub sections, i.e. using two layers of welded wire mesh, two layers of welded wire mesh plus straight transverse bars, and two layers of welded wire mesh plus bent bars in a herring-bone pattern (Fig. 6.14). In addition, the third specimen also incorporated a wide centre rib formed by a sheet steel pan. Ultimate failure of each specimen was caused by crushing of the concrete rib at the inside end of an end-stub accompanied by a splitting/shear failure on a horizontal plane just below the heads of shear studs, at the level of transverse reinforcement. (See Fig. 6.15). It was suggested by the researchers that when compared with results of the specimen incorporating straight bars, initial slab cracking can be delayed by using a wide centre rib along with the herring-bone transverse reinforcing. Delay of the crushing failure mechanism for the herring-bone specimen permitted increased deformation in the steel components and greatly improved the ductility of the stub-girder. Also, the ultimate capacity of the herring-bone reinforced girder was increased by 15%. In the opinion of the authors, use of the herring-bone pattern transverse reinforcement, resulting in a direct link from the stud shear connectors to the lines of principal tensile stress in the concrete, distributes the concentrated shear force from the studs to a wider section of slab, thus lowering the critical compressive slab stress. Further testing should be carried out to verify this opinion.

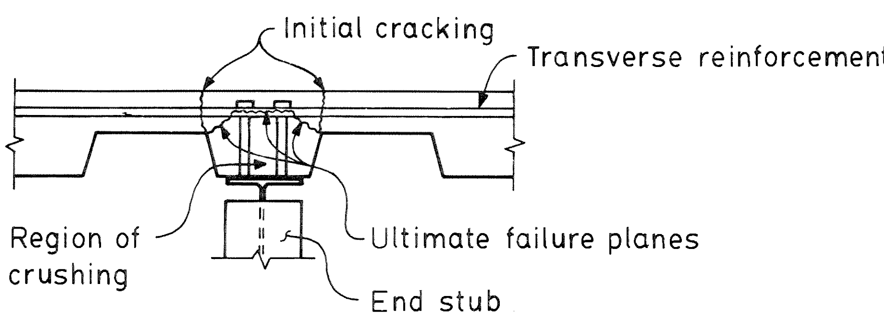


Figure 6.15
Slab Failure Mechanism

6.12 STUD SHEAR CONNECTION DESIGN

Stud shear connectors are commonly used to provide shear transfer at interfaces between slabs and stubs. The combined effect of two types of forces, namely shear forces and direct tensile forces, is considered in the stub-to-slab connections. A conservative approach is adopted, so that each stud installed provides either shear or tensile resistance, but not both. The factored shear resistance of studs can be computed using equation 2.3. The factored pull-out resistance of stud connectors may be computed[6.6], taking into account the stud embedment lengths and the amount of shear cone overlap.

Having calculated the number of stud connectors per stub based on the above-stated design assumptions, the total number is arbitrarily increased by 50% to prevent a failure mechanism occurring at the stub-to-slab interface. It is believed that this design provision, coupled with the proper transverse reinforcing design of deck-slab system, can force the failure mechanism into a combined stub-web shear yielding and concrete slab compression shear failure, (see Fig. 6.16) at ultimate load.

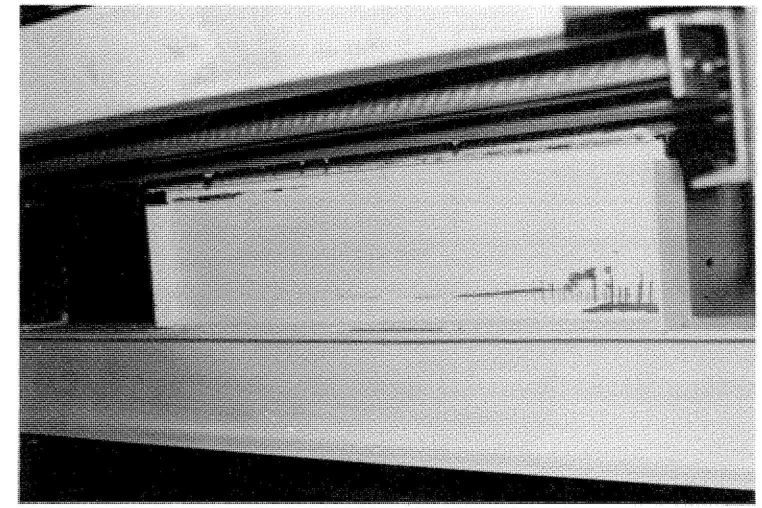


Figure 6.16
Shear Yielding of Stub-Web

6.13 SHEAR CAPACITY OF STUBS AND STUB STIFFENER DETAILS

In stub-girder design, the depth of both stubs and beams must be the same to provide a level support for the steel deck. Therefore, when light wide-flange beams, with thin slender webs, are selected for beams and stubs, some stiffening of the webs of stubs can be expected. Structural analysis of stub-girders generally shows larger horizontal shear forces and overturning moments in the outer stubs, with the interior stubs subjected to only moderate amounts of shear and overturning. Hence, web stiffeners are usually required only on the outer stubs to develop the required stub-web shear and overturning resistances.

Full scale sub-assemblage specimens of stiffened and unstiffened end-stubs, with slabs, were tested by Zimmerman and Bjorhovde[6.2,6.3]. Capacities of stubs, stiffened with fitted web stiffeners, partial end-plate stiffeners, and no stiffeners, were compared. Details of several feasible stub-web stiffening configurations are shown in Figure 6.17. The partial end-plate stiffened stubs proved to provide adequate load capacity with the simplest fabrication. Also, full scale stub-girder tests were conducted to verify the analytical evaluation and the sub-assemblage tests.

6.14 DESIGN OF WELDMENTS AT STUB TO GIRDER INTERFACE

During the design of a stub-girder, it is desirable to select compatible stub and girder sections for the purpose of welding the stub bottom flange to the girder top flange. A stub to girder flange width difference of at least 17 mm should be allowed to permit the use of 8 mm fillet welds, although 10 mm fillet welds can also be used where the difference of flange width at stub to girder interface is greater than about 22 mm. A rational analysis and design of weldments (as shown in the example problem 6.19) for effects of horizontal shear and overturning-induced tension can easily be adopted by a designer. In the case of short girder spans and long beam spans, the stub flange width may exceed the girder flange width. In such cases the stub to girder welds can be made with the girder in the up-side-down position, or other suitable details, such as trimming the stub bottom flange to a suitable width, can be considered.

6.15 STUB-GIRDER DEFLECTION CHECKS

With the deep out-to-out structural depth, a stub-girder should not deflect significantly due to axial strains in top and bottom chord members. However, due to the effect of web openings, a stub-girder behaves like a vierendeel girder. Additional deflection due to "web action" must be accounted for in the total girder deflection. Also, since the composite section relies totally on a reinforced concrete top flange, concrete creep and shrinkage will have an impact on deflection.

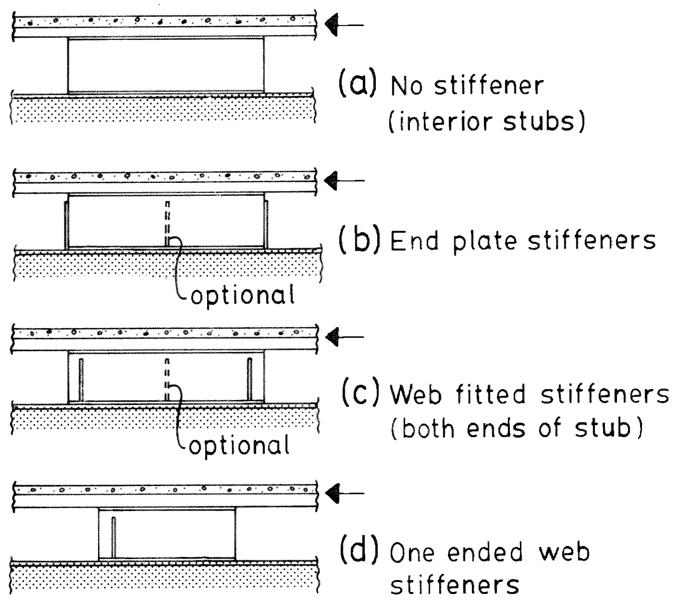


Figure 6.17
Stub-Web Stiffener Details

When designing a stub-girder, deflection of the girder under several loading conditions must be checked:

- Deflection of the stub-girder under steel deck and beam loading is small. However, this deflection should be recognized during the shoring operation, when the girder can be returned to its theoretical elevation using the shores as jacks.

- Deflection of the stub-girder at time of shore-removal is calculated to determine whether girder camber is required. If cambering is chosen, inclusion of steel dead load deflection should be considered. Also, cambering with shoring jacks, in lieu of shop cambering, may be considered. Flexural strains induced in the girder by jacking will be small compared to the axial tensile strains imposed on the steel bottom chord in the fully loaded girder assembly.

- Deflection of the stub-girder under long term dead plus live loading is computed so that comparison can be made with acceptable deflection limitations to ensure the integrity of non-structural building elements.

Elastic deflections of a typical stub-girder with equally spaced beams and symmetrical loading can be manually computed with good accuracy through a process of summing the deflections due to "chord action" and "web action" of the stub-girder structural elements (see worked example calculations Section 6.19). For the computation of more "exact" elastic deflections of stub-girders with symmetric or asymmetric geometry and loading, a plane-frame stiffness analysis computer program can be used to analyse the vierendeel equivalence of the actual stub-girder structure. An estimate of long term loaded girder deflection is also possible by reducing the concrete modulus, E_c. A reduction factor of 2.5 to the value of E_c as computed by equation 2.4 is considered satisfactory.

6.16 FLOOR VIBRATION CHECKS

Field measurements of dynamic response of a typical stub-girder floor to the heel-drop test were carried out by Matthews et al[6.18]. It was determined that, under the bare floor condition (i.e. no ceiling, ducts, carpet, etc.), the interaction of floor beams and stub-girders results in a complex

dynamic response with four modes of vibration occurring in frequency range of 5 to 7 Hz. The average value of damping of the bare floor alone was 2.5 percent of critical, significantly better than would be expected from a purely composite system. It was concluded that the floor system in the evaluated structure is stiff and presents no vibration problems. It should be noted that the use of W460 beams, selected to maximize mechanical openings, deepened the stub-girder on the 12.5 metre span and provided a stiffer beam assembly on the 9 metre span than might be chosen if using only structural considerations.

The procedure for manual calculation of vibration characteristics of stub-girder floor bays is only marginally more complicated than that for a hollow-composite floor bay. See Chapter 7 for example calculations. It involves the selection of an "equivalent" composite girder, having the same force to deflection relationship as that of the stub-girder incorporating the same deck-slab system. As a result, the natural frequency of a floor bay may be computed, to be followed by the computation of peak acceleration due to the specified impulse floor load simulating the effects of "typical" heel drops. Rules given by Appendix G of S16.1 may be used to evaluate the required damping of stub-girder floor bays. For more information see Chapter 7.

6.17 SHORING CHECKS FOR STUB-GIRDERS

A stub-girder design is based on the assumption of shored composite construction. Careful structural evaluation of stub-girders and shoring members for effects of shoring and construction sequence, is essential. A tendency to "over shoring" can occur when too many levels of floors are left shored during construction, causing the lowest floor girder to be overloaded. On the other hand, premature removal of shores can lead to girder deflections greater than anticipated or can even cause failure if the concrete has not reached a satisfactory strength level. For multi-storey applications, a maximum of 5, and a minimum of 3, shored levels is common. Figure 6.18 illustrates an analytical procedure for the assessment of maximum load effects on shores and shored members under an assumed construction sequence. Seven analytical models are needed in this case. Each model is loaded at the top floor with an applied force, F, calculated for the fresh-concrete condition with construction load allowance due to the top-most floor. At the same time, the structural models, with shores removed at bottom-most level, are loaded at shore locations with downward forces, T, equal to the previously calculated shoring forces (see Fig. 6.18). Final forces carried by the shores are calculated based on the method of superposition.

6.18 SPECIAL DESIGN AND CONSTRUCTION CONSIDERATIONS

A stub-girder floor system is a unique floor framing system, which requires the use of a structural reinforced concrete slab to carry both axial and flexural loads. Several design and construction considerations which are essential and unique to this floor system are listed below.

Cambering of Beams and Stub-Girders

Long span Gerber beams and stub-girders often require shop camber to ensure a theoretically "flat" floor after shore removal. In general, Gerber beams should be cambered when the computed deflection under fresh-concrete condition load approaches 20 mm. Stub-girders should be cambered when the calculated deflection of girders at shore removal exceeds 15 mm. Cambering of stub-girders can best be done subsequent to the welding of stub pieces atop the girder members, otherwise part of the camber may be released during the welding process. Field cambering with shoring jacks may be considered as an alternative to shop camber.

The Use of Screed Discs

Since a stub-girder depends on the reinforced concrete deck-slab to perform as a top chord, any significant deviation in the thickness of cover slab at critical girder locations can greatly affect the overall girder strength and stiffness. Therefore screed discs, attached to the top of the stub flanges,

ANALYSIS MODEL	Loading Condition	Composite Girder Stiffness	Shoring forces at floor levels shown	Objective
(1)	F F S_{11} S_{11}	non-composite	S_{11}	to calculate maximum shoring forces
(2)	F F S_{21} S_{21} S_{22} S_{22}	non-composite 3-days old	S_{21} $S_{22} + S_{11}$	
(3)	F F S_{31} S_{31} S_{32} S_{32} S_{33} S_{33}	non-composite 3-days old 6-days old	S_{31} $S_{32} + S_{31}$ $S_{33} + S_{22} + S_{11}$	
(4)	F F S_{41} S_{41} S_{42} S_{42} S_{43} S_{43} S_{44} S_{44}	non-composite 3-days old 6-days old 9-days old	S_{41} $S_{42} + S_{31}$ $S_{43} + S_{32} + S_{21}$ $S_{44} + S_{33} + S_{22} + S_{11}$	
(5)	F F S_{51} S_{51} S_{52} S_{52} S_{53} S_{53} S_{54} S_{54} S_{55} S_{55}	non-composite 3-days old 6-days old 9-days old 12-days old	S_{51} $S_{52} + S_{41}$ $S_{53} + S_{42} + S_{31}$ $S_{54} + S_{43} + S_{32} + S_{21}$ $S_{55} + S_{44} + S_{33} + S_{22} + S_{11}$	
(6) shore removal	F F S_{61} S_{61} S_{62} S_{62} S_{63} S_{63} S_{64} S_{64} S_{65} S_{65} T_6 T_6	non-composite 3-days old 6-days old 9-days old 12-days old 15-days old	S_{61} $S_{62} + S_{51}$ $S_{63} + S_{52} + S_{41}$ $S_{64} + S_{53} + S_{42} + S_{31}$ $S_{65} + S_{54} + S_{43} + S_{32} + S_{21}$ $T_6 = S_{55} + S_{44} + S_{33} + S_{22} + S_{11}$	to calculate maximum load on shored girders
(7) shore removal	F F S_{71} S_{71} S_{72} S_{72} S_{73} S_{73} S_{74} S_{74} S_{75} S_{75} T_7 * T_7	non-composite 3-days old 6-days old 9-days old 12-days old 15-days old 18-days old	S_{71} $S_{72} + S_{61}$ $S_{73} + S_{62} + S_{51}$ $S_{74} + S_{63} + S_{52} + S_{41}$ $S_{75} + S_{64} + S_{53} + S_{42} + S_{31}$ $T_7 = S_{65} + S_{54} + S_{43} + S_{32} + S_{21}$ *compute final deflection of girder	

Figure 6.18
Analysis of Shoring Forces under an Assumed Construction Sequence

indicating the desired depth of concrete cover slab, are often used to ensure that the cover slabs are screeded to the calculated thickness. See Ref. (6.7), for details of deck-slab construction.

Installation and Inspection of Shear Studs

Field applied shear studs should be installed and inspected in accordance with the procedures recommended by the latest version of CSA Standard W59 (also see Chapter 2). In addition, all shear studs atop stubs of all stub-girders should be inspected by experienced personnel. In general, shear studs exhibiting visual weld defects or producing lower pitched sounds when struck by a hammer should be further tested by bending to approximately 15 degrees off perpendicular towards the nearest support of the girder.

Girder-to-Column Connections

A typical girder-to-column connection is shown in Figure 6.19. It should be noted that when slabs are continuous beyond columns (and thus the end of the stub-girder), final tightening of bolts at the connection is sometimes done after the removal of shoring, so that slabs at column locations are free from construction stresses induced by an otherwise fixed-end condition; as a result, the deck-slabs are less susceptible to premature cracking. Crack-control reinforcement of slabs adjacent to column locations is also recommended, see Fig. 6.20.

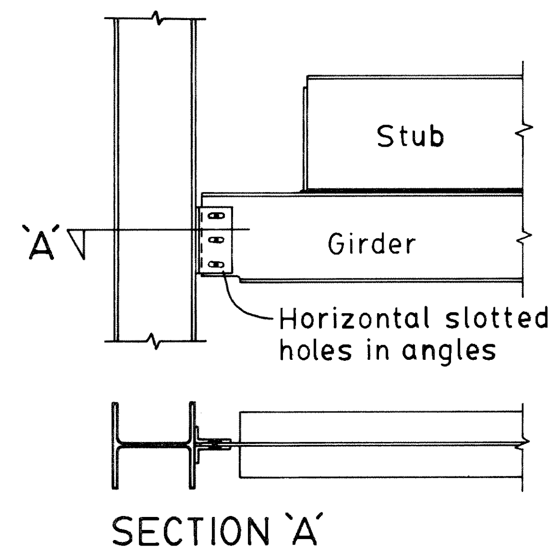


Figure 6.19
Typical Girder-Column Connection

Stub-Girder with Electrified Floor Deck

An underfloor raceway system to accommodate electrical power, as well as telephone and other communication systems, can be incorporated in a stub-girder floor framing system. Headers are thus placed in the deck-slab for cross-distribution purposes. Use of either a standard header or a trench header eliminates a portion of the concrete top chord. Removal of a portion of the concrete cover slab may require the development of an alternate means of load transfer. To minimize the problem, a header duct should be placed as close to a support as possible, with the end stub extended under the header duct. Bottom chord members near supports are often able to carry design moments without the aid of the concrete top chord. Provision must also be made for the transfer of diaphragm shears through the deck, in the case of core lateral load resisting structures. A typical detail at a trench header location on a stub-girder is shown in Fig. 6.21.

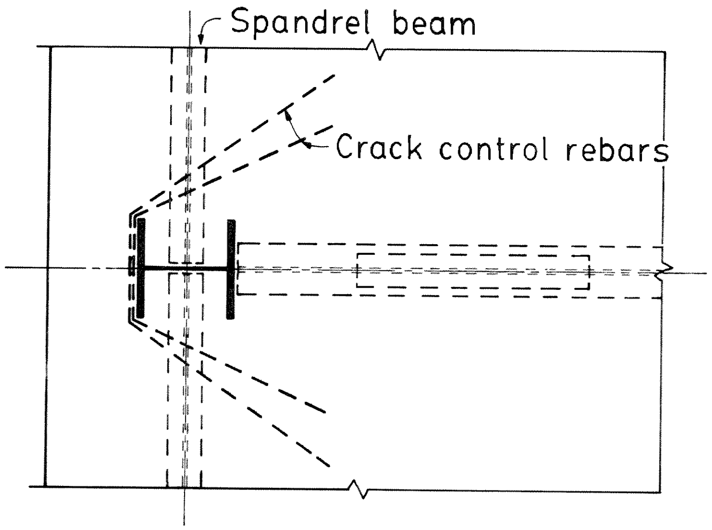


Figure 6.20
Crack Control ReBars at Column Support

Stub-Girders with One-Sided Deck-Slab Overhang

A stub-girder may be situated next to a stairwell or to an exterior wall such that a reinforced deck-slab extends only a short distance to one side of the girder member. A method of determining effective slab width as illustrated in Section 1.4 and Figure 1.13 may be used. The effective concrete compression area should be calculated in the same manner as described in Section 6.7. The stub-girder design methodology outlined in this chapter should also be followed during the design of stub-girders with one sided deck-slab overhang. Such an application might also be found around the perimeter of a building, with beams running over the girder to support a cantilevered floor system as shown in Fig. 6.22.

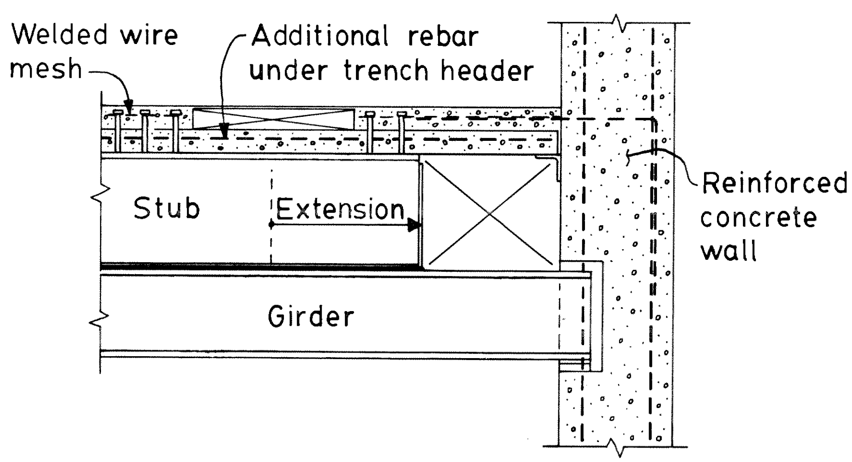


Figure 6.21
Typical Construction Detail at Trench Header Location

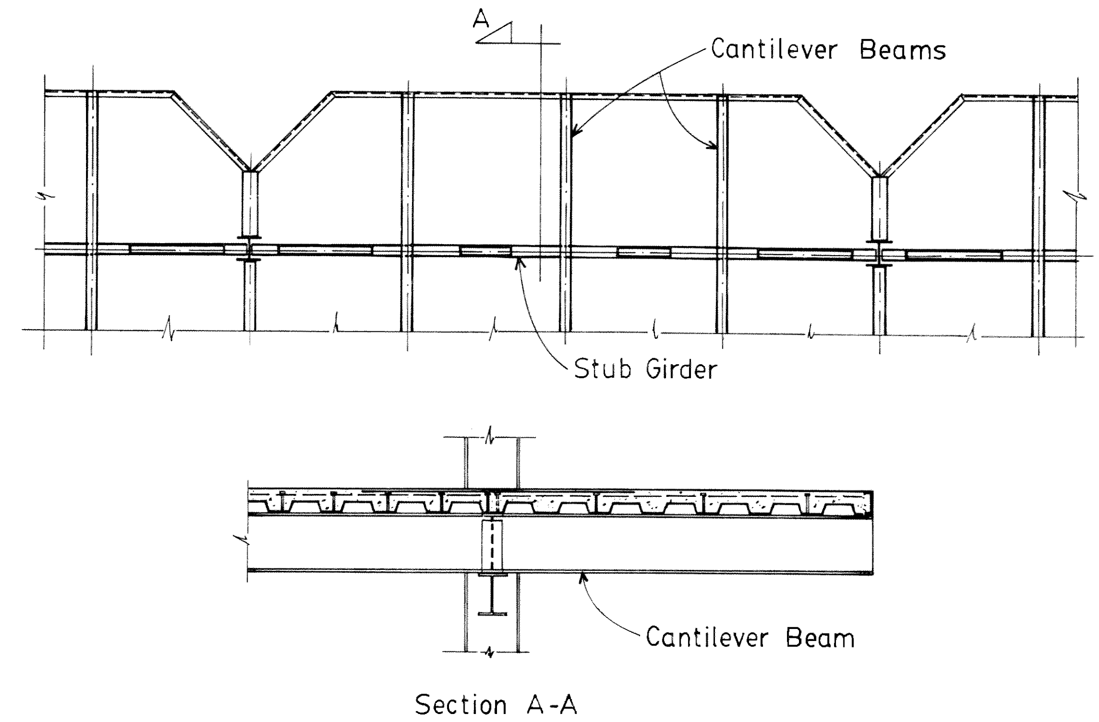


Figure 6.22
Construction of Cantilevered Floor Bays

Upgrading Existing Stub-Girders for Heavier Loading

One very important consideration in office building construction, where tenant requirements are unknown during the design and construction stages, is in the area of specified live load. Occasionally, a designer is required to upgrade a portion of a floor for heavier occupancy loading, after the completion of construction. Like other types of steel floor construction, a stub-girder framed floor can be strengthened to accommodate changes of occupancy loading. The key to stiffening of a girder lies primarily in the reduction of local moments introduced in the top compression chord due to local bending, by blocking existing web holes with plates, and, where necessary, by introducing a longitudinal plate to strengthen the bottom tension chord.

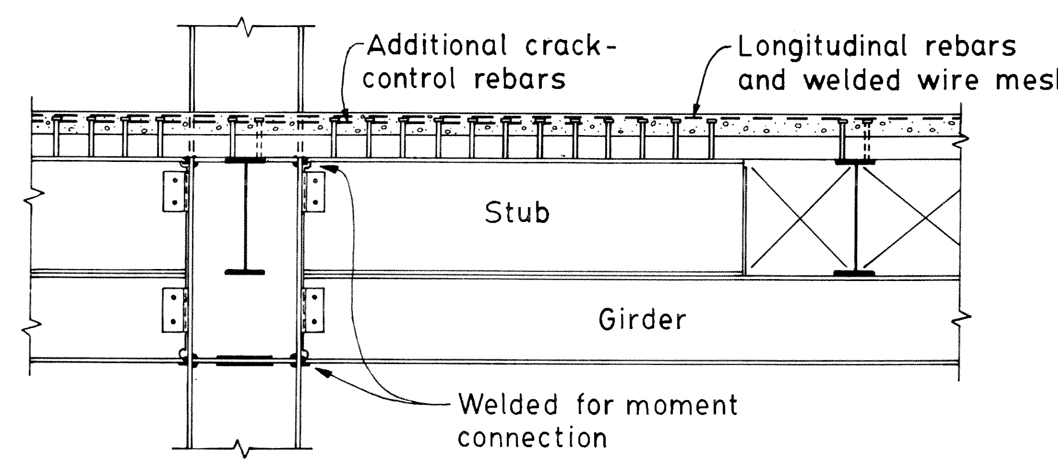


Figure 6.23
Typical Moment Joint (for Large End Moments)

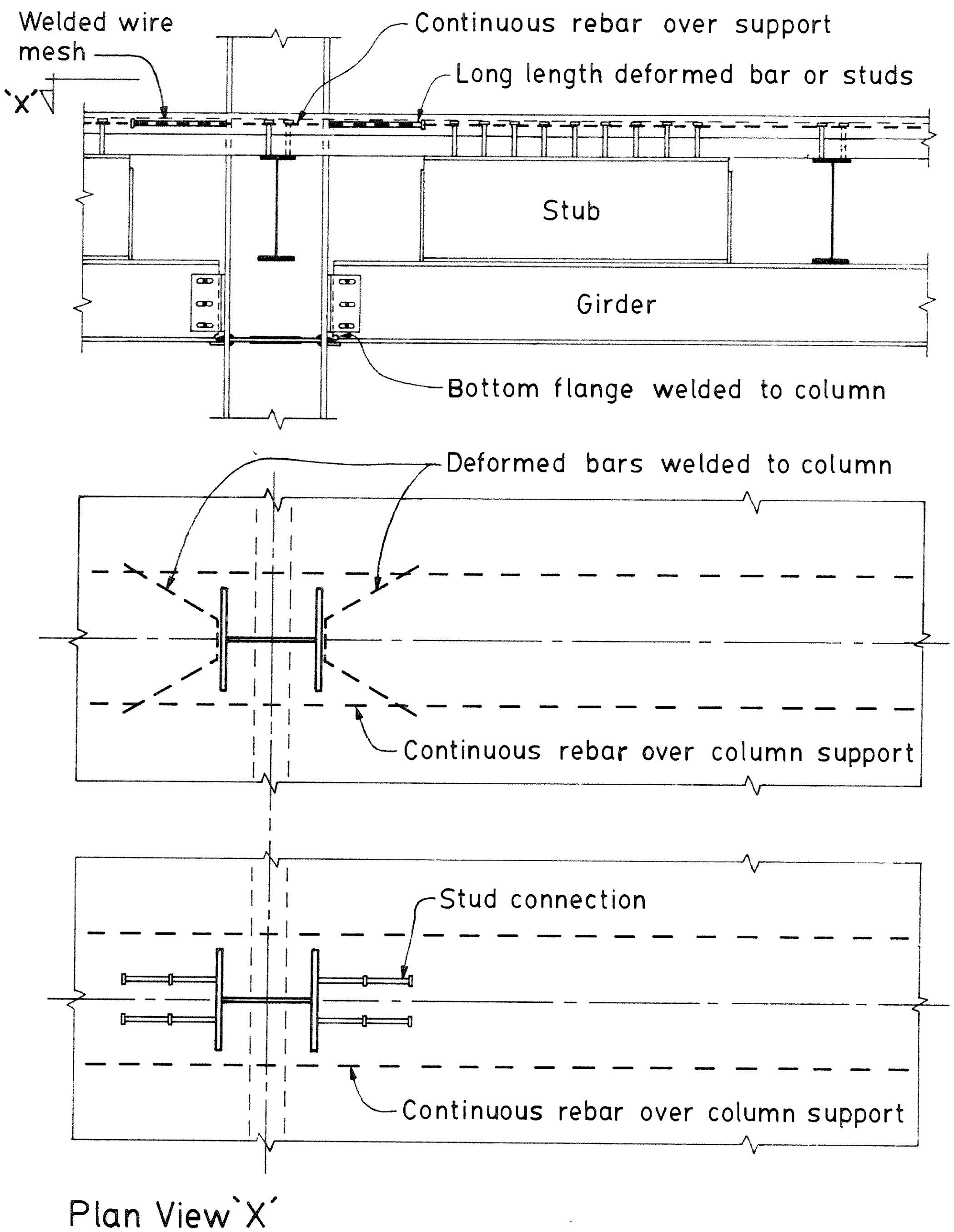


Figure 6.24
Typical Moment Joint
(for Moderate End Moments)

Rigid-Ended Stub-Girders for Participation in Lateral Load Resistance

The deep effective girder depth of a stub-girder provides a large moment of inertia which is effective for inter-storey drift control in a rigid framework, and a number of lowrise stub-girder framed buildings have been built with stub-girders connected to columns to take advantage of this capacity. The most obvious problem during the design of such a building frame involves the moment transfer at the column-girder interface. Figure 6.23 illustrates the extension of an exterior stub to a column face to provide a large moment connection. The girder system must be checked along its full length for critical load combinations under gravity and lateral forces. Bottom chord lateral bracing may also be required. This system will not likely be satisfactory beyond 5 to 10 storeys because moments due to lateral forces may override the gravity designed girders, reducing their efficiency to the point where other systems should be considered.

When wind moments are small, details such as shown in Fig. 6.24 may be used. In this case, the deck-slab is attached to the column either by long deformed bars or shear studs welded to the steel column. In addition, the bottom flange of the stub-girder is field welded or bolted to the column face to complete the moment transfer.

6.19 FLOOR DESIGN EXAMPLE

The following example illustrates the design of some typical members in a stub-girder floor layout. One line of composite Gerber beams will be selected using design tables provided in Chapter 4. In addition, a typical stub-girder will be analysed manually as well as by a computer. Detailed design checks, required for various critical locations in the stub-girder assembly, will also be illustrated.

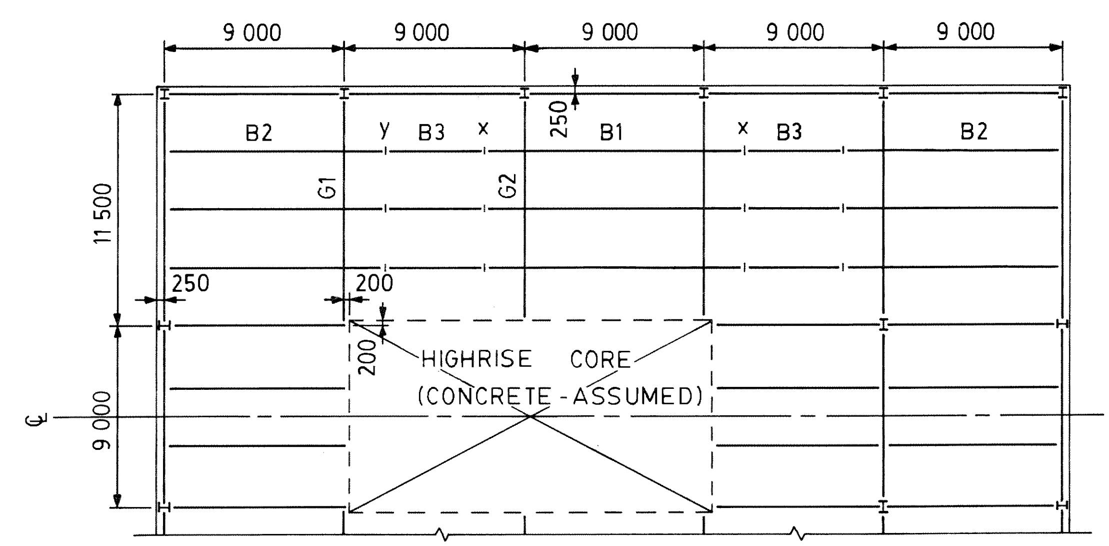


Figure 6.E1
Floor Design Example Key Plan
(Stub-Girder Floor)

The half floor plan of a typical floor in a multistorey office building is shown in Fig. 6.E1. Select Gerber beams B1, B2, B3 and stub-girder G1 using loadings and design criteria given in the floor design example 4.14, with exceptions as listed below:

Storey heights given:
floor to floor height = 3 560 mm

floor to ceiling height	=	2 590 mm
plenum depth	=	970 mm
maximum longitudinal duct depth	=	370 mm
(see Fig. 6.E2)		

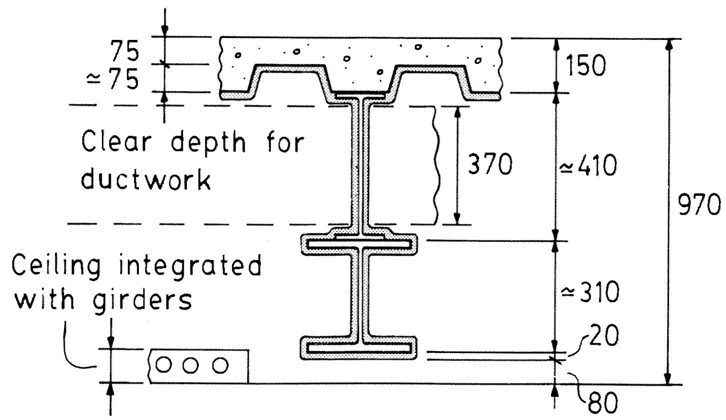


Figure 6.E2
Plenum Depth Computation

Materials: Same as specified in section 4.14 except concrete strength, f'_c = 25 MPa.

Solution

A 76 mm deep wide-rib profile composite steel deck is selected to span approximately 3 metres using the same criteria shown in Section 4.14. In this example, 76 mm wide-rib composite steel deck of 0.91 mm nominal thickness and a cover slab thickness of 75 mm satisfy all deck-slab design criteria; and the design parameters as listed in the manufacturer's catalogue are as follows:

Load due to steel deck, q_d	=	0.10 kPa
Moment of inertia of deck, I_d	=	1.10×10^6 mm^4/m
Dead load due to deck-slab, q	=	2.60 kPa

Composite Gerber beams B1, B2, B3

It should be pointed out that ultimate strength design of beams B1 and B2 for positive moment effects (as illustrated below) may seem conservative, since no account is taken of the negative moment relieving effect caused by the overhanging cantilevers. However it does provide a more convenient method of calculation. In addition, this design approach permits possible future alterations of floor framing in suspended span areas, independent of adjacent floor bay framing.

Trial Member Selection, Beams B1, B2

– live load:

Tributary area,	$A = 11.5(9.0)/4$	= 25.9 m^2
Reduction factor,	$RF_2 = 0.3 + \sqrt{9.8/25.9}$	= 0.92
Total live load for beam,	$W_L = 0.92(2.4)(25.9)$	= 57.2 kN

– Fresh-concrete condition dead load including concrete ponding:

$w = (1 + 0.2\, w_c\, s^4/I_d)\, s\, q$ (Table 3.1, triple span)
$= (1 + 0.2\,(2\,300)(3)^4/1.10 \times 10^6)\, s\, q$
$= 1.034\,(3)(2.6)$
$= 8.07$ kN/m (Note, conservative deck span of 3 000 mm is used)

Total fresh-concrete condition load support to support,
$W_c = (8.07 + 0.4)(9.0) = 76.2$ kN (Beam steel assumed 0.4 kN/m)

– Superimposed dead loads between supports:

$W_p = 1.2\,(25.9) = 31.1$ kN
$W_{OD} = (0.5 + 0.2)(25.9) = 18.1$ kN

– Factored maximum positive moment, M_f

$W_f = 1.25\,(76.2 + 31.1 + 18.1) + 1.5\,(57.2) = 243$ kN
$M_f = 243 \times 9/8 = 273$ kN·m
$V_f = 243/2 = 122$ kN

– Composite factored moment resistance of beams B1 and B2, using trial section **W410×39**

$t_o = t_c + t_d = 75 + 76 = 151$ mm

$16\, t_o + b = 16(151) + 140 = 2\,556$ mm
$L/4 = 9\,000/4 = 2\,250$ mm (governs)
beam spacing $= 11\,500/4 = 2\,875$ mm

Therefore effective slab width, $b_1 = 2\,250$ mm

From Table 4.6 , for W410×39,
M_{rc} for 50% shear connection may be interpolated,
$M_{rc50\%} = 360$ kN·m > 273 kN·m OK
$Q_{rc50\%} = 1\,350 \times 50\% = 675$ kN $q_r = 87.8$ kN using 19 mm studs (Table 2.1)

Therefore use 16 studs per beam (B1 and B2)

– Unshored beam requirement

Moment due to specified fresh-concrete condition load acting on bare steel beam,

$M_b = W_c\, L/8 = 76.2\,(9.0)/8 = 85.7$ kN·m

Moment due to all specified superimposed loads acting on composite beam
(i.e. after concrete attained 75% of f'_c),
$M_t = (W_L + W_p + W_{OD})\, L/8 = (52.7 + 31.1 + 18.1)(9.0)/8.0 = 115$ kN·m

From Table 4.6, $S_x = 0.634 \times 10^6$ mm^3, $S_t = 1.17 \times 10^6$ mm^3 by interpolation.

Combined stresses in bottom flange under specified loads become,

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{85.7}{0.634} + \frac{115}{1.17} = 233 \text{ MPa} < 0.9\, F_y$$

Therefore shoring is not required.

Before we proceed to design check beams B1 and B2, cantilever lengths (and hence span of suspended segment B3) must be determined.

– Compute Cantilever Lengths

Using the trial sections of beams B1 and B2 as W410×39, and using the maximum factored cantilever moment resistance under a fully laterally supported condition, the suspended floor bay framing may be modelled as shown in Figure 6.E3.

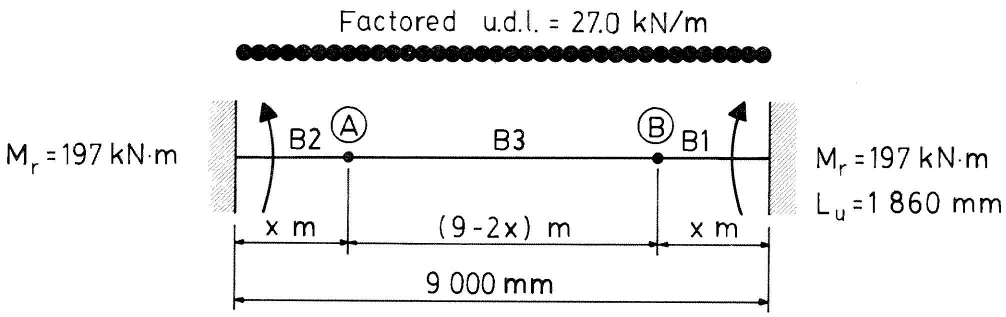


Figure 6.E3
Simplified Structural Model for Cantilever Length Computation

Factored total u.d.l. for the 9 metre framing

$= [1.25\ (W_c + W_p + W_{OD}) + 1.5\ W_L]/9$
$= [1.25\ (76.2 + 31.1 + 18.1) + 1.5\ (57.2)\,]/9$
$= 27.0$ kN/m

Factored end shear at reaction A,

$V_f = 27.0\ (9 - 2x\ )\ /\ 2 = 121.5 - 27.0x$ kN
where x = cantilever lengths in metres

Cantilever end moment due to V_f calculated above

$= (121.5 - 27.0x\ )\ x$
$= 121.5x - 27.0x^2$ kN·m

Cantilever end moment due to factored u.d.l. acting directly on the overhanging segment

$= 27.0x^2/\ 2 = 13.5x^2$ kN·m

Equating cantilever-end factored applied moment to factored moment resistance,

$197 = 121.5x - 27.0x^2 + 13.5x^2$

and simplifying, we obtain,

$13.5x^2 - 121.5x + 197 = 0$

Solving for value of x,

$$x = \frac{121.5 - \sqrt{(-121.5)^2 - 4(13.5)(197)}}{2(13.5)}$$

$x = 2.12$ m $> L_u$ (= 1 860 mm)

Use x value = 1 850 mm. Therefore the length of beam B3 can be conveniently calculated as 5 300 mm.

– Compute factored cantilever moment using "hinge" locations determined above.

$M_f = 121.5x - 27.0x^2 + 13.5x^2$
$= 179$ kN·m $< M_r$ (= 197 kN·m)

– Deflection estimates – beam B1:

a) Camber requirement
Deflection of unshored beam under fresh-concrete condition load,
Δ_c = deflection due to loading between supports minus the deflection due to cantilever end moments caused by loading on cantilever overhangs.

$$= \frac{5W_cL^3}{384EI_x} - \frac{M_cL^2}{8EI_x}$$

where $M_c = \left(\frac{76.2}{9}\right)\left(\frac{5.3}{2}\right)(1.85) + \left(\frac{76.2}{9}\right)\left(\frac{1.85^2}{2}\right) = 56.0$ kN·m;

$W_c = 76.2$ kN

$$\Delta_c = \frac{5}{384}\frac{(76.2)(9)^3}{(200)(127)} \times 10^3 - \frac{(56)(9)^2}{8(200)(127)} \times 10^3$$

$= 6.2$ mm Therefore **camber not required.**

b) Shrinkage deflection and creep deflection
Since beam B1 is continuous over stub-girders, the amount of shrinkage deflection is minimal. Also creep deflection is found to be not critical due to the fact that beam B1 does not require shoring. Use the following deflection calculations.

c) Deflection of composite beam due to live load and partition including long term effects

Total loading support to support $= W_L + W_p = 57.2 + 31.1 = 88.3$ kN

$$\Delta = \frac{5}{384}\frac{88.3\ (9)^3\ 10^3}{(200)\ I_e}(1.15)$$

$= 11.7$ mm $<$ L/300 OK

where $I_e = I_s + 0.85\ (p)^{0.25}\ (I_t - I_s)$
$= [127 + 0.85\ (0.5)^{0.25}\ (527 - 127)]$
$= 413\ (\ \times 10^6\ \text{mm}^4)$

Note: I_t for W410×39 (composite section, b_1 = 2 250 mm)= 527×10^6 mm^4

– Deflection estimate – beam B2:

a) Camber requirement

$$\Delta_c \simeq \frac{5}{384}\frac{W_cL^3}{EI_x} - \frac{M_cL^2}{16EI_x}$$

$$= \left\{ \frac{5}{384} \frac{(76.2)(9)^3}{(200)(127)} - \frac{(56.0)(9^2)}{16(200)(127)} \right\} 10^3$$

$= 17.3$ mm < 20 mm

Therefore **camber not required**.

b) Shrinkage and creep deflections
Detailed calculation ignored for same reason as given for B1.

c) Deflection of composite beam due to live and partition loads including long term effects

Total loading support to support $= W_L + W_p = 88.3$ kN

$$\Delta = \frac{5}{384} \frac{(88.3)(9)^3}{(200)(413)} 10^3 \times 1.15$$

$= 11.7$ mm $< L/300$ OK

Final size selected for beams B1 and B2 and other construction details:

B1	length	= 9 000 + 2(1 850) = 12 700 mm
	section	= W410×39
	studs*	= 24 − 19 mm diameter (per beam)
	camber	= None
B2	length	= 9 000 + 1 850 = 10 850 mm
	section	= W410×39
	studs*	= 22 − 19 mm diameter (per beam)
	camber	= None

*including 2 studs at each beam to girder joint, and 2 studs at each cantilever end.

Trial Member Selection, B3

Span = 5 300 mm

– live load:

$$A = \frac{11.5}{4}(5.3) = 15.2 \text{ m}^2 < 20 \text{ m}^2, RF_2 = 1.0$$

Total live load per beam, $W_L = 1.0\,(2.4)(15.2) = 36.5$ kN

– Dead loads:

$W_c = (8.07 + 0.3)(5.3) = 44.4$ kN Assuming beam steel = 0.3 kN/m
$W_p = 1.2(15.2) = 18.2$ kN
$W_{OD} = 0.7(15.2) = 10.6$ kN

– Factored maximum positive moment, M_f

$W_f = 1.25(44.4 + 18.2 + 10.6) + 1.5(36.5) = 146$ kN

$M_f = 146 \times 5.3/8 = 96.7$ kN·m
$V_f = 146/2 = 73$ kN

– Composite factored moment resistance of beam B3 using trial section **W200×27**

$16\ t_o + b$	= 16(151)+133	= 2 549 mm
L/4	= 5 300/4	= 1 325 mm (governs b_1)
beam spacing	= 11 500/4	= 2 875 mm

From Table 4.6 , for W200×27, by interpolation,
$M_{rc50\%} = 156$ kN·m > 96.7 kN·m

$$Q_{r50\%} = \frac{915}{2} = 458 \text{ kN} \qquad q_r = 87.8 \text{ kN}$$

Therefore use **12 studs** per beam. (19 mm diameter)

– Unshored beam requirement

Moment due to specified fresh-concrete condition load action on bare steel beam,
$M_b = W_cL/8 = 44.4\,(5.3)/8 = 29.4$ kN·m

Moment due to all specified superimposed loads acting on composite beam (i.e after concrete attained 75% of f'_c),
$M_t = (W_L + W_p + W_{OD})L/8 = (36.5+18.2+10.6)(5.3)/8 = 43.3$ kN·m

From Table 4.6, $S_x = 0.249\times10^6$ mm³ ; $S_t = 0.570\times10^6$ mm³ by interpolation.
Combined stresses in bottom flange under specified loads become,

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{29.4}{0.249} + \frac{43.3}{0.57} = 194 \text{ MPa} < 0.9\,F_y$$

Therefore shoring is not required.

– Deflection estimates:

a) Camber requirement

$$\Delta_c = \frac{5\,W_cL^3}{384EI_x} = \frac{5}{384}\frac{(44.4)(5.3)^3}{(200)(25.8)} \times 10^3$$

$= 16.7$ mm (between beam supports)

Deflection at cantilever ends (x) of beam B1, due to u.d.l. on B1, (see Figure 6.E4a),

$$\Delta_{x1} = -\frac{wL^3N}{24\,EI_x}\left(1 - 6\frac{N^2}{L^2} - 3\frac{N^3}{L^3}\right)$$

$$= -\frac{8.37(9)^3(1.85)}{24(200)(127)}\left[1 - \frac{6(1.85)^2}{9^2} - \frac{3(1.85)^3}{9^3}\right] \times 10^3$$

$= -\ 13.3$ mm (-ve sign means upward deflection)

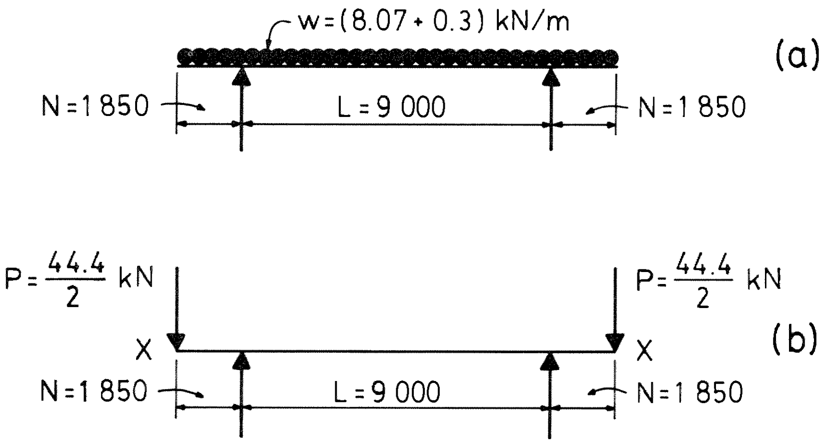


Figure 6.E4
Cantilever Segment – Beam 'B1'

Deflection at cantilever ends (x) of beam B1, due to point load at cantilever ends, (see Figure 6.E4b),

$$\Delta_{x2} = \frac{PN^2}{EI}\left(\frac{L}{2} + \frac{N}{3}\right)$$

$$= \frac{(22.2)(1.85)^2}{(200)(127)}\left(\frac{9}{2} + \frac{1.85}{3}\right) \times 10^3$$

$$= 15.3 \text{ mm} \quad \text{(downward)}$$

Therefore total downward support deflection at x,
$\Delta_x = \Delta_{x1} + \Delta_{x2} = 15.3 - 13.3 = 2$ mm

Deflection at cantilever end y of beam B2, due to u.d.l. on B2, (See Figure 6.E5a),

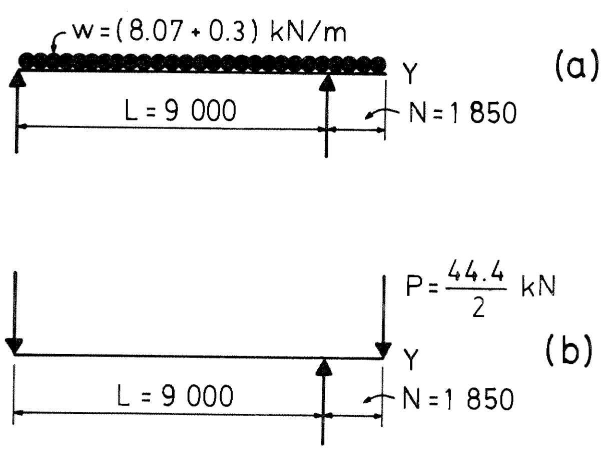


Figure 6.E5
Cantilever Segment – Beam 'B2'

$$\Delta_{y1} = \frac{wL^3N}{24EI}\left[3\frac{N^3}{L^3} + 4\frac{N^2}{L^2} - 1\right]$$

$$= \frac{8.37(9)^3(1.85)}{24(200)(127)}\left[\frac{3(1.85)^3}{9^3} + \frac{4(1.85)^2}{9^2} - 1\right] \times 10^3$$

$$= -14.9 \text{ mm} \quad \text{(-ve sign means upward)}$$

Deflection at cantilever end y of beam B2, due to point load on B2, (See Figure 6.E5b),

$$\Delta_{y2} = \frac{PN^2}{3EI}\left(N + L\right)$$

$$= \frac{22.2\,(1.85)^2}{3(200)(127)}\left(1.85 + 9\right) \times 10^3$$

$$= 10.8 \text{ mm} \quad \text{(downward)}$$

Total downward support deflection at y,
$\Delta_y = \Delta_{y1} + \Delta_{y2} = -14.9 + 10.8$

$$= -4.1 \text{ mm} \quad \text{(upward)}$$

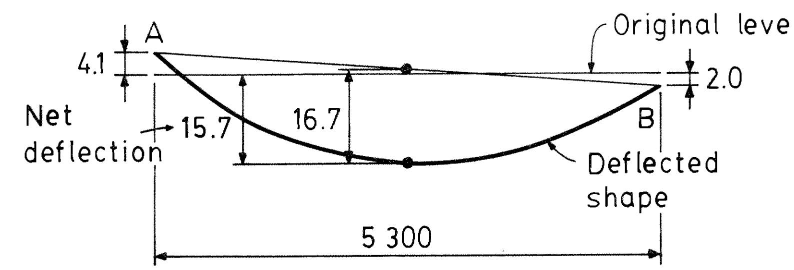


Figure 6.E6
Deflection of Suspended Segment
Beam 'B3' (under Fresh-Concrete Loading)

From Figure 6.E6, net deflection at midspan of beam B3 under fresh-concrete condition load, taking account of support movements,

$$\Delta_{c1} = 16.7 + (-4.1 + 2.0)/2 = 15.7 \text{ mm}$$

Camber not required. (Amount too small to achieve accuracy, and dead load accumulation due to level screeding of concrete will have minimal impact on total cantilever system.)

b) Deflection due to live load and partition including long term effects using composite section for B3 and steel section only for beams B1 and B2 (conservatively),

$$\Delta \simeq \Delta_c \left(\frac{I_x}{I_e}\right)\left(\frac{W_L + W_P}{W_c}\right)(1.15) + \left(\frac{\Delta_x + \Delta_y}{2}\right)\left(\frac{W_L + W_P}{W_c}\right)$$

$$= (16.7)\left(\frac{25.8}{I_e}\right)\left(\frac{36.5 + 18.2}{44.4}\right)(1.15) + \left(\frac{2 - 4.1}{2}\right)\left(\frac{36.5 + 18.2}{44.4}\right)$$

$$= 3.8 \text{ mm} < (9\,000/300) \qquad \text{OK.}$$

where $I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$
$= [25.8 + 0.85\,(0.5)^{0.25}\,(158 - 25.8)]$
$= 120\ (\times 10^6\ \text{mm}^4)$

Final size selected for beams B3 and other details:

B3 length = 5 300 mm
section = W200×27
studs = 12 studs per beam (19 mm diameter)
camber = None

Notes for Design Checks on Beams B1, B2, and B3:

a) V_f of beams B1, B2, B3 are checked with V_r of selected sections and are found to be satisfactory (see illustrated example in Chapter 4).

b) Web crippling of beams B1 and B2 at girder supports due to shoring forces is to be design checked following the analysis of maximum shoring forces. See section 6.17.

c) Additional stud connectors are to be installed for Beams B1 and B2 at beam/girder joints and at the free ends of cantilevers. See Figure 6.8.

d) Safety at construction stages such as deck placement and concrete placement is assumed to be checked using loadings as illustrated in Chapter 4 (also see example calculation in section 4.14).

Stub Girders G1 (and G2, similar)

– Live load:

Tributary area of member B2 carried by girder G1 is calculated in Fig. 6.E7, and is equal to 28.0 m².

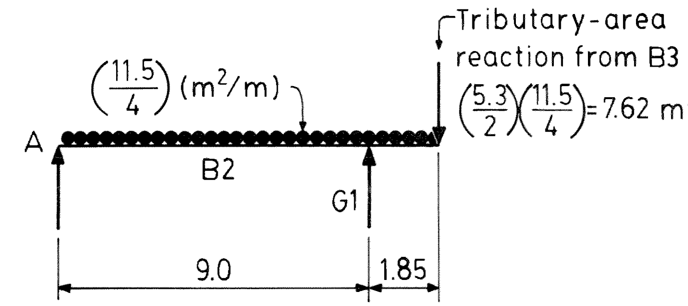
$A = 28 \times 3 = 84\ \text{m}^2$
$RF_2 = 0.3 + \sqrt{9.8/84} = 0.64$
$P_L = 0.64\,(2.4)(28.0) = 43.0\ \text{kN}$

– Dead Load:

$$P_c = \left(\frac{8.07 + 0.4}{3}\right)(28.0) + \left(\frac{11.5}{4}\right)(1.0)$$

$= 81.9$ kN   Assuming girder steel = 1 kN/m

$P_p = 1.2\,(28.0) = 33.6\ \text{kN}$
$P_{OD} = (0.5 + 0.2)\,(28.0) = 19.6\ \text{kN}$



Taking moment about A, the tributary-area reaction at G1

$$= \left\{\left(\frac{11.5}{4}\right)(9 + 1.85)^2/2 + 7.62\,(9 + 1.85)\right\}/9$$

$= 28\ \text{m}^2$

Figure 6.E7
Tributary Area for Reaction at Point 'G1'

– Factored maximum positive moment, M_f

$P_f = 1.25(81.9 + 33.6 + 19.6) + 1.5(43) = 233\ \text{kN}$

$$M_f = 233\left(\frac{11.5}{2}\right) = 1\,340\ \text{kN·m}$$

$$V_f = 233\left(\frac{3}{2}\right) = 350\ \text{kN}$$

– Elastic properties of deck-slab system (top chord)

Using **W410×39 stubs,** $b = 140$ mm

$16t_o + b = 16(151) + 140$ = 2 556 mm, (governs b_l)
girder spacing = 9 000 mm
$L/4 = 11\,500/4$ = 2 875 mm

Figure 6.E8 shows the cross section of the deck-slab system within the effective width b_l.

Longitudinal reinforcing bars of size 15M are assumed. Five 15M bars near the top surface, and five 15M bars near the bottom of deck-slab, are placed in continuous lengths for the span of the girder.

In addition, two layers of welded wire mesh of size 152×152 MW18.7×MW18.7 are assumed to be placed near the top of the slab.

$E_c = w_c^{1.5} \times 0.043\,\sqrt{f'_c}$
$= (2\,300)^{1.5}\,(0.043)\,\sqrt{25} = 23\,700\ \text{MPa}$
$n = E/E_c = 200\,000/23\,700 = 8.44$

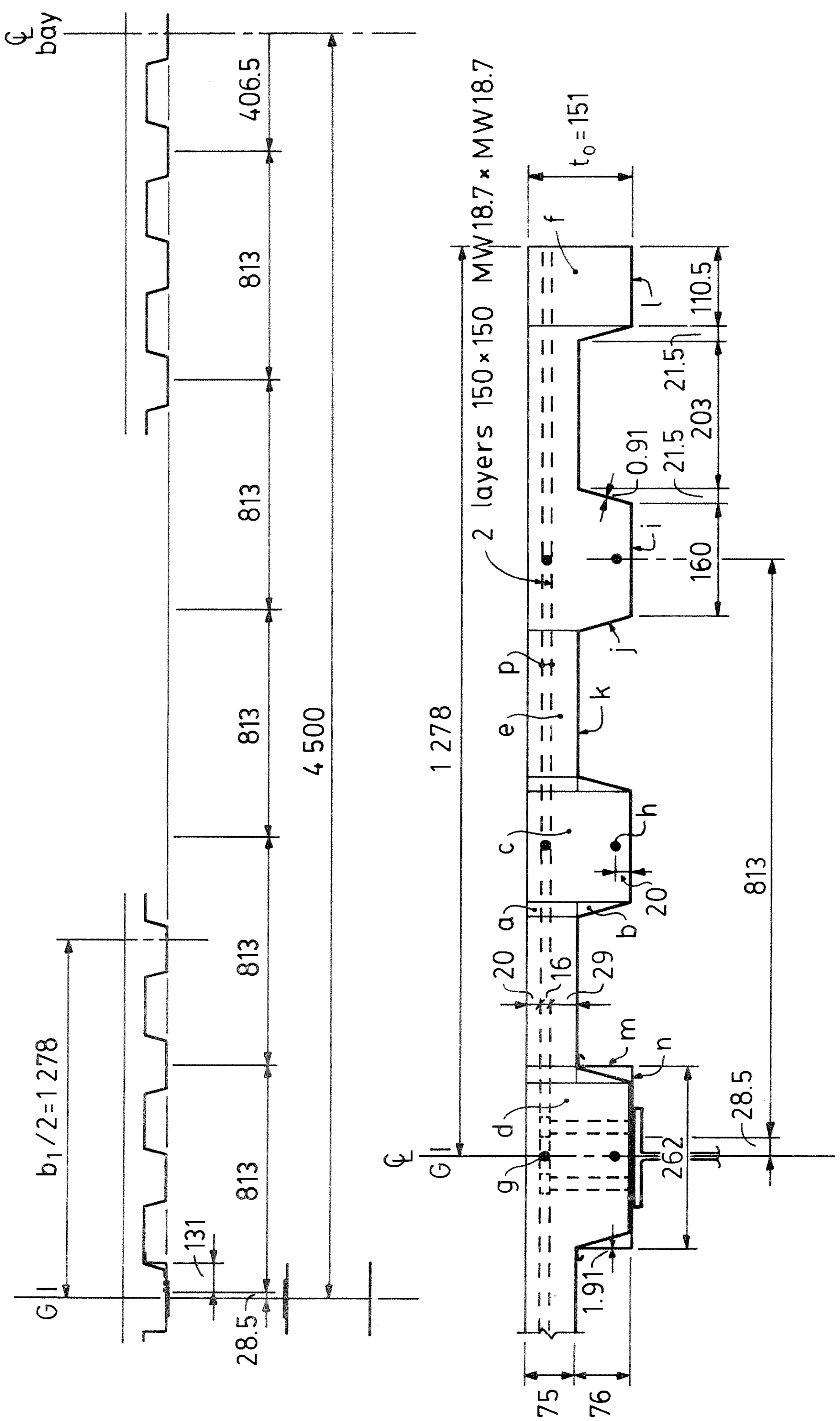


Figure 6.E8
Cross Section of Reinforced Deck-Slab within Effective Slab Width

The following table is prepared for the calculation of the neutral axis and the moment of inertia of the composite deck-slab (within the design effective width b_l).

Element I/D	No. of pieces	Total transformed area, A (mm^2)	Distance from top of slab, y (mm)	Product A y (mm^3)	Product A y^2 (mm^4)	I_{local} in steel unit (mm^4)
a	12	2 293	37.5	85 988	3.2×10^6	1.1×10^6
b	12	1 162	100.3	116 549	11.7×10^6	0.3×10^6
c	4	11 450	75.5	864 475	65.3×10^6	21.8×10^6
d	1	3 918	75.5	295 809	22.3×10^6	7.4×10^6
e	6	10 823	37.5	405 863	15.2×10^6	5.1×10^6
f	2	3 954	75.5	298 527	22.5×10^6	7.5×10^6
g	5	1 000	28	28 000	0.8×10^6	
h	5	1 000	123	123 000	15.1×10^6	
i	4	582	151	87 882	13.3×10^6	
j	12	862	113	97 406	11.0×10^6	0.4×10^6
k	6	1 108	75	83 100	6.2×10^6	
l	2	201	151	30 351	4.6×10^6	
m	2	290	113	32 770	3.7×10^6	0.1×10^6
n	1	500	151	75 500	11.4×10^6	
p	2	$\simeq$ 560	28	15 680	0.4×10^6	
TOTAL		39 703		2 640 900	206.7×10^6	43.7×10^6

$\bar{y}$ (Neutral axis) $= \Sigma$ (A y) / Σ A $= 66.5$ mm (from top of slab)

I_t = Transformed moment of inertia (steel unit)
$= \Sigma I_{local} + \Sigma (A\ y^2) - y^2 \Sigma A$
$= (43.7 + 206.7) \times 10^6 - (66.5)^2(39\ 703) = 74.8\times10^6\ mm^4$

Note: value of I_t represents the uncracked deck-slab top chord.

– Approximate girder size selection

Assuming depth of girder section used $\simeq$ 310 mm. Estimated effective depth of girder section, as illustrated in Figure 6.E9, is shown as 638 mm.

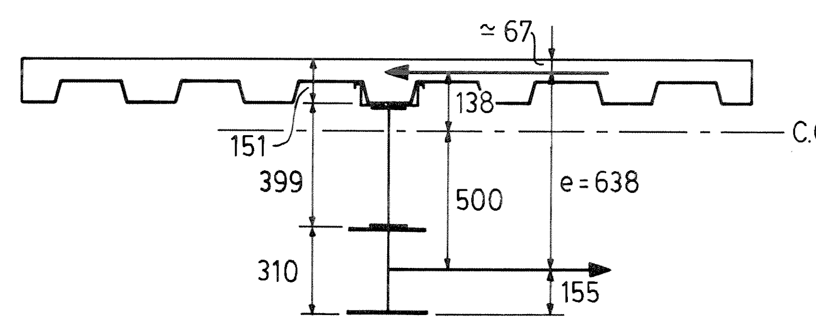


Figure 6.E9
Approximate Lever Arm Length for Bottom Chord Force Computation

Maximum bottom chord factored axial tension $\simeq M_f/e = 1\ 340(10^3)/638 = 2\ 100$ kN

Allowing for effects of local bending (say about 35%), factored bottom chord tension $\simeq$ 2 100(1+0.35) is used for trial selection.

$$\text{Estimated bottom chord area} = \frac{2\ 100(1+0.35)}{0.9\ F_y}$$

$$= \frac{2\ 100(1+0.35)}{0.9(300)} \times 10^3$$

$$= 10\ 500\ \text{mm}^2$$

Check trial section **W310×86**. $A_s = 11\ 000$ mm²
$I_x = 199\times10^6$ mm⁴ $Z_x = 1\ 420\times10^3$ mm³ $d = 310$ mm
$b = 254$ mm $t = 16.3$ mm $w = 9.1$ mm
(from Handbook of Steel Construction - 1984, p.6-48)

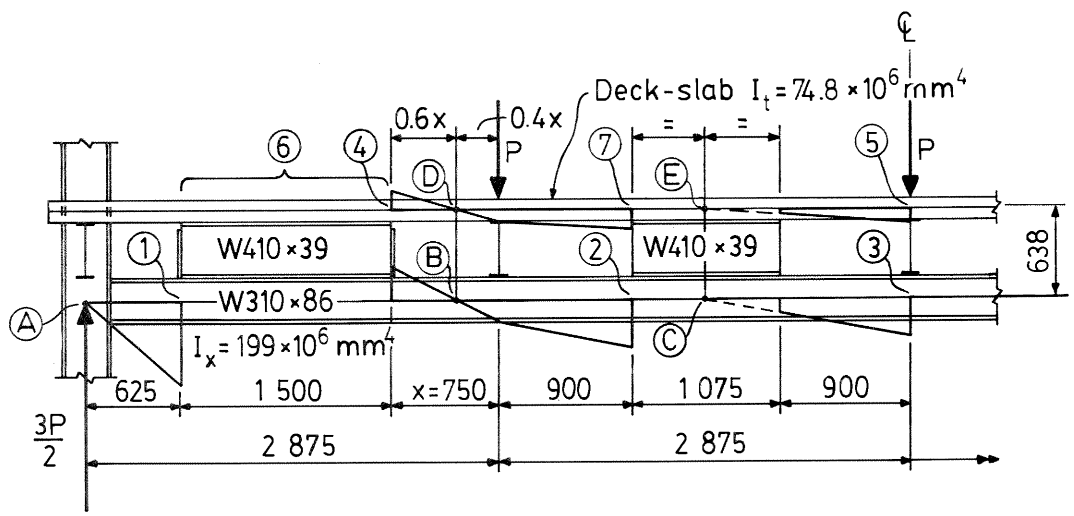


Figure 6.E10
Simplified Vierendeel Girder Model

– Approximate 'statically-determinate' model analysis

Figure 6.E10 illustrates the vierendeel girder model of the stub-girder similar to the girder shown in Figure 6.9.

Moment of inertia of top chord $= 74.8\times10^6$ mm⁴ (or 27%)
Moment of inertia of bottom chord $= 199\times10^6$ mm⁴ (or 73%)

Shear at location	A = $3P_f/2$	= 350	kN
	B = $(3P_f/2)(0.73)$	= 255	kN
	C = $(P_f/2)(0.73)$	= 85	kN
	D = $(3P_f/2)(0.27)$	= 94.4	kN
	E = $(P_f/2)(0.27)$	= 31.5	kN
	where $P_f = 233$ kN		

Bending moment at points:

1	350 (0.625)	= 219 kN·m
2	255 (0.4)(0.75)+85(0.9)	= 153 kN·m
3	85(1.075/2+0.9)	= 122 kN·m
4	94.4 (0.6)(0.75)	= 42.5 kN·m
5	31.5(1.075/2+0.9)	= 45.3 kN·m
7	94.4(0.4)(0.75)+31.5(0.9)	= 56.7 kN·m

Axial forces at points:
B and D $= (3P_f/2)(2.875-0.4\times0.75)/0.638 = 1\ 411$ kN
C and E $= [(3P_f/2)(2.875)(1.5) - P_f(2.875/2)]/0.638 = 1\ 837$ kN

– Design checks to critical locations of stub-girder model

Location 1

Check bending in steel member:
Factored bending moment, $M_f = 219$ kN·m
Factored moment resistance,
M_r for unbraced length of 625 mm = 383 kN·m ($L_u = 4\ 250$ mm)

Therefore, utilization of moment resistance can be calculated as

$(M_f/M_r) = 57\%$ OK, not greater than 100%

Check shear in steel member:
Factored shear force, $V_f = 350$ kN
Factored shear resistance, $V_r = 503$ kN

$$\text{Therefore, utilization of shear resistance} = \frac{V_f}{V_r} = 70\%$$

OK, not greater than 100%

Location 2

Check combined bending and axial tension of steel member:

Factored bending moment,	M_f	= 153 kN·m
Factored tension,	T_f	= 1 411 kN
Factored moment resistance,	M_r	= 383 kN·m
Factored tensile resistance,	T_r	$= \phi F_y A_s$
		$= 0.9(300)(11\ 000)/10^3$
		= 2 970 kN

Therefore, utilization of combined moment and tensile resistance,

$$\frac{M_f}{M_r} + \frac{T_f}{T_r} = 87\%$$ OK, not greater than 100%

Location 3

Check combined bending and axial tension of steel member:

$M_f = 122$ kN·m $M_r = 383$ kN·m

$T_f = 1\ 837$ kN $\qquad T_r = 2\ 970$ kN

Utilization of combined moment and tensile resistance,

$$\frac{M_f}{M_r} + \frac{T_f}{T_r} = 94\% \qquad \text{OK, not greater than } 100\%$$

Locations 4, 5 and 7

In computing the top chord's resistance to combined compression and bending, it is assumed that shear bond capacity is exceeded in the ultimate state and thus no steel deck contribution can be credited. The ribbed concrete slab reinforced by top and bottom re-bars must then resist the combined forces at each critical location along the top chord. $\phi_c = 0.60$ is suggested[6.19] for members with nominal axial compression greater than the balanced nominal load of the reinforced concrete section. This condition usually applies to a stub-girder top chord since the eccentricity (i.e. the ratio of M_f to C_f) is normally quite small. A design using this ϕ_c value also satisfies the CAN3-A23.3-M77 Code as

$$0.60 \leq 0.70\ \frac{1.25\ \text{D.L} + 1.5\ \text{L.L}}{1.4\ \text{D.L} + 1.7\ \text{L.L}} \text{ for all positive ratios of } \frac{\text{L.L}}{\text{D.L}}$$

Location 7

Check combined axial compression and positive bending

$M_f = 56.7$ kN·m $\qquad C_f = 1\ 411$ kN

$$e = \frac{M_f}{C_f} = 40.2 \text{ mm}$$

$$r = \sqrt{\frac{74.8 \times 10^6}{39\ 703}} = 43.4 \text{ mm}$$

$$\frac{Kl}{r} = \frac{1.0(900)}{43.4} = 20.7 < 34 - 12(M_1/M_2)$$

where $M_1 = 28.3$ and $M_2 = 56.7$

Hence neglect slenderness effect (Clause 8.12.5.1 of CAN3-A23.3-M77)

Find ultimate resistance by successive approximation

Try a = 75 mm

From Fig. 6.E11(a)

$$\epsilon_s = \frac{0.85d\epsilon_u}{a} - \epsilon_u = \frac{0.85(123)0.003}{75} - 0.003 = 0.00118 < \frac{f_y}{E_s} \text{ of rebar}$$

i.e. $\quad f_s = E_s\, \epsilon_s = 200\ 000\ (0.00118) = 236$ MPa

$$\epsilon_s' = \frac{a - 0.85d'}{a}\,\epsilon_u = \frac{75 - 0.85(28)}{75}(0.003) = 0.00205 > \frac{f_y}{E_s} \text{ of rebar}$$

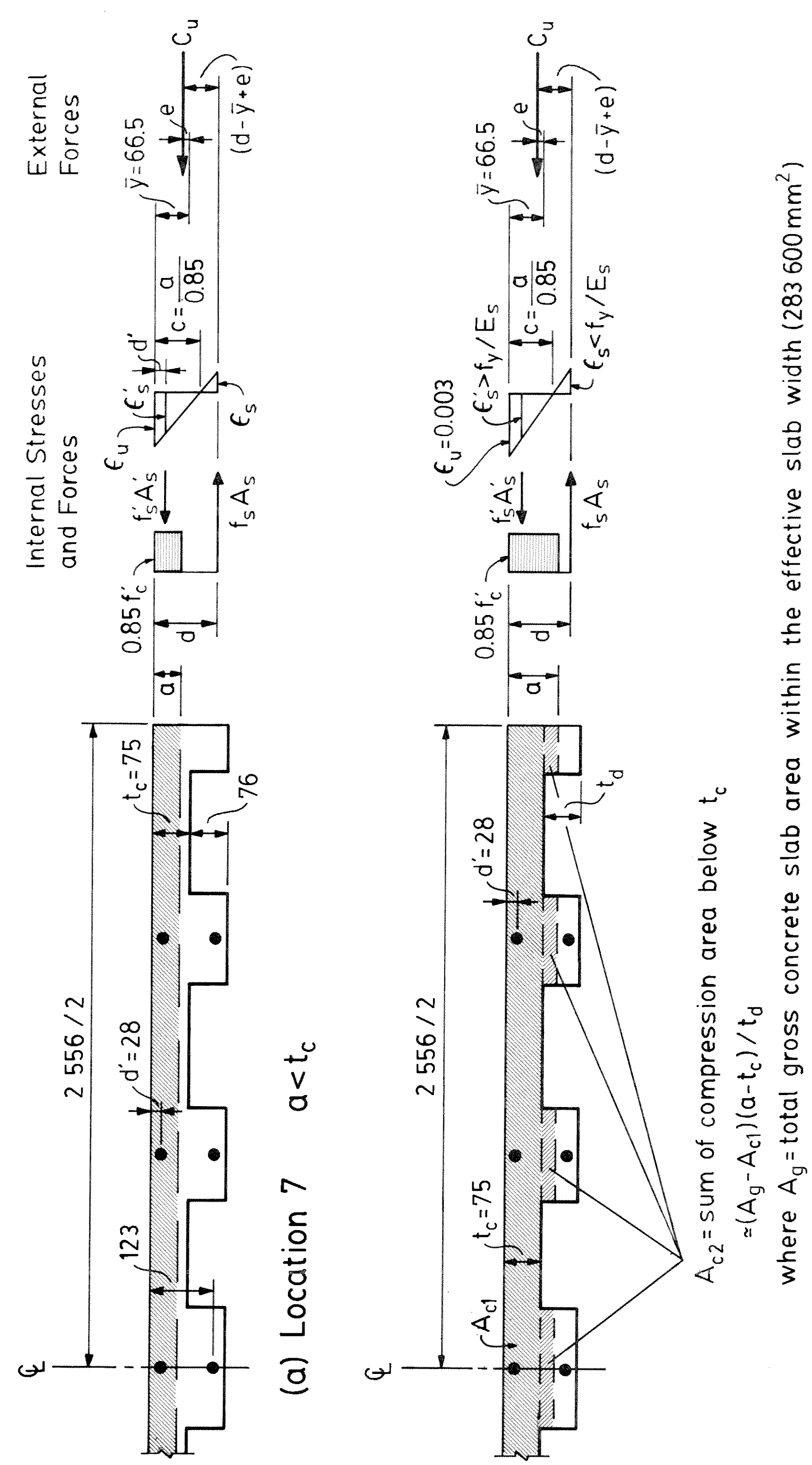


Figure 6.E11
Idealized Top Chord Cross Section Showing Internal Forces and Strains under Combined Compression and Positive Bending

$f_s' = f_y = 400$ MPa (used for rebar)

$$C_u = 0.85\, f_c'\, a\, b_1 + f_y A_s' - f_s A_s$$

$$= 0.85(0.025)(75)(2\,556) + 0.400(1\,000) - 0.236(1\,000)$$

$$= 4\,074 + 400 - 236 = 4\,238 \text{ kN}$$

Taking moment about centroid of tension bars,

$$(e + d - \overline{y})\, C_u = 0.85\, f_c'\, a\, b_1 \left(d - \frac{a}{2}\right) + f_y A_s' (d - d')$$

i.e. $(e + 56.5)4\,238 = 4\,074(85.5) + 400(95)$

Solving for e,

$e = 34.7$ mm < 40.2 mm (Therefore, new 'a' value should be tried)

Try $a = 70$ mm , thus $e = 38.9$ mm < 40.2 mm
Try $a = 69$ mm , thus $e = 39.8$ mm $\simeq 40.2$ mm (accepted)
$C_u = 3\,840$ kN
$f_s = 309$ MPa $< f_y$; therefore $\phi_c = 0.60$
$C_r = \phi_c C_u = 2\,300$ kN
$M_r = e\, C_r = 92.6$ kN·m

$$\text{Utilization ratio} = \frac{C_f}{C_r} = \frac{M_f}{M_r} = \frac{1\,411}{2\,300} = 0.61 < 1.0 \qquad \text{OK}$$

Location 5

Check combined axial compression and bending,

$M_f = 45.3$ kN·m $\qquad C_f = 1\,837$ kN

$$e = \frac{M_f}{C_f} = 24.7 \text{ mm}$$

By successive approximation,
Try $(a = 90) > t_c$

From Fig. 6.E11(b),

$$f_s = \frac{0.85(123)\epsilon_u E_s}{90} - \epsilon_u E_s = 97.0 \text{ MPa (Where } \epsilon_u E_s = 600 \text{ MPa)}$$

$$A_{c1} = 75(2\,556) = 191\,700 \text{ mm}^2$$

$$A_{c2} = \frac{a - t_c}{t_d}(A_g - A_{c1}) = \frac{90 - 75}{76}(283\,600 - 191\,700) = 18\,140 \text{ mm}^2$$

$$C_u = 0.85 f_c' A_{c1} + 0.85 f_c' A_{c2} + f_y A_s' - f_s A_s$$
$$= 4\,074 + 385 + 400 - 0.097(1\,000)$$
$$= 4\,762 \text{ kN}$$

Taking moment about centroid of tension bars,

$$(e + d - \overline{y})C_u = 0.85 f_c' A_{c1}\left(d - \frac{t_c}{2}\right) + 0.85 f_c' A_{c2}\left(d - \frac{t_c}{2} - \frac{a}{2}\right) + f_y A_s'(d - d')$$

$$\text{Thus, } e = \frac{4\,074\,(123 - 37.5) + 385\,(123 - 37.5 - 45) + 400\,(123 - 28)}{4\,762} - 56.5$$

$$= 27.9 \text{ mm} > 24.7 \text{ mm}$$

Try $a = 100$ mm, thus $e = 23.9$ mm < 24.7 mm
Try $a = 98$ mm, thus $e = 24.7$ mm $= 24.7$ mm (value 'a' OK)
$C_u = 5\,020$ mm
$f_s = 40.1$ MPa $< f_y$; therefore $\phi_c = 0.60$

$C_r = 0.6\, C_u = 3\,010$ kN
$M_r = e\, C_r = 74.3$ kN·m

$$\text{Utilization ratio} = \frac{1\,837}{3\,010} = 0.61 < 1.0 \qquad \text{OK}$$

Location 4

Check combined axial compression and bending (negative)

$M_f = 42.5$ kN·m $\qquad C_f = 1\,411$ kN

$$e = \frac{M_f}{C_f} = 30.1 \text{ mm}$$

Try $a = 100$ mm

From Fig. 6.E12,

$$f_s = \frac{0.85(123)600}{100} - 600 = 27.3 \text{ MPa}$$
$$A_{c1} = A_g - b_1 t_c = 283\,600 - 2\,556(75) = 91\,900 \text{ mm}^2$$
$$A_{c2} = b_1(a - t_d) = 2\,556(100 - 76) = 61\,340 \text{ mm}^2$$

$$C_u = 0.85 f_c' A_{c1} + 0.85 f_c' A_{c2} + f_y A_s' - f_s A_s$$
$$= 1\,953 + 1\,303 + 400 - 27.3$$
$$= 3\,629 \text{ kN}$$

Taking moment about centroid of tension bars,

$$(e + \overline{y} - t_o + d)C_u = 0.85 f_c' A_{c1}(d - t_d/2) + 0.85 f_c' A_{c2}\left(d - \frac{t_d}{2} - \frac{a}{2}\right) + f_y A_s'(d - d')$$

$$e = \frac{1\,953(123 - 38) + 1\,304(123 - 38 - 50) + 400(123 - 28)}{3\,629} - 38.5$$

$$= 30.3 \text{ mm} \simeq 30.1 \text{ mm (accepted)}$$

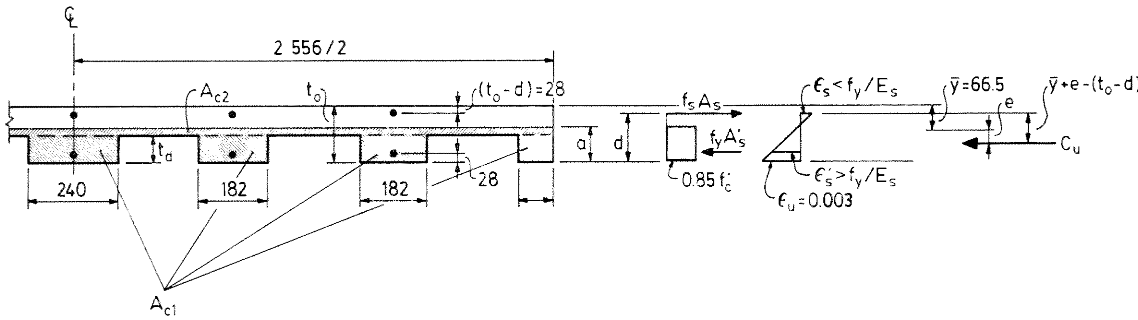


Figure 6.E12
Idealized Top Chord Section Showing
Internal Forces and Strains under
Combined Compression and Negative Bending ($a > t_d$)

$f_s = 27.3$ MPa $< f_y$; therefore $\phi_c = 0.60$
$C_r = 0.6\,(3\,629) = 2\,180$ kN
$M_r = e\,C_r = 65.5$ kN·m

$$\text{Utilization ratio} = \frac{1\,411}{2\,180} = 0.65 < 1.0 \qquad \text{OK}$$

Check shear at Location 4

In the calculation below, steel deck contribution to shear resistance is ignored and the ribbed concrete slab is replaced by a number of tee sections such that each concrete rib becomes a stem of a tee section. Shear resistance can now be computed in accordance with Clause 9.5 of CAN3-A23.3-M77. The total web area, b_wd, (shown as shaded parts in Fig. 6.E13) can be computed:

$$b_wd = 283\,600 - 28(2\,556) - 6(75-28)(203) = 155\,000 \text{ mm}^2$$

The axial compression in the top chord is usually very high and therefore the value of M_m as defined by Eq. (32) of Clause 9.5.4 (CAN3-A23·3-M77) is negative.
Hence Eq. (34) can be used to compute v_c.

$$v_c = 0.3\,\sqrt{f'_c}\,\sqrt{1 + 0.3\,\frac{C_f}{A_g}}$$

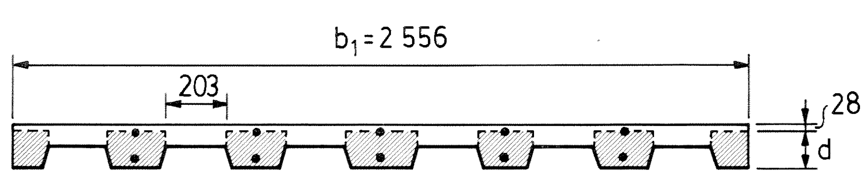


Figure 6.E13
Effective Web Area for Shear Resistance Calculation

$$= 0.3\,\sqrt{25}\,\sqrt{1 + 0.3(1\,411\,000)/283\,600} = 2.37 \text{ MPa}$$

$$V_n = v_cb_wd = 2.37(155) = 367 \text{ kN}$$

$\phi_v = 0.60$ for concrete in shear, as explained at the end of Section 4.9

$$V_r = \phi_v\,V_n = 0.6(367) = 220 \text{ kN}$$
$$V_f = 94.4 \text{ kN} < 220 \text{ kN} \qquad \text{OK}$$

– Stud shear connector design

Studs in Exterior Stub Figure 6.E14 illustrates the interface between the exterior stub and the deck-slab system.

Horizontal factored shear at A-A is represented by the axial force computed for point D and is equal to 1 411 kN.

Factored overturning moment of the deck-slab at level A-A may be calculated as,

$$\frac{1\,411\,(151-67)}{10^3} - 42.5 = 76 \text{ kN·m}$$

The factored shear resistance of a 19 mm shear stud in a 25 MPa-2 300 kg/m^3 concrete slab can be found from Table 2.1 as 87.8 kN, (q_r).

Studs assumed in tension are shown at location 'x' of Fig. 6.E14. The amount of shear cone overlap is indicated by the measure of stud distance to "free" edges. Reference 2.10 is used to determine stud ultimate tension capacity P_{uc}. The P_{uc} values obtained are by means of interpolation after deducting for effects of shear cone overlap.

$$q_t = \phi_{sc}\,(P_{uc})$$
$$\simeq 0.8\,(39.3) = 31 \text{ kN (or 62 kN for 2 studs)}$$

$$\text{Total number of studs for shear action} = \frac{1\,411}{q_r} = 16.1$$

$$\text{Total number of studs for overturning} = \frac{76.0}{31 \times 1.45} = 1.7$$

$$\underline{\text{Total for all effects} = 17.8}$$

Allowing for 50% extra, use **26 studs** for each exterior stub.

Studs in Interior Stub Figure 6.E15 illustrates the interface between the interior stub and the deck-slab system.

Horizontal factored shear at A-A can be calculated as (1 837 – 1 411) = 426 kN.

Factored overturning moment of the deck-slab at level A-A may be calculated as,

$$\frac{426\,(151-67)}{10^3} - 56.7 + 16.9 = -\,4.0 \text{ kN·m}$$

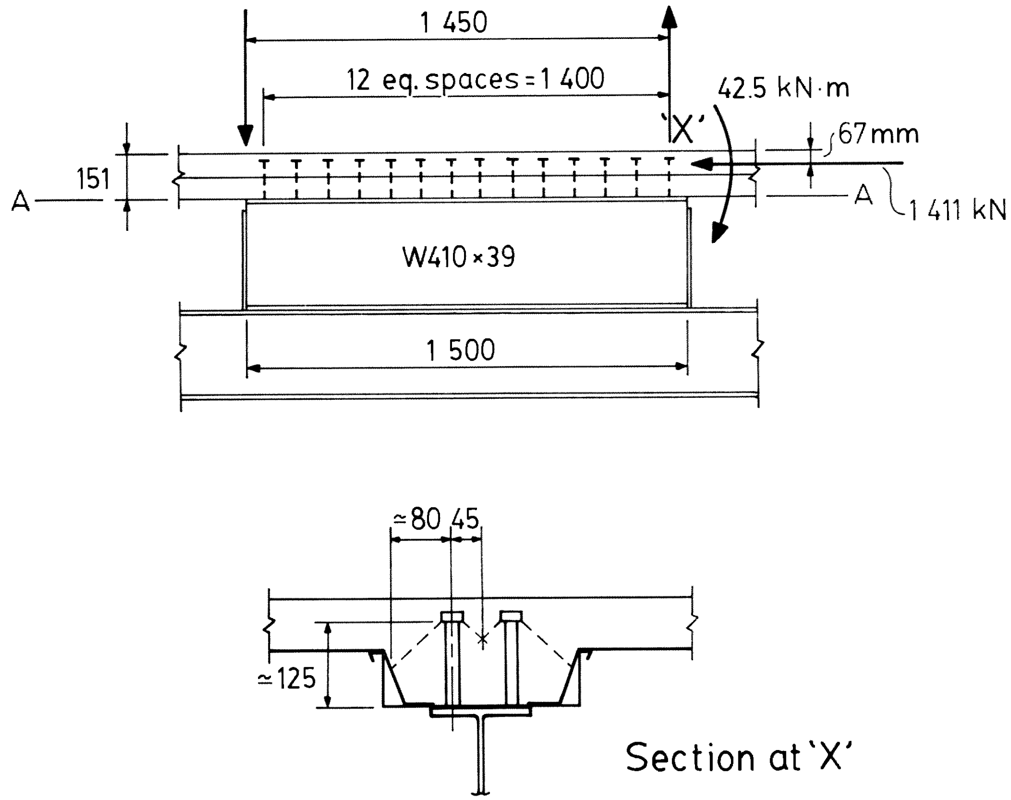


Figure 6.E14
Stud Distribution in Exterior Stubs

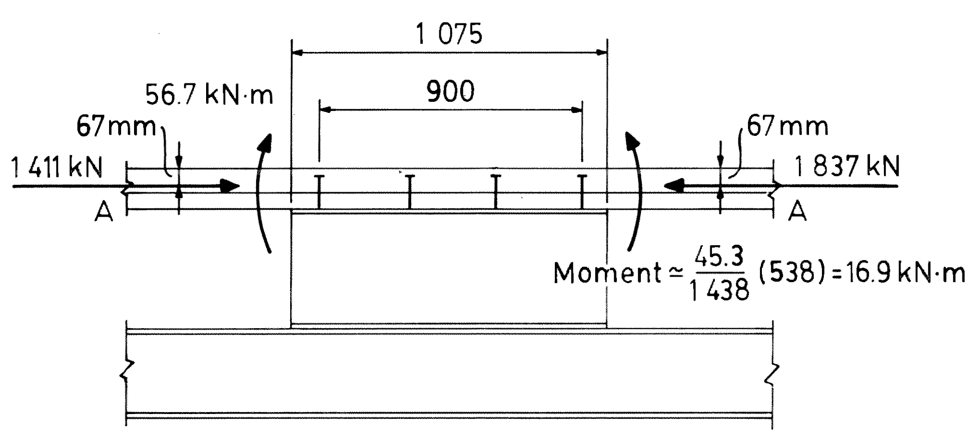


Figure 6.E15
Stud Distribution in Interior Stubs

Total number of studs for shear action $= \dfrac{426}{q_r} \qquad = 4.9$

Total number of studs for overturning $= \dfrac{4.0}{31 \times 0.9} \qquad = 0.14$

Total for all effects $=$ 5.04

Allowing for 50% extra, use **8 studs** for each interior stub.

– Design of transverse slab reinforcement to provide adequate deck-slab longitudinal shear resistance (Location 6, Fig. 6.E10).

The results of 5 Canadian and 2 U.S. full scale stub-girder tests, using concrete slabs of normal density and semi-low density of strengths 22 to 33 MPa, transversely reinforced with rebars of various configurations, were analysed during the preparation of the following design method. Up to four components of horizontal resisting forces are considered in the idealized failure mechanism, which include,

a) Axial resistance due to concrete in compression,
b) Axial resistance due to longitudinal steel in compression,
c) Longitudinal shear resistance of slab, reinforced by transverse rebars, and
d) Additional longitudinal shear resistance due to the longitudinal component of transverse reinforcement in tension, when "herring-bone" patterned transverse reinforcing is used.

The mean value of the computed ratios of total ultimate horizontal resistance tested to that predicted, for the seven full scale models, is found to be 1.03. An overall ϕ factor of 0.60 is used for the following example calculation.

(I) Try transverse reinforcing configuration type (I) .
Use 2 layers of mesh over stub-girder; see Fig. 6.E16, case (A)

(a) Axial ultimate resistance due to concrete in compression,

$$0.85\, f'_c\, A_{cs} = 0.85\,(0.025)(109)(151) = 350 \text{ kN}$$

(b) Axial ultimate resistance due to longitudinal steel in compression,

$$f_y A_s = (0.4)(200)(2) = 160 \text{ kN}$$

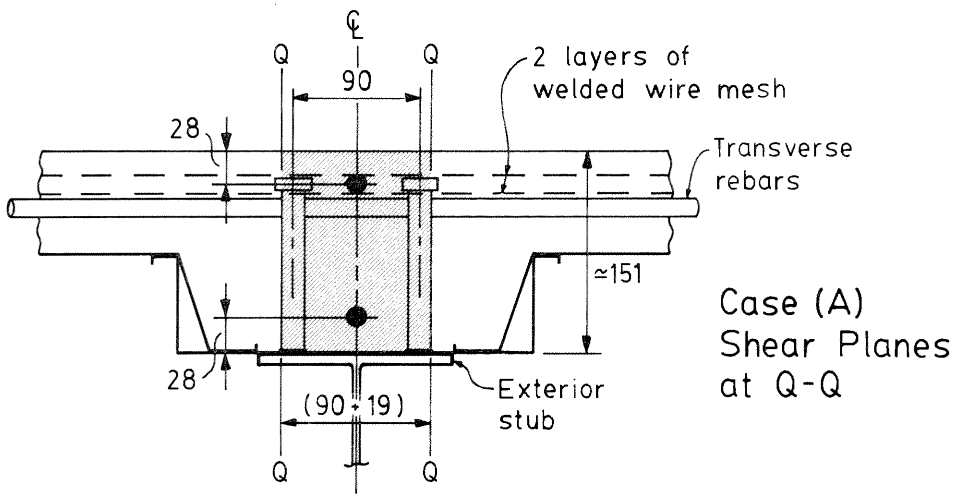
(c) Longitudinal shear ultimate resistance of the reinforced slab (credit one layer of mesh only; mesh size - 152×152 MW18.7×MW18.7).

$$\rho = \frac{18.7}{152(75+76)} = 0.000\,815$$

$$\begin{aligned} v_u &= 0.8\,\rho\, f_y + 2.76 \qquad \text{(from Eq. 4.20)}\\ &= 0.8\,(0.000\,815)(400) + 2.76\\ &= 3.02 \text{ MPa} < 0.3 f'_c\\ V_u &= 2\, l_{sh}(t_c + t_d) v_u\\ &= 2(1\,500)(75+76)(3.02)10^{-3}\\ &= 1\,368 \text{ kN}\end{aligned}$$

Total factored resistance of the failure mechanism,

$$0.6\,(350 + 160 + 1\,368) = 1\,127 \text{ kN} < 1\,411 \text{ kN}$$



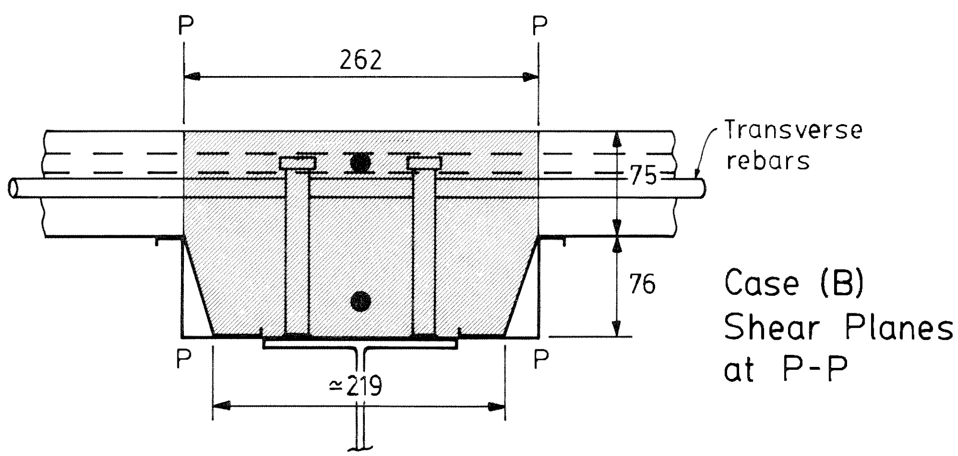


Figure 6.E16
Idealized Failure Mechanisms Used for Transverse Reinforcing Design

Therefore, transverse reinforcing configuration type (I) is not satisfactory for use in this example girder. No need to check case (B).

(II) Try transverse reinforcing configuration type (II) (same as in type (I) except add 5-15M straight bars transversely to girder span, over the exterior stub).

For Case (A), Calculations (a) and (b) are same as above; calculation (c) is to be modified as,

$$\rho = \frac{18.7}{152(75+76)} + \frac{5(200)}{1\,500(75+76)} = 0.00523$$

$$\begin{aligned} v_u &= 0.8\,(0.00523)(400) + 2.76 \\ &= 4.43 \text{ MPa} < 0.3\, f'_c \\ V_u &= 2\, l_{sh}\,(t_c + t_d)\, v_u \\ &= 2\,(1\,500)(75+76)(4.43)\, 10^{-3} \\ &= 2\,007 \text{ kN} \end{aligned}$$

Total factored resistance of the failure mechanism, for Case (A) shear planes,
0.6 (350 + 160 + 2 007) = 1 510 kN > 1 411 kN

For Case (B), shear planes are assumed to occur at P-P

(a) Axial ultimate resistance due to concrete in compression,

$$\begin{aligned} 0.85\, f'_c\, A_{cs} &= 0.85\,(0.025)\,[(75)(262) + (76)(262 + 219)/2] \\ &= 806 \text{ kN} \end{aligned}$$

(b) Axial ultimate resistance due to longitudinal steel in compression,

$$f_y\, A_s = (0.4)(200)(2) = 160 \text{ kN}$$

(c) Longitudinal shear ultimate resistance of the reinforced slab (one layer of mesh plus 5-15M straight bars)

$$\rho = \frac{18.7}{152(75)} + \frac{5(200)}{1\,500(75)} = 0.0105$$

$$\begin{aligned} v_u &= 0.8\, \rho\, f_y + 2.76 \\ &= 0.8(0.0105)(400) + 2.76 \\ &= 6.12 \text{ MPa} < 0.3\, f'_c \end{aligned}$$

$$\begin{aligned} V_u &= 2\, l_{sh}\,(t_c)\, v_u \\ &= 2(1\,500)(75)(6.12)\, 10^{-3} \\ &= 1\,377 \text{ kN} \end{aligned}$$

Total factored resistance of the failure mechanism, for Case (B) shear planes,
0.6(806 + 160 + 1 377) = 1 406 kN (more critical than Case (A))

(III) Try transverse reinforcing configuration type (III) (same as in type (I) except add 4-15M bent bars to be arranged in a "herring-bone" pattern over the exterior stub). See similar detail in Fig. 6.14.

For Case (A), shear planes are assumed to occur at Q-Q
Axial ultimate resistance due to concrete and longitudinal steel reinforcing,
(350 + 160) = 510 kN.

Longitudinal shear ultimate resistance of the reinforced slab (one layer of mesh plus rebar at a 45 degree angle),

$$\rho = \frac{18.7}{152(75+76)} + \frac{4(200)/\sqrt{2}}{1\,500(75+76)} = 0.00331$$

$$\begin{aligned} v_u &= 0.8(0.00331)(400) + 2.76 \\ &= 3.82 \text{ MPa} < 0.3\, f'_c \end{aligned}$$

$$\begin{aligned} V_u &= 2\, l_{sh}\,(t_c + t_d)\, v_u \\ &= 2\,(1\,500)(75+76)(3.82)\, 10^{-3} \\ &= 1\,730 \text{ kN} \end{aligned}$$

Additional longitudinal shear resistance due to bent bars in tension,
$2\, f_y\, A_s/\sqrt{2} = 2(0.4)(200)(4)/\sqrt{2} = 453$ kN

Total factored resistance of the failure mechanism for Case (A) shear planes,
0.6 (510 + 1 730 + 453) = 1 616 kN

For Case (B), the total factored resistance of the failure mechanism in type (III) transverse reinforcing configuration can be shown as,
0.6 (966 + 1 101 + 453) = 1 512 kN (more critical than Case (A))

The following table summarizes the factored resistance* of slabs with 3 types of transverse reinforcing configurations, under two types of failure mechanisms:

Failure Mechanisms	Double Mesh (I)	Double Mesh plus 5-15M straight bars (II)	Double Mesh plus 4-15M bent bars at 45° angle (III)
Case (A)	1 127	1 510	1 616
Case (B)	1 023 (governs)	1 406 (governs)	1 512 (governs)

*values shown, in the above table, are in (kN) unit.

It is therefore concluded that either type (II) or type (III) transverse reinforcing configuration is satisfactory for use over the exterior stubs, since the governing factored ultimate resistance of the failure mechanism is, in each case, greater than (or very closely equal to) the factored applied load of 1 411 kN.

Also, it can be shown that transverse rebars are not required for the deck-slab over the interior stubs. Double mesh is assumed throughout the full length of the girder.

– Stiffener design at exterior stubs (see Fig. 6.E17)

Area of 'T' section in compression = (140)(10) + (54)(6.4) = 1 746 mm^2

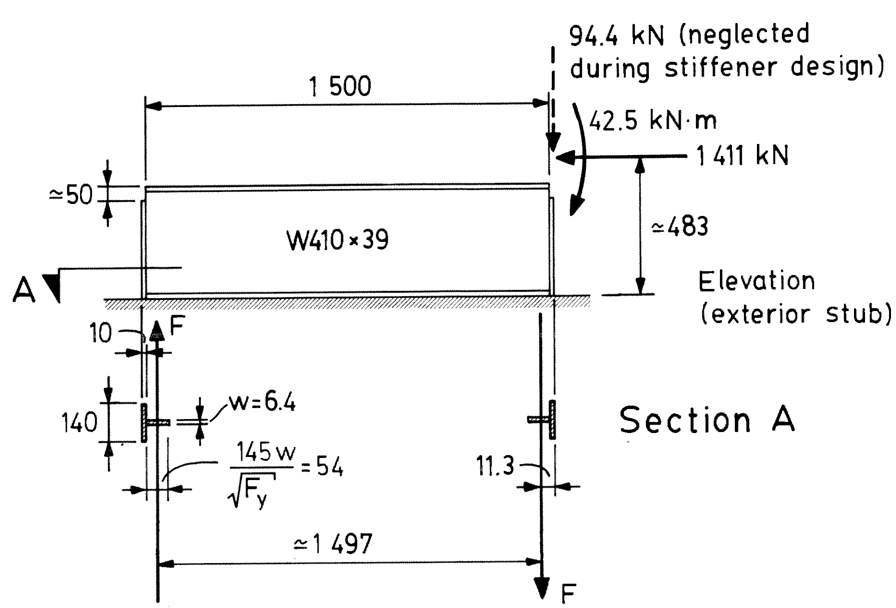


Figure 6.E17
End Stiffener Design – Exterior Stubs

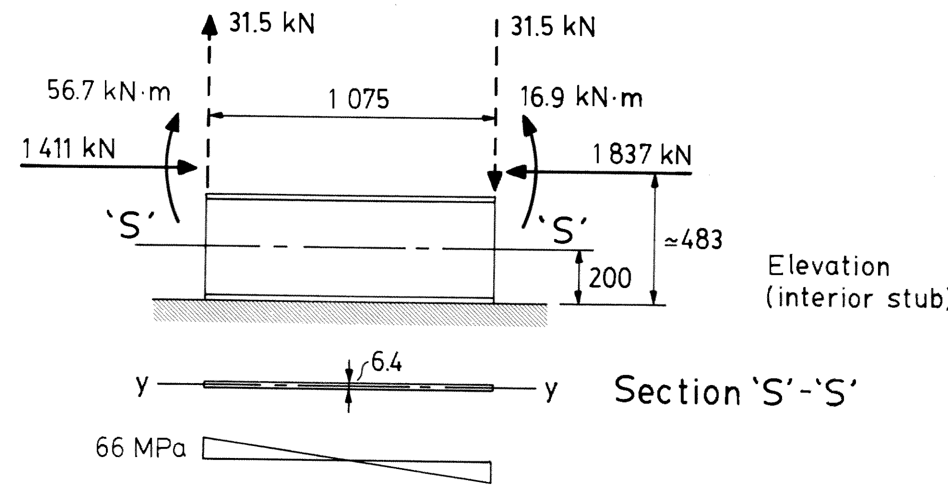


Figure 6.E18
Overturning on Interior Stubs
(Slab Shear Neglected During Design Checks)

C.G. of 'T' section from ends of stiffened stub,

$$\frac{140(10)(5) + (54)(6.4)(10 + 54/2)}{(1\ 746)} = 11.3 \text{ mm}$$

Total factored overturning moment = 1 411(0.483) − 42.5 = 639 kN·m

Total factored resisting moment = F[1 520 − 2(11.3)] 10^{-3} = (1.497)F

Equating moments,
1.497 F = 639
F = 639/1.497 = 427 kN

Factored axial resistance of the 'T' section,
$\phi A_s F_y$ = 0.9 (1 746)(0.3)
= 471 kN > 427 kN OK

Use end stiffeners (size 10 × 140 × 350 long)

– Check stiffener requirement at interior stubs (see Fig. 6.E18)

Total factored overturning moment acting on the interior stub about level 'S'-'S',
(1 837 − 1 411)(0.483 − 0.200) + 16.9 − 56.7
= 81.0 kN·m (say)

$$\text{Section modulus of web section} = \frac{6.4 \times 1\ 075^2}{6} = 1\ 233 \times 10^3 \text{ mm}^3$$

$$\text{Compressive stress due to bending} = \frac{(81)\ 10^6}{(1\ 233)\ 10^3} = 66 \text{ MPa}$$

r_{yy} of web plate $= \dfrac{6.4}{\sqrt{3}} = 3.69$ mm

$\dfrac{Kl}{r}$ of web plate acting as a small column with both ends fixed

$$= \frac{0.65\,(d - 2k)}{r}$$

$$= \frac{0.65\,[399 - 2(26)]}{3.69}$$

$$= 61$$

Unit factored compressive resistance, C_r/A for the small column = 205 MPa. This is greater than 66 MPa; OK. (See Handbook of Steel Construction P. 4-11).

– Check shear in web of exterior stub (horizontal shear)
(Cl. 13.4.1 of S16.1)

$$a/h = (399)/1\,500 = 0.266 < 1.0$$
$$h/w = 1\,500/6.4 = 234$$
$$k_v = 4 + 5.34/(a/h)^2 = 79.5$$

$$h/w > 439\sqrt{\frac{k_v}{F_y}} \quad \text{or} \quad 234 > 226$$

$$h/w < 502\sqrt{\frac{k_v}{F_y}} \quad \text{or} \quad 234 < 258$$

$$F_s = \frac{290\sqrt{F_y\,k_v}}{(h/w)} = 191 \text{ MPa}$$

Factored shear resistance of the web,
(0.9)(1 500)(6.4)(0.191) = 1 650 kN > 1 411 kN (OK)

– Check shear in web of interior stub (horizontal shear)

Factored shear acting on the web of interior stub
= 1 837 − 1411 = 426 kN

Factored shear resistance of the web of interior stub
$= 0.5\,\phi\,A_s\,F_y = 0.5(0.9)(1\,075)(6.4)(0.3)$
= 929 kN > 426 kN (OK)

Note: Cl.13.4.4 of S16.1, factored shear resistance for gusset plate is assumed.

– Design of stub-to-girder welding (exterior stub)

Factored overturning moment at base of exterior stub = 639 kN·m (as shown before)
Factored shear at base of exterior stub = 1 411 kN (see Fig. 6.E19)

Try weld configuration as shown in Fig. 6.E19, assuming the overturning moment is resisted by couple forces, F.

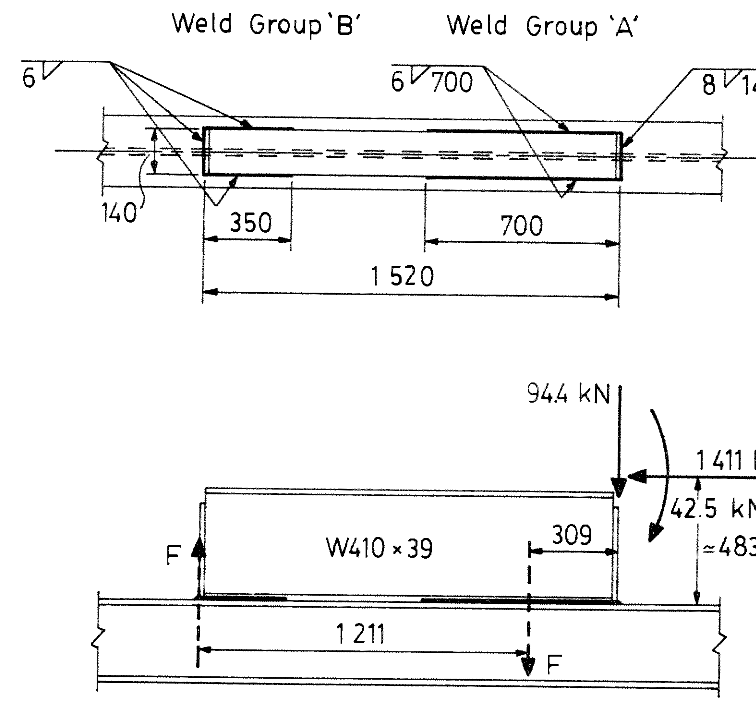


Figure 6.E19
Design of Exterior Stub to Girder Welding

C.G. of weld group 'A' from right end of stub

$$= \frac{140(1.22)(0) + 2(700)(0.918)(700/2)}{140(1.22) + 2(700)(0.918)} = 309 \text{ mm}$$

where, values 1.22 and 0.918 represent the factored shear resistance (kN) per millimetre length of 8 mm and 6 mm fillet welds respectively (see Handbook of Steel Construction P3-37).

Lever arm for overturning resistance = 1 211 mm

Factored tensile force, $F = \dfrac{639}{1.211} = 528$ kN

Reduction of tensile force due to factored slab shear of 94.4 kN
= 94.4(1 520)/(1 211) = 118 kN

Factored tensile resistance of weld group 'A'
= 140(1.22) + 2(700)(0.918) = 1 456 kN

Factored shear resistance of weld group 'A' and 'B'
= (350 + 350 + 140 + 700 + 700)(0.918) + (140)(1.22) = 2 227 kN

Percentage of utilization of weld group 'A'
= 100 [(528−118)/1 456 + 1 411/2 227]
= 91.5 < 100 OK

– Design of stub-to-girder welding (interior stub)

Factored O.T.M. at base of interior stub
$= (426)\,(0.483) = 206$ kN·m

Factored shear at base of interior stub
$= 426$ kN

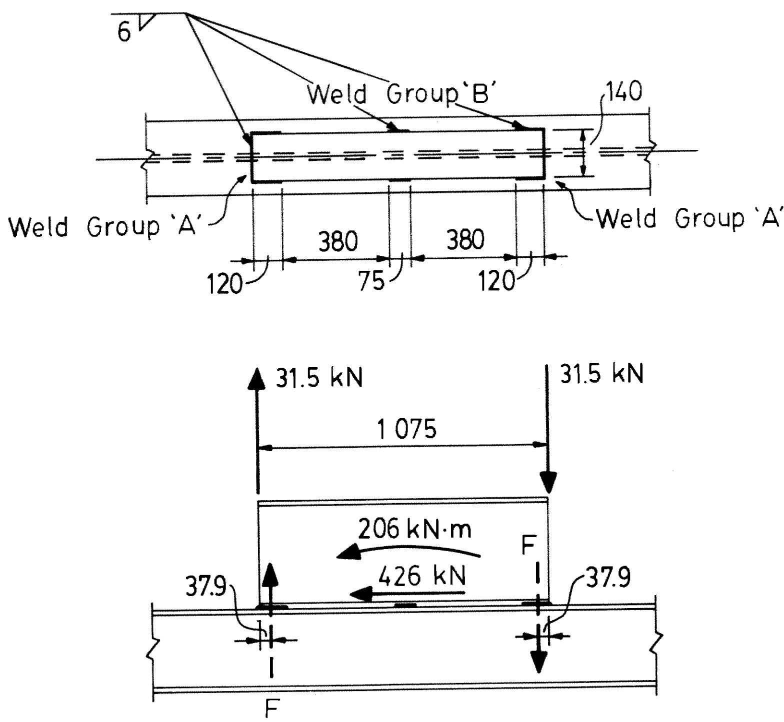


Figure 6.E20
Design of Interior Stub to Girder Welding

Try weld configuration as shown in Fig. 6.E20, assuming the overturning is resisted by couple forces, F.

C.G. of weld group 'A' from end of stub
$= 240\,(120/2)/(240 + 140)$
$= 37.9$ mm

Factored tension force F due to the factored O.T.M. plus the effects of slab shear,
$= [(206) - 31.5(1.075)]/[1.075 - 2(0.0379)]$
$= 172$ kN

Factored tensile resistance of weld group 'A'
$= (120 + 140 + 120)(0.918) = 349$ kN

Factored shear resistance of all welds
$= (2)(120 + 140 + 120 + 75)(0.918) = 835$ kN

Percentage of utilization of weld group 'A'
$= 100\,(172/349 + 426/835)$
$= 100.3 \simeq 100$ OK

– Check elastic deflection with no consideration of creep of concrete, (in terms of equally spaced point load, P kN)

a) Mid span deflection due to "chord action"

Transformed area of top chord, $A_c = 39\,703$ mm^2
Area of steel bottom chord, $A_s = 11\,000$ mm^2

Moment of inertia of transformed concrete top chord,
$I_c = 74.8 \times 10^6$ mm^4 (same as I_t as computed from Fig. 6.E8)

Moment of inertia of steel bottom chord,
$I_s = 199 \times 10^6$ mm^4

From Figure 6.E9, moment of inertia of top and bottom chord,
$I = A_c(138)^2 + A_s(500)^2 + I_c + I_s$
$= 3\,780 \times 10^6$ mm^4

$$\Delta_a = \frac{19}{384}\,\frac{Pl^3}{EI}$$

$$= \frac{19}{384}\,\frac{P\,(11\,500)^3}{200\,000(3\,780)}\,10^{-3} = 0.0995\ P\ \text{(mm)}$$

b) Deflection due to bending of end cantilevers

Cantilever length $(l_c) = 625$ mm
End reaction $= (3)P/2$

$$\Delta_b = \frac{\left(\frac{3P}{2}\right) l_c^3}{3\,E\,I_s}$$

$$\frac{P\,(625)^3}{2\,E\,(199)}\,10^{-3} = 0.0031\ P\ \text{(mm)}$$

c) Deflection due to bending of chords between points F and H, Fig. 6.E21

Since shear forces at top and bottom chords are proportioned on the basis of their relative stiffnesses, additional flexural deflection computed using the bottom chord should be equal to that computed based on the top chord.

Shear at hinge B $= 1.095$ P (bottom chord)
Shear at location G $= 0.365$ P

Flexural deflection in segment FG,

$$= \frac{(1.095P)(0.6x)^3}{3\,E\,I_s} + \frac{(1.095P)(0.4x)^3}{3\,E\,I_s} + \frac{2(1.095P)(0.4x) + (0.365P)(y)}{2\,E\,I_s}\,(y)(0.4x)$$

$$= \frac{(1.095P)(450)^3}{3\,E\,(199)(10^3)} + \frac{(1.095P)(300)^3}{3\,E\,(199)(10^3)} + \frac{2(1.095P)(300) + (0.365P)(900)}{2\,E\,(199)(10^3)}\,(900)(300)$$

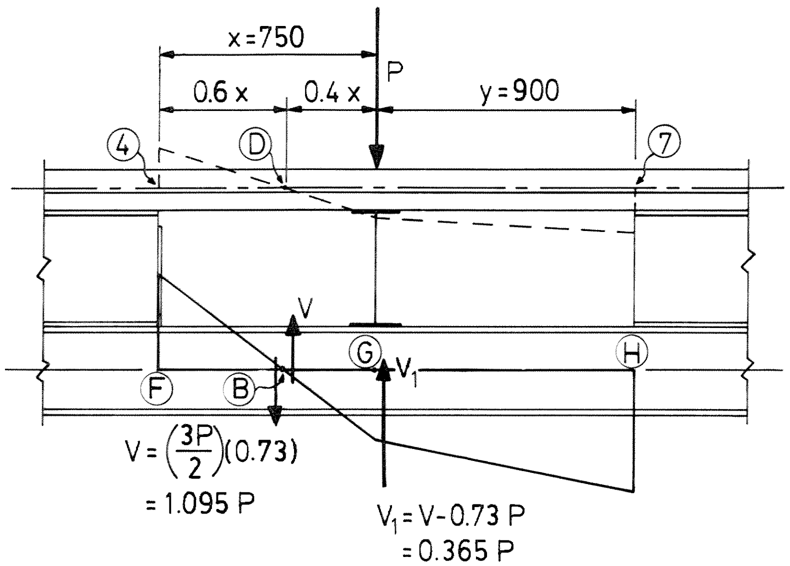


Figure 6.E21
Bending in Bottom Chord Member
Between Points F and H

$$= \frac{10^6}{E\,(199)(10^3)}(33.26 + 9.86 + 133)\,P = 0.0044\,P\ (mm)$$

Flexural deflection in segment GH,

$$= \frac{(0.365P)(900)^3}{3\,E(199)(10^3)} + \frac{(1.095P)(300)(900)^2}{2\,E\,(199)(10^3)} = 0.0056\,P\ (mm)$$

Therefore, the flexural deflection of chord members between points F and H,
$\Delta_c = (0.0044 + 0.0056)\,P$
$= 0.0100\,P$ (mm)

d) Deflection due to bending of chords at central openings, Fig. 6.E22.
Shear at bottom chord = 0.365 P

$$\Delta_d = \frac{(0.365P)(900)^3}{3\,E\,(199)(10^3)} + \frac{(0.365P)(900)^2(537.5)}{2\,E\,(199)(10^3)} = 0.0042\,P\ (mm)$$

Total deflection of stub-girder (chord + flexural)
$= \Delta_a + \Delta_b + \Delta_c + \Delta_d$
$= (0.0995 + 0.0031 + 0.0100 + 0.0042)\,P$
$= 0.1168\,P$ (mm)

Since $P_c = 81.9$ kN
$P_L + P_p + P_{OD} = 43 + 33.6 + 19.6 = 96.2$ kN

Deflection due to fresh-concrete condition load
= 0.1168(81.9) = 9.6 mm (girder not cambered)

Deflection due to live load plus superimposed dead loads (96.2 kN)
= 0.1168(96.2) = 11.2 mm

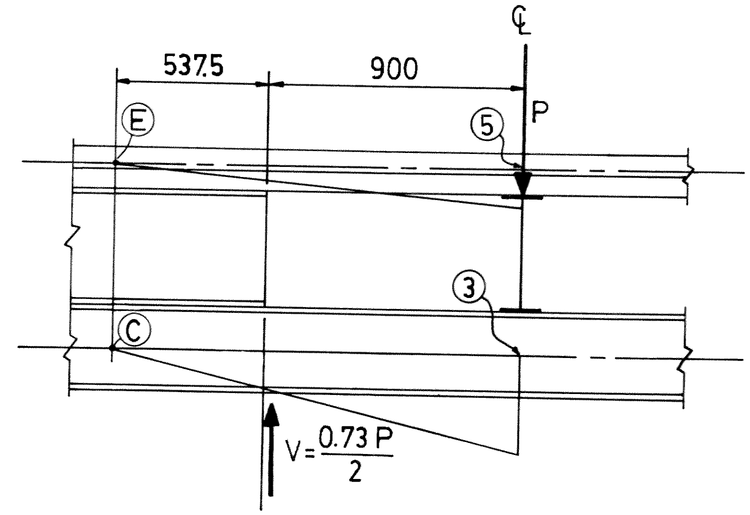


Figure 6.E22
Bending in Bottom Chord Member
at Central Opening

Since there is no camber, total deflection of girder
$= 9.6 + 11.2 \simeq 21$ mm; $< L/300$.
Therefore OK

– Check girder deflection with consideration of concrete creep using $n = E_s/(E_c/2.5) = 21.1$

$A_c = 19\ 543\ mm^2$ $\quad I_c = 39.9 \times 10^6\ mm^4$
$A_s = 11\ 000\ mm^2$ $\quad I_s = 199 \times 10^6\ mm^4$

Effective depth: top-bottom chord = 633 mm

Shear shared by bottom chord = 83%, instead of 73% computed in the previous case.

Moment of inertia of combined top and bottom chords (spaced 633 mm apart)
$= 3\ 060 \times 10^6\ mm^4$

$\Delta_a = 0.1230\,P$ (mm)
$\Delta_b = 0.0031\,P\,(83)/(73) = 0.0035\,P$ (mm)
$\Delta_c = 0.0100\,P\,(83)/(73) = 0.0114\,P$ (mm)
$\Delta_d = 0.0042\,P\,(83)/(73) = 0.0048\,P$ (mm)

Total deflection of stub-girder (chord + flexural) including concrete creep effect, under live load plus partition plus other dead load, plus fresh concrete condition load,

$$\Delta = \Delta_a + \Delta_b + \Delta_c + \Delta_d = 0.1427\,P \simeq 25.4\ mm$$

This is less than L/300 OK

– Structural modelling of stub-girder for detailed analysis using a stiffness analysis computer program (Colaco method), see Fig. 6.11 and Fig. 6.E23

$I_l = I_{xx}/6 = 566 \times 10^6\ mm^4$

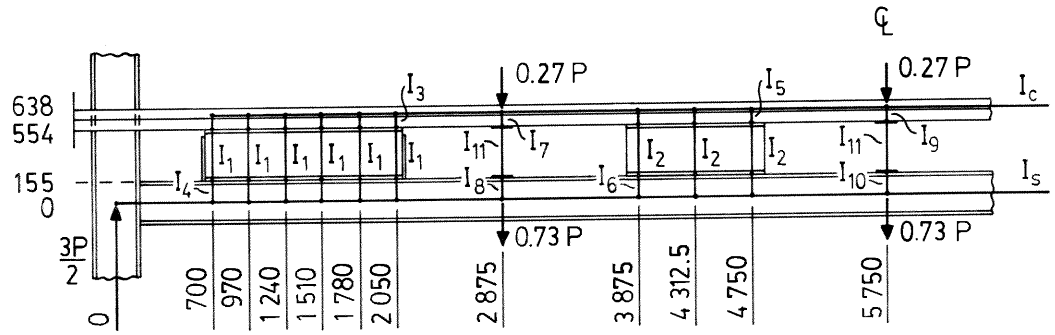


Figure 6.E23
Structural Modelling – Colaco Method

where $I_{xx} = 2(140)(10)(755)^2 + \frac{6.4\ (1\ 500)^3}{12} = 3\ 396 \times 10^6\ mm^4$

$I_2 = I_{x'x'}/3 = 221 \times 10^6\ mm^4$

where $I_{x'x'} = \frac{6.4(1\ 075)^3}{12} = 662.6 \times 10^6\ mm^4$

I_3 to I_{10} are very large fictitious values, say
$= 5$ times $I_1 \simeq 2\ 800 \times 10^6\ mm^4$

$I_{11} = \frac{2(399)(6.4)^3}{12} \simeq 0.02 \times 10^6\ mm^4$

$I_c = 74.8 \times 10^6\ mm^4$

$I_s = 199 \times 10^6\ mm^4$

$A_1 = [2(10)(140) + (6.4)(1\ 500)]/6 = 2\ 067\ mm^2$

$A_2 = 6.4(1\ 075)/3 = 2\ 293\ mm^2$

A_3, A_5, A_7, A_9 assume $8\ 000\ mm^2$

A_4, A_6, A_8, A_{10} assume $2\ 300\ mm^2$

$A_{11} = 6.4(399) = 2\ 550\ mm^2$

$A_c = 39\ 703\ mm^2$

$A_s = 11\ 000\ mm^2$

The result of the stiffness analysis run is compared to the forces computed based on the "statically-determinate" model, see Table 6.E1.

– Structural modelling of stub-girder using a stiffness analysis computer program (simplified method), see Fig. 6.E24.

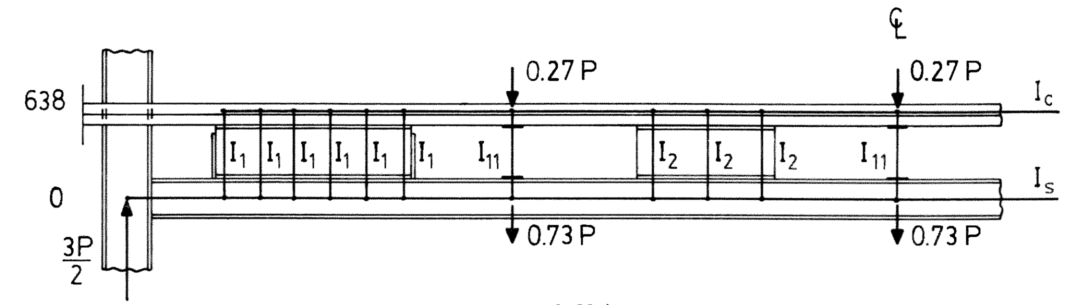


Figure 6.E24
Structural Modelling – Simplified Method

The values of A and I are as computed above. The results of the stiffness analysis run are also compared to the "statically determinate" model; see table below.

COMPARISON OF ANALYSIS METHODS

Location on the stub-girders	Structural Actions*		Analysis Methods		
			Statically determinate Model	Calaco Model	Simplified Vierendeel Model
1	Axial	(kN)	0	0	0
	Bending	(kN·m)	219	219	219
2	Axial	(kN)	1 411	1 413	1 415
	Bending	(kN·m)	153	147	143
3	Axial	(kN)	1 837	1 860	1 855
	Bending	(kN·m)	122	110	111
	Deflection	(mm)	21	20.4	20.8
4	Axial	(kN)	1 411	1 413	1 415
	Bending	(kN·m)	42.5	49.5	53.1
5	Axial	(kN)	1 837	1 860	1 855
	Bending	(kN·m)	45.3	43.4	45.2
7	Axial	(kN)	1 411	1 413	1 415
	Bending	(kN·m)	56.7	61.8	63.9

*Note: 1. Floor load for deflection calculation includes total structural plus superimposed loads.
2. All axial and bending forces listed are based on factored loads in accordance with S16.1.

6.20 TRIAL SELECTION TABLES FOR STUB-GIRDER FLOOR BAY DESIGN

The Trial Selection Tables for stub-girder floor bay design in the following pages provide typical preliminary interior stub-girder floor-bay designs suitable for more detailed structural analysis.

The information provided within the selection tables was computed based on design rules similar to the design procedures shown in section 6.19. However, certain procedures of design checks have been further simplified to provide tabulated output suitable only for trial selection purposes.

Two sets of trial selection tables are provided:
- stub-girder floor bays with stub-girders each containing four stubs and three equally spaced Gerber beams.
- stub-girder floor bays with stub-girders each containing three stubs and two equally spaced Gerber beams.

Two live load cases (2.4 kPa and 3.6 kPa) are included for each set of trial selection tables. Live load reduction as defined by equation 3.2 is assumed for all floor bay designs. Partition loading of 1.2 kPa is included in the dead load.

Two cover slab thicknesses and two concrete densities are available for each combination of stub-girder configuration and live load case:
- 75 mm thick, normal density concrete (2 300 kg/m^3)
- 85 mm thick, semi-low density concrete (1 850 kg/m^3)

To summarize, trial selection tables for stub-girder floor bays occur in the order shown below:

	Live Load (2.4 kPa)		Live Load (3.6 kPa)	
Description	75 mm ND Concrete	85 mm SLD Concrete	75 mm ND Concrete	85 mm SLD Concrete
four-stub configuration	Table 6.2	Table 6.3	Table 6.4	Table 6.5
three-stub configuration	Table 6.6	Table 6.7	Table 6.8	Table 6.9

In addition to the items discussed above, design criteria common to all floor bay designs are listed below:

a) Superimposed dead load = 1.9 kPa
b) Steel deck profile: 76 mm deep, wide-rib profile deck (T-30-V assumed)
c) Concrete strength: 25 MPa
d) Longitudinal bars:
3-15M bars at 25 mm from top of slab (with one 15M bar occurring within mid-flute)
4-15M bars at 120 mm from top of slab
e) Transverse bars: 5-15M 'straight' bars over exterior stub
f) Welded wire mesh:

One layer 152×152 MW9.1×MW9.1 mesh over all floor areas.
One additional layer over girder locations.

g) Shear stud diameter: 19 mm
h) Steel Costing Method:
Use costing method as described in Ref.(17).
Cost Index used = 1 000

For each typical floor bay, the following design data are provided in the trial selection table:

a) Cantilever segment member size selected along with the number of stud shear connectors required for each cantilever segment member.
b) Length of the cantilever segment member and the computed mid-span deflections of the cantilever segment under the conditions of:
   - during slab pour (fresh-concrete loading)
   - superimposed dead load
   - live loads

c) Percentage of member resistance utilization at:
   - mid span of the cantilever segment
   - cantilever support of the cantilever segment
d) Suspended segment member size selected; required number of stud shear connectors.
e) Same as in b) except for suspended segment.
f) Percentage of member resistance utilization at mid span of the suspended member.
g) Selected stub-girder bottom chord size; required total number of stud shear connectors per girder.
h) Width of hole No. 1 (see Fig. 6.5),
Length of exterior stub,
Number of shear studs atop exterior stub.
i) Width of hole No. 3 (see Fig. 6.5),
Length of interior stub,
Number of shear studs atop interior stub.
j) Percentage of resistance utilization at bottom chord locations 1, 2 and 3. (see Figs. 6.9-10)
k) Percentage of resistance utilization at top chord locations 4, 5, and 6. (see Figs. 6.9-10).
l) Mid span deflection of stub-girder under the conditions of:
   - shore removal (fresh concrete loading on composite girder)
   - superimposed dead load
   - live load
   - total load
m) Steel mass and cost per typical bay.

All length dimensions and deflection values provided are expressed in millimetres. The steel mass presented in each floor bay is expressed in kilograms; and the cost per typical bay of girder plus average Gerber beam system (structural steel only) is shown in dollars.

To illustrate the use of the trial selection tables, the floor design example as presented in Section 6.19 is reused here and results compared.

Basic design data for table selection:

a) Stub-girder configuration 4 stubs/girder.
b) Live load = 2.4 kPa.
c) Cover Slab: 75 mm, ND Concrete
d) Superimposed Dead Load = 1.9 kPa. (Partition = 1.2, Mechanical/Ceiling = 0.5, Fire Protection and Floor Finish = 0.2).
e) Other design criteria (similar to the design criteria as shown in the explanations above)
f) Girder span = 11 500 mm
g) Beam spans for beam selection = 9 000 mm
Beam span for girder selection = 10 000 mm
(since the tributary area for each beam reaction at the typical girder designed = 28 m^2)

Description	Results by Hand-computation	Results by Trial Selection Table
Cantilever Segment Size (studs)	W410×39(16)	W410×39(16)
B1 length (mm)	12 700	12 720
Utilization B1 (%)	90	86
Suspended Segment Size (studs)	W200×27(12)	W200×27(12)
B3 length (mm)	5 300	5 280
Utilization B3 (%)	62	55
Stub-girder size (studs)	W310×86(68)	W310×79(72)
Utilization (%) at 1 / 2 / 3	70/87/94	81/94/99
Utilization (%) at 4 / 5 / 6	65/61/84	80/67/75
Total deflection (mm)	21	22

REFERENCES

6.1 Colaco, J.P., "A Stub-Girder System for Highrise Buildings", AISC Engineering Journal, July 1972.

6.2 Zimmerman, T.J., "Analysis and Design of Stub-Girders", M.Sc. Thesis, University of Alberta, March 1981.

6.3 Bjorhovde, R., and Zimmerman, T.J., "Some Aspects of Stub-Girder Design", Canadian Structural Engineering Conference, Proceedings, 1980.

6.4 Kullman, R.B., and Hosain, M.U., "Shear Strength of Stub-Girders: Full-Scale Tests", Canadian Society for Civil Engineering, Annual Conference Proceedings, 1980.

6.5 Buckner, C.D., Deville, D.J., and McKee, D.C., "Shear Strength of Slabs in Stub Girders", Journal of the Structural Division, ASCE, ST2, 1981.

6.6 "Embedment Properties of Headed Studs", TRW Nelson Division, Design Data 10, 1977.

6.7 Stringer, D.C., "Staggered Truss and Stub-Girder Framing Systems in Western Canada", Canadian Structural Engineering Conference, Proceedings, 1982.

6.8 Gunnin, B.L., "The First International Building in Dallas, Texas (USA)", Acier-Stahl-Steel, March 1976.

6.9 Colaco, J.P., "Partial Tube Concept for Mid-Rise Structures", AISC Engineering Journal, Fourth Quarter 1974.

6.10 "Stubs Atop Girder Flange Cut Building Cost", Engineering News Record, August 31, 1972.

6.11 "Pennzoil Place, Houston, Texas", Bethlehem Steel, Building Case History, No. 47.

6.12 "One Houston Centre, Houston, Texas", Bethlehem Steel, Building Case History, No. 60.

6.13 Taranath, B.S., "Composite Design for First City Tower", The Structural Engineer, September 1982.

6.14 "Five-Layer Transfer Braces Tower Notch", Engineering News Record, February 1980 (also ENR March 1982).

6.15 "Big Tower Crane Overcomes Height and Space Restrictions", Heavy Construction News, September 1982.

6.16 "Georgia Power Company Corporate Headquarters Building, Atlanta, Georgia", Bethlehem Steel, Building Case History, No. 76, 1982.

6.17 "A Project Analysis Approach to Building Costs", Canadian Institute of Steel Construction and Canadian Steel Construction Council, 1983.

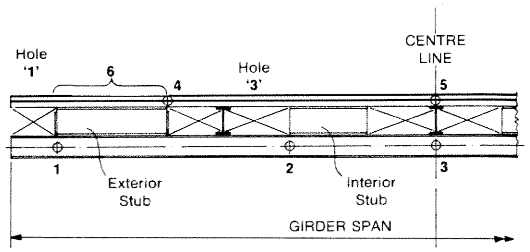
6.18 Matthews, C.M., Montgomery, C.J., Murray, D.W., "Designing Floor Systems for Dynamic Response", Structural Engineering Report No. 106, University of Alberta, October 1982.

6.19 Mirza, S.A., MacGregor, J.G., "Probabilistic Study of Strength of Reinforced Concrete Members", Canadian Journal of Civil Engineering, Vol. 9, No. 3, 1982.

6.20 Mills, R.W., "401 West Georgia Project, Vancouver, B.C.", Proceedings, Canadian Structural Engineering Conference, Feb. 1984.

Table 6.2
STUB-GIRDER FLOOR BAY+
Trial Selection Table

4-STUB CONFIGURATION



Live Load: 2.4 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
10000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 9/2/7	14720 16/4/10	15860 20/5/11	17660 9/3/9
	% Utilization: Mid span/Support	35 / 67	45 / 77	56 / 86	69 / 96	68 / 90	50 / 80
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X31 (14)	W250X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 13/1/1	5280 14/2/2	6280 21/3/3	7280 22/3/4	8140 19/4/5	7340 27/4/5
	% Utilization: Mid span	47	48	57	66	67	67
	S-G bottom chord size (Studs)	W310X67 (48)	W310X67 (52)	W310X74 (60)	W310X74 (64)	W310X74 (64)	W310X79 (64)
	Length: Hole 1/Ext stub, (Studs)	710/1140 (18)	710/1140 (20)	710/1140 (24)	710/1140 (26)	710/1140 (26)	690/1160 (26)
	Length: Hole 3/Int stub, (Studs)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)
	% Utilization at: 1/2/3	61 / 68 / 73	68 / 76 / 81	66 / 76 / 81	72 / 83 / 88	78 / 89 / 95	73 / 84 / 89
	% Utilization at: 4/5/6	86 / 49 / 50	95 / 55 / 56	64 / 58 / 61	69 / 63 / 67	78 / 68 / 72	85 / 66 / 66
	Defl:Shore removed+SD+LL=Total	5+4+3=12	5+4+4=13	5+4+4=13	6+5+4=15	6+5+4=15	6+4+3=13
	Steel mass / Cost per bay	2036 / 2565	2219 / 2782	2484 / 3080	2662 / 3289	3079 / 3744	3876 / 4294
10500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 10/2/7	14720 17/4/10	15860 21/5/12	17660 9/3/9
	% Utilization: Mid span/Support	37 / 70	47 / 80	59 / 90	72 / 100	71 / 93	52 / 83
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X31 (14)	W250X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 14/1/1	5280 14/2/2	6280 22/3/3	7280 23/4/5	8140 20/4/5	7340 28/4/5
	% Utilization: Mid span	49	50	60	69	70	70
	S-G bottom chord size (Studs)	W310X67 (52)	W310X74 (56)	W310X74 (64)	W310X74 (68)	W310X79 (72)	W310X86 (68)
	Length: Hole 1/Ext stub, (Studs)	750/1195 (20)	750/1195 (22)	750/1195 (26)	740/1205 (28)	660/1285 (30)	740/1205 (28)
	Length: Hole 3/Int stub, (Studs)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)
	% Utilization at: 1/2/3	67 / 75 / 80	67 / 76 / 80	73 / 83 / 88	79 / 90 / 96	71 / 92 / 98	74 / 85 / 89
	% Utilization at: 4/5/6	100 / 53 / 54	70 / 57 / 60	77 / 62 / 66	84 / 68 / 71	70 / 72 / 75	66 / 68 / 71
	Defl:Shore removed+SD+LL=Total	6+4+4=14	6+5+4=15	7+5+4=16	7+6+4=17	7+6+4=17	6+5+4=15
	Steel mass / Cost per bay	2079 / 2601	2340 / 2894	2531 / 3119	2710 / 3328	3192 / 3838	4007 / 4413
11000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 10/2/7	14860 13/3/9	16960 11/3/10	17660 10/3/9
	% Utilization: Mid span/Support	38 / 73	49 / 83	61 / 93	61 / 88	61 / 100	54 / 87
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X31 (14)	W250X33 (14)	W250X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 15/2/2	6280 23/3/4	7140 23/4/4	7040 26/3/4	7340 29/4/5
	% Utilization: Mid span	51	52	63	70	68	74
	S-G bottom chord size (Studs)	W310X74 (56)	W310X74 (60)	W310X74 (68)	W310X79 (72)	W310X86 (76)	W310X86 (72)
	Length: Hole 1/Ext stub, (Studs)	780/1260 (22)	780/1260 (24)	780/1260 (28)	760/1280 (30)	680/1360 (32)	760/1280 (30)
	Length: Hole 3/Int stub, (Studs)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)
	% Utilization at: 1/2/3	65 / 74 / 79	72 / 83 / 88	79 / 91 / 97	79 / 93 / 99	69 / 93 / 99	80 / 93 / 98
	% Utilization at: 4/5/6	67 / 56 / 57	75 / 62 / 64	83 / 68 / 70	71 / 72 / 76	72 / 75 / 76	73 / 74 / 76
	Defl:Shore removed+SD+LL=Total	6+5+4=15	7+6+5=18	8+6+5=19	8+6+5=19	8+6+5=19	8+6+5=19
	Steel mass / Cost per bay	2205 / 2717	2388 / 2933	2580 / 3158	3072 / 3684	3683 / 4073	4068 / 4458

+ See Section 6.20 for explanation.

4-STUB CONFIGURATION

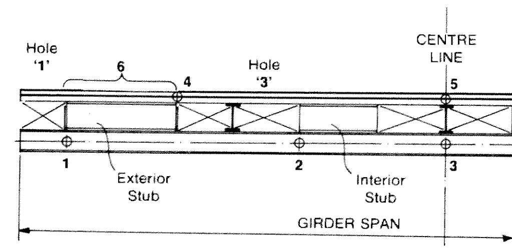


Table 6.2 (continued)
STUB-GIRDER FLOOR BAY+
Trial Selection Table

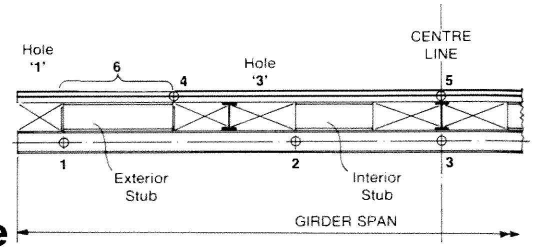
Live Load: 2.4 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
11500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 10/3/7	14860 14/3/9	16700 13/4/11	17660 10/3/9
	% Utilization: Mid span/Support	40 / 75	51 / 86	63 / 97	64 / 92	64 / 100	56 / 90
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X31 (14)	W250X33 (14)	W250X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 16/2/2	6280 23/3/4	7140 24/4/5	7300 29/4/5	7340 31/4/5
	% Utilization: Mid span	53	55	66	73	76	77
	S-G bottom chord size (Studs)	W310X74 (60)	W310X74 (64)	W310X79 (72)	W310X86 (76)	W310X97 (80)	W310X97 (76)
	Length: Hole 1/Ext stub, (Studs)	820/1315 (24)	820/1315 (26)	820/1315 (30)	770/1365 (32)	690/1445 (34)	690/1445 (32)
	Length: Hole 3/Int stub, (Studs)	920/1035 (6)	920/1035 (6)	920/1035 (6)	920/1035 (6)	920/1035 (6)	920/1035 (6)
	% Utilization at: 1/2/3	71 / 81 / 86	79 / 90 / 96	81 / 94 / 99	75 / 94 / 100	65 / 92 / 97	67 / 92 / 97
	% Utilization at: 4/5/6	72 / 61 / 61	80 / 67 / 68	71 / 73 / 75	74 / 76 / 80	75 / 79 / 81	76 / 78 / 78
	Defl:Shore removed+SD+LL=Total	8+6+5=19	9+7+5=21	9+7+6=22	9+7+5=21	9+7+5=21	8+6+5=19
	Steel mass / Cost per bay	2252 / 2756	2436 / 2972	2688 / 3257	3211 / 3810	3865 / 4238	4274 / 4636
12000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 6/2/5	13680 11/3/8	14860 15/3/9	17160 8/2/8	17660 10/3/10
	% Utilization: Mid span/Support	41 / 78	52 / 89	66 / 100	66 / 95	53 / 89	57 / 93
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (14)	W250X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 16/1/2	5280 16/2/2	6320 16/2/3	7140 25/4/5	6840 25/3/4	7340 32/4/5
	% Utilization: Mid span	56	57	60	76	70	80
	S-G bottom chord size (Studs)	W310X74 (64)	W310X79 (68)	W310X86 (76)	W310X97 (80)	W310X107 (80)	W310X107 (84)
	Length: Hole 1/Ext stub, (Studs)	850/1370 (26)	850/1370 (28)	850/1370 (32)	780/1440 (34)	780/1440 (34)	700/1520 (36)
	Length: Hole 3/Int stub, (Studs)	960/1080 (6)	960/1080 (6)	960/1080 (6)	960/1080 (6)	960/1080 (6)	960/1080 (6)
	% Utilization at: 1/2/3	77 / 88 / 94	79 / 93 / 98	79 / 94 / 100	71 / 93 / 98	69 / 88 / 92	64 / 92 / 96
	% Utilization at: 4/5/6	84 / 66 / 65	70 / 72 / 73	74 / 76 / 80	77 / 80 / 85	76 / 79 / 82	79 / 82 / 83
	Defl:Shore removed+SD+LL=Total	9+7+6=22	10+7+6=23	10+8+6=24	10+8+6=24	9+7+5=21	9+7+5=21
	Steel mass / Cost per bay	2300 / 2794	2546 / 3073	2852 / 3417	3407 / 3994	4358 / 4707	4468 / 4813
12500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W460X61 (24)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 6/2/5	13520 13/3/8	16160 4/2/6	17160 8/3/8	17660 11/3/10
	% Utilization: Mid span/Support	42 / 81	54 / 93	69 / 100	45 / 83	54 / 92	59 / 97
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W200X27 (12)	W250X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 17/1/2	5280 17/2/2	6480 18/3/4	5840 27/3/4	6840 26/3/4	7340 33/4/6
	% Utilization: Mid span	58	60	65	73	73	83
	S-G bottom chord size (Studs)	W310X79 (68)	W310X86 (76)	W310X97 (80)	W310X107 (80)	W310X107 (84)	W310X118 (100)
	Length: Hole 1/Ext stub, (Studs)	890/1425 (28)	890/1425 (32)	870/1445 (34)	870/1445 (34)	710/1605 (36)	630/1685 (38)
	Length: Hole 3/Int stub, (Studs)	1000/1125 (6)	1000/1125 (6)	1000/1125 (6)	1000/1125 (6)	1000/1125 (6)	1000/1125 (12)
	% Utilization at: 1/2/3	78 / 90 / 96	78 / 93 / 98	75 / 92 / 98	74 / 88 / 92	65 / 95 / 100	55 / 91 / 95
	% Utilization at: 4/5/6	68 / 70 / 69	72 / 75 / 77	76 / 79 / 84	76 / 79 / 82	82 / 85 / 84	82 / 86 / 85
	Defl:Shore removed+SD+LL=Total	10+8+7=25	11+8+7=26	11+8+7=26	10+7+6=23	10+8+6=24	10+7+6=23
	Steel mass / Cost per bay	2413 / 2899	2688 / 3207	3049 / 3604	4150 / 4473	4442 / 4763	4696 / 5013

+ See Section 6.20 for explanation.

Table 6.3
STUB-GIRDER FLOOR BAY+
Trial Selection Table

4-STUB CONFIGURATION



Live Load: 2.4 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
10000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 4/1/5	13720 8/2/7	14720 14/4/9	15860 18/5/11	17660 8/3/9
	% Utilization: Mid span/Support	33 / 64	42 / 73	53 / 83	64 / 92	66 / 86	46 / 77
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 12/1/1	5280 12/2/2	6280 18/3/3	7280 19/3/4	8140 17/4/5	7340 24/4/5
	% Utilization: Mid span	43	45	54	62	64	63
	S-G bottom chord size (Studs)	W310X45 (52)	W310X52 (56)	W310X60 (64)	W310X67 (68)	W310X67 (72)	W310X67 (68)
	Length: Hole 1/Ext stub, (Studs)	710/1200 (20)	710/1200 (22)	710/1200 (26)	710/1200 (28)	630/1280 (30)	710/1200 (28)
	Length: Hole 3/Int stub, (Studs)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)
	% Utilization at: 1/2/3	86 / 91 / 98	81 / 88 / 95	80 / 87 / 94	77 / 86 / 92	74 / 93 / 99	86 / 93 / 98
	% Utilization at: 4/5/6	91 / 44 / 53	61 / 46 / 59	46 / 50 / 66	48 / 52 / 72	52 / 56 / 75	53 / 56 / 71
	Defl:Shore removed+SD+LL=Total	6+5+5=16	6+5+4=15	6+5+4=15	6+5+4=15	6+6+4=16	6+5+4=15
	Steel mass / Cost per bay	1810 / 2382	2067 / 2624	2342 / 2933	2593 / 3216	3020 / 3670	3756 / 4168
10500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 4/1/5	13720 8/2/7	14720 15/4/10	15860 19/5/12	17660 8/3/9
	% Utilization: Mid span/Support	34 / 67	44 / 77	55 / 86	67 / 95	68 / 89	48 / 80
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 12/1/1	5280 13/2/2	6280 19/3/3	7280 20/4/5	8140 17/4/5	7340 25/4/5
	% Utilization: Mid span	45	47	56	65	67	66
	S-G bottom chord size (Studs)	W310X52 (56)	W310X60 (60)	W310X67 (68)	W310X74 (72)	W310X74 (76)	W310X74 (72)
	Length: Hole 1/Ext stub, (Studs)	750/1255 (22)	750/1255 (24)	750/1255 (28)	720/1285 (30)	640/1365 (32)	640/1365 (30)
	Length: Hole 3/Int stub, (Studs)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)
	% Utilization at: 1/2/3	81 / 86 / 93	80 / 86 / 93	78 / 86 / 92	73 / 85 / 91	70 / 92 / 98	73 / 92 / 97
	% Utilization at: 4/5/6	72 / 44 / 57	55 / 48 / 65	47 / 51 / 71	49 / 53 / 77	52 / 57 / 80	53 / 57 / 74
	Defl:Shore removed+SD+LL=Total	6+5+5=16	6+6+5=17	6+6+5=17	6+6+5=17	7+6+5=18	6+6+4=16
	Steel mass / Cost per bay	1919 / 2435	2191 / 2740	2459 / 3042	2717 / 3328	3145 / 3783	3896 / 4280
11000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 9/2/7	14720 15/4/10	15860 19/5/12	17660 9/3/9
	% Utilization: Mid span/Support	36 / 70	46 / 80	57 / 90	69 / 99	71 / 93	50 / 83
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 13/2/2	6280 20/3/4	7280 21/4/5	8140 18/4/5	7340 26/4/5
	% Utilization: Mid span	47	49	59	68	70	69
	S-G bottom chord size (Studs)	W310X60 (60)	W310X67 (68)	W310X74 (72)	W310X74 (76)	W310X86 (80)	W310X86 (80)
	Length: Hole 1/Ext stub, (Studs)	780/1320 (24)	780/1320 (28)	780/1320 (30)	740/1360 (32)	580/1520 (34)	660/1440 (34)
	Length: Hole 3/Int stub, (Studs)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)
	% Utilization at: 1/2/3	78 / 85 / 91	77 / 85 / 92	76 / 85 / 91	78 / 93 / 100	56 / 88 / 94	66 / 87 / 93
	% Utilization at: 4/5/6	58 / 46 / 62	46 / 50 / 69	48 / 52 / 76	52 / 57 / 82	53 / 58 / 84	53 / 57 / 79
	Defl:Shore removed+SD+LL=Total	7+6+5=18	7+6+5=18	7+6+5=18	8+7+5=20	7+6+5=18	7+6+5=18
	Steel mass / Cost per bay	2048 / 2555	2313 / 2852	2585 / 3158	2766 / 3367	3343 / 3960	4090 / 4458

+ See Section 6.20 for explanation.

4-STUB CONFIGURATION

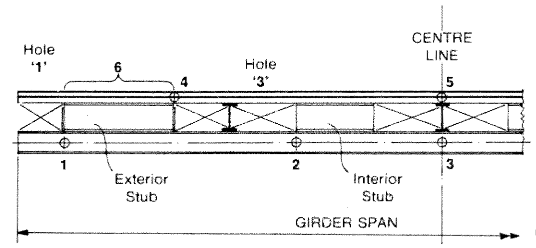


Table 6.3 (continued)
STUB-GIRDER FLOOR BAY+
Trial Selection Table

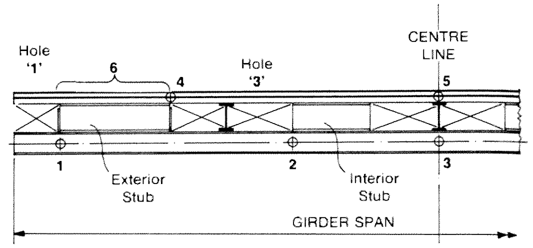
Live Load: 2.4 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
11500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 9/3/7	14560 17/4/10	15860 20/5/12	17660 9/3/10
	% Utilization: Mid span/Support	37 / 72	47 / 83	59 / 93	73 / 100	74 / 97	52 / 86
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 14/2/2	6280 21/3/4	7440 24/4/5	8140 19/4/5	7340 27/4/5
	% Utilization: Mid span	49	51	62	74	73	73
	S-G bottom chord size (Studs)	W310X60 (64)	W310X67 (72)	W310X74 (76)	W310X86 (80)	W310X97 (96)	W310X97 (84)
	Length: Hole 1/Ext stub, (Studs)	820/1385 (26)	820/1385 (30)	820/1385 (32)	680/1525 (34)	600/1605 (38)	680/1525 (36)
	Length: Hole 3/Int stub, (Studs)	920/1035 (6)	920/1035 (6)	920/1035 (6)	920/1035 (6)	920/1035 (10)	920/1035 (6)
	% Utilization at: 1/2/3	85 / 92 / 100	85 / 93 / 100	83 / 93 / 100	63 / 88 / 95	54 / 86 / 92	63 / 86 / 92
	% Utilization at: 4/5/6	57 / 51 / 67	50 / 54 / 74	52 / 57 / 81	53 / 59 / 85	55 / 61 / 90	55 / 60 / 85
	Defl:Shore removed+SD+LL=Total	8+7+6=21	8+7+6=21	8+7+6=21	8+7+6=21	8+7+5=20	7+6+5=18
	Steel mass / Cost per bay	2089 / 2586	2357 / 2887	2633 / 3197	2966 / 3548	3534 / 4138	4285 / 4636
12000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 5/2/5	13720 10/3/8	14860 13/3/9	17160 7/3/8	17660 9/3/10
	% Utilization: Mid span/Support	38 / 75	49 / 86	61 / 96	63 / 91	49 / 85	54 / 89
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W250X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 14/1/2	5280 14/2/2	6280 22/3/4	7140 22/4/5	6840 22/3/4	7340 28/4/5
	% Utilization: Mid span	51	53	64	72	66	76
	S-G bottom chord size (Studs)	W310X67 (68)	W310X74 (76)	W310X86 (80)	W310X97 (96)	W310X97 (88)	W310X97 (88)
	Length: Hole 1/Ext stub, (Studs)	850/1450 (28)	850/1450 (32)	780/1520 (34)	700/1600 (38)	700/1600 (38)	620/1680 (38)
	Length: Hole 3/Int stub, (Studs)	960/1080 (6)	960/1080 (6)	960/1080 (6)	960/1080 (10)	960/1080 (6)	960/1080 (6)
	% Utilization at: 1/2/3	82 / 91 / 98	81 / 92 / 98	69 / 88 / 94	60 / 87 / 93	65 / 90 / 96	60 / 94 /99
	% Utilization at: 4/5/6	49 / 53 / 71	51 / 56 / 79	53 / 58 / 85	55 / 61 / 90	57 / 63 / 87	60 / 66 / 88
	Defl:Shore removed+SD+LL=Total	9+8+7=24	9+8+6=23	9+8+6=23	9+8+6=23	8+7+6=21	9+8+6=23
	Steel mass / Cost per bay	2218 / 2706	2490 / 3010	2839 / 3387	3423 / 3994	4254 / 4581	4363 / 4687
12500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W460X61 (30)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 5/2/5	13720 10/3/8	16160 4/2/6	17160 7/3/9	17660 10/3/10
	% Utilization: Mid span/Support	40 / 77	51 / 89	63 / 100	42 / 80	51 / 88	56 / 93
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W200X27 (14)	W250X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 15/2/2	6280 22/3/4	5840 24/3/4	6840 23/3/4	7340 29/4/6
	% Utilization: Mid span	53	56	67	68	69	79
	S-G bottom chord size (Studs)	W310X74 (72)	W310X86 (80)	W310X97 (96)	W310X97 (88)	W310X107 (104)	W310X107 (108)
	Length: Hole 1/Ext stub, (Studs)	890/1505 (30)	890/1505 (34)	790/1605 (38)	790/1605 (38)	630/1765 (40)	630/1765 (42)
	Length: Hole 3/Int stub, (Studs)	1000/1125 (6)	1000/1125 (6)	1000/1125 (10)	1000/1125 (6)	1000/1125 (12)	1000/1125 (12)
	% Utilization at: 1/2/3	80 / 89 / 96	74 / 87 / 93	65 / 87 / 93	71 / 90 / 96	55 / 89 / 95	57 / 93 / 98
	% Utilization at: 4/5/6	50 / 55 / 75	52 / 57 / 84	55 / 61 / 90	57 / 63 / 87	60 / 66 / 90	62 / 68 / 94
	Defl:Shore removed+SD+LL=Total	9+8+7=24	9+8+7=24	9+8+7=24	9+8+6=23	9+8+6=23	9+8+6=23
	Steel mass / Cost per bay	2354 / 2833	2695 / 3207	3039 / 3576	4040 / 4342	4464 / 4763	4563 / 4869

+ See Section 6.20 for explanation.

Table 6.4
STUB-GIRDER FLOOR BAY[+]
Trial Selection Table

4-STUB CONFIGURATION



Live Load: 3.6 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
10000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/7	13600 10/2/10	14200 20/4/14	15600 22/5/17	17660 9/3/13
	% Utilization: Mid span/Support	45 / 80	57 / 91	71 / 100	88 / 100	85 / 100	62 / 94
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W360X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 14/2/3	6400 14/2/4	7800 15/3/6	8400 21/4/8	7340 27/4/7
	% Utilization: Mid span	56	57	61	74	86	80
	S-G bottom chord size (Studs)	W310X60 (56)	W310X67 (60)	W310X74 (64)	W310X79 (68)	W310X86 (72)	W310X86 (72)
	Length: Hole 1/Ext stub, (Studs)	710/1200 (22)	710/1200 (24)	710/1200 (26)	630/1280 (28)	550/1360 (30)	630/1280 (30)
	Length: Hole 3/Int stub, (Studs)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)
	% Utilization at: 1/2/3	79 / 86 / 93	78 / 86 / 93	76 / 86 / 92	68 / 88 / 95	58 / 88 / 94	69 / 88 / 93
	% Utilization at: 4/5/6	84 / 58 / 57	59 / 62 / 63	61 / 65 / 69	65 / 69 / 73	67 / 71 / 76	67 / 71 / 73
	Defl:Shore removed+SD+LL=Total	5+4+5=14	5+4+5=14	5+4+5=14	6+4+5=15	6+4+5=15	5+4+5=14
	Steel mass / Cost per bay	1967 / 2492	2224 / 2782	2514 / 3110	2720 / 3334	3220 / 3861	3966 / 4367
10500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/7	13420 11/3/10	14040 22/5/15	15440 25/5/17	17660 9/3/14
	% Utilization: Mid span/Support	47 / 83	59 / 95	75 / 100	92 / 100	90 / 100	64 / 98
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W360X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 14/1/2	5280 14/2/3	6580 15/2/5	7960 16/4/7	8560 23/5/9	7340 28/4/7
	% Utilization: Mid span	58	60	68	81	93	84
	S-G bottom chord size (Studs)	W310X67 (60)	W310X74 (64)	W310X79 (72)	W310X86 (76)	W310X97 (80)	W310X97 (76)
	Length: Hole 1/Ext stub, (Studs)	750/1255 (24)	750/1255 (26)	720/1285 (30)	640/1365 (32)	560/1445 (34)	640/1365 (32)
	Length: Hole 3/Int stub, (Studs)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)
	% Utilization at: 1/2/3	77 / 85 / 92	77 / 86 / 92	75 / 89 / 95	66 / 89 / 96	55 / 87 / 93	66 / 87 / 92
	% Utilization at: 4/5/6	65 / 60 / 61	60 / 64 / 68	64 / 69 / 74	67 / 72 / 78	69 / 75 / 82	68 / 73 / 78
	Defl:Shore removed+SD+LL=Total	6+4+6=16	6+5+6=17	6+5+6=17	6+5+6=17	6+5+6=17	6+4+5=15
	Steel mass / Cost per bay	2085 / 2601	2345 / 2894	2617 / 3201	2847 / 3450	3396 / 4023	4150 / 4534
11000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/7	13260 13/3/11	14660 15/3/13	16020 17/4/16	17540 10/3/14
	% Utilization: Mid span/Support	49 / 86	62 / 99	78 / 100	77 / 100	79 / 100	67 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 15/2/3	6740 17/3/5	7340 25/4/7	7980 20/4/7	7460 31/4/8
	% Utilization: Mid span	61	63	74	88	85	91
	S-G bottom chord size (Studs)	W310X67 (64)	W310X79 (68)	W310X86 (76)	W310X97 (80)	W310X107 (92)	W310X107 (80)
	Length: Hole 1/Ext stub, (Studs)	780/1320 (26)	780/1320 (28)	740/1360 (32)	660/1440 (34)	500/1600 (36)	580/1520 (34)
	Length: Hole 3/Int stub, (Studs)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (10)	880/990 (6)
	% Utilization at: 1/2/3	84 / 93 / 100	77 / 89 / 95	73 / 90 / 96	63 / 88 / 94	46 / 87 / 93	56 / 87 / 92
	% Utilization at: 4/5/6	71 / 66 / 65	64 / 68 / 73	67 / 72 / 78	70 / 75 / 83	71 / 78 / 81	71 / 77 / 81
	Defl:Shore removed+SD+LL=Total	7+5+7=19	7+5+7=19	7+5+7=19	7+5+6=18	7+5+6=18	6+5+6=17
	Steel mass / Cost per bay	2129 / 2636	2451 / 2990	2748 / 3321	3291 / 3885	3913 / 4271	4336 / 4693

[+] See Section 6.20 for explanation.

4-STUB CONFIGURATION

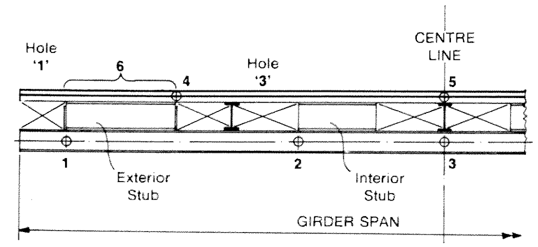


Table 6.4 (continued)
STUB-GIRDER FLOOR BAY[+]
Trial Selection Table

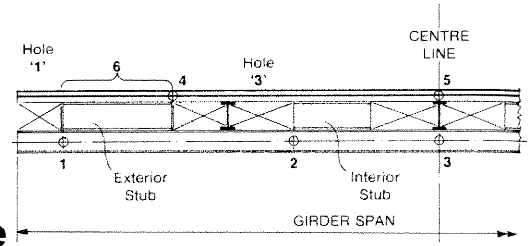
Live Load: 3.6 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
11500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12600 6/2/8	13120 14/3/11	14500 16/4/13	15840 20/5/16	17300 12/3/14
	% Utilization: Mid span/Support	50 / 90	64 / 100	82 / 100	80 / 100	83 / 100	70 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (14)	W360X33 (14)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5400 16/2/4	6880 19/3/6	7500 28/4/8	8160 22/4/8	7700 35/5/9
	% Utilization: Mid span	64	69	81	96	93	96
	S-G bottom chord size (Studs)	W310X74 (68)	W310X86 (76)	W310X97 (80)	W310X107 (92)	W310X118 (108)	W310X107 (88)
	Length: Hole 1/Ext stub, (Studs)	820/1385 (28)	820/1385 (32)	760/1445 (34)	600/1605 (36)	520/1685 (38)	600/1605 (38)
	Length: Hole 3/Int stub, (Studs)	920/1035 (6)	920/1035 (6)	920/1035 (6)	920/1035 (10)	920/1035 (16)	920/1035 (6)
	% Utilization at: 1/2/3	82 / 92 / 99	76 / 89 / 95	70 / 88 / 95	54 / 88 / 94	46 / 87 / 92	60 / 94 / 100
	% Utilization at: 4/5/6	64 / 68 / 69	66 / 71 / 77	70 / 76 / 83	72 / 79 / 85	74 / 82 / 86	77 / 84 / 86
	Defl:Shore removed+SD+LL=Total	8+6+7=21	7+6+7=20	8+6+7=21	7+6+7=20	7+6+7=20	7+6+7=20
	Steel mass / Cost per bay	2258 / 2756	2584 / 3113	2935 / 3497	3479 / 4052	4109 / 4451	4396 / 4734
12000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12440 7/2/8	12980 16/3/11	14340 18/4/13	16860 9/3/12	17080 14/4/15
	% Utilization: Mid span/Support	52 / 93	67 / 100	85 / 99	84 / 100	67 / 100	73 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (16)	W250X33 (14)	W360X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 16/1/2	5560 19/2/4	7020 22/4/7	7660 32/5/10	7140 28/4/7	7920 23/4/7
	% Utilization: Mid span	67	76	88	99	91	91
	S-G bottom chord size (Studs)	W310X86 (72)	W310X97 (80)	W310X107 (96)	W310X118 (112)	W310X118 (100)	W310X118 (100)
	Length: Hole 1/Ext stub, (Studs)	850/1450 (30)	850/1450 (34)	700/1600 (36)	620/1680 (40)	620/1680 (40)	540/1760 (40)
	Length: Hole 3/Int stub, (Studs)	960/1080 (6)	960/1080 (6)	960/1080 (12)	960/1080 (16)	960/1080 (10)	960/1080 (10)
	% Utilization at: 1/2/3	74 / 87 / 93	74 / 88 / 94	60 / 88 / 94	53 / 87 / 93	57 / 90 / 96	51 / 94 / 99
	% Utilization at: 4/5/6	65 / 70 / 74	69 / 75 / 82	72 / 79 / 86	76 / 83 / 90	78 / 85 / 87	80 / 88 / 88
	Defl:Shore removed+SD+LL=Total	8+6+8=22	8+6+8=22	8+6+8=22	8+6+7=21	8+6+7=21	8+6+7=21
	Steel mass / Cost per bay	2458 / 2946	2773 / 3292	3127 / 3672	3682 / 4242	4511 / 4827	4604 / 4915
12500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W460X61 (24)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12280 8/2/8	12860 17/4/12	16160 4/2/9	16640 11/3/13	16880 16/4/15
	% Utilization: Mid span/Support	54 / 96	70 / 100	89 / 99	57 / 98	69 / 100	77 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W200X27 (12)	W250X33 (16)	W360X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 17/1/3	5720 21/3/5	7140 24/4/8	5840 27/3/5	7360 32/5/9	8120 26/5/9
	% Utilization: Mid span	69	83	95	87	96	100
	S-G bottom chord size (Studs)	W310X97 (76)	W310X107 (96)	W310X118 (112)	W310X118 (100)	W310X129 (120)	W310X129 (124)
	Length: Hole 1/Ext stub, (Studs)	890/1505 (32)	870/1525 (36)	710/1685 (40)	710/1685 (40)	550/1845 (42)	550/1845 (44)
	Length: Hole 3/Int stub, (Studs)	1000/1125 (6)	1000/1125 (12)	1000/1125 (16)	1000/1125 (10)	1000/1125 (18)	1000/1125 (18)
	% Utilization at: 1/2/3	72 / 85 / 92	70 / 86 / 93	57 / 87 / 93	62 / 91 / 96	47 / 90 / 95	49 / 93 / 98
	% Utilization at: 4/5/6	67 / 73 / 79	71 / 78 / 86	75 / 83 / 90	78 / 85 / 87	80 / 89 / 90	83 / 92 / 93
	Defl:Shore removed+SD+LL=Total	9+7+8=24	9+7+8=24	9+7+8=24	9+7+8=24	8+7+7=22	9+7+8=24
	Steel mass / Cost per bay	2656 / 3135	2961 / 3469	3337 / 3870	4327 / 4618	4733 / 5019	4816 / 5108

[+] See Section 6.20 for explanation.

Table 6.5
STUB-GIRDER FLOOR BAY+
Trial Selection Table

4-STUB CONFIGURATION



Live Load: 3.6 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
10000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 4/1/7	13720 8/2/10	14340 17/4/14	15780 18/5/17	17660 8/3/13
	% Utilization: Mid span/Support	43 / 77	54 / 88	66 / 99	82 / 100	82 / 100	58 / 91
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 12/1/2	5280 12/2/3	6280 18/3/5	7660 23/4/8	8220 17/4/7	7340 24/4/7
	% Utilization: Mid span	52	54	65	83	78	76
	S-G bottom chord size (Studs)	W310X52 (60)	W310X60 (64)	W310X67 (72)	W310X74 (76)	W310X79 (80)	W310X79 (76)
	Length: Hole 1/Ext stub, (Studs)	710/1200 (24)	710/1200 (26)	630/1280 (30)	550/1360 (32)	470/1440 (34)	470/1440 (32)
	Length: Hole 3/Int stub, (Studs)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)	800/900 (6)
	% Utilization at: 1/2/3	85 / 92 / 99	84 / 92 / 99	73 / 91 / 98	61 / 90 /96	53 / 92 / 98	55 / 92 / 97
	% Utilization at: 4/5/6	63 / 48 / 62	48 / 52 / 69	51 / 55 / 74	52 / 57 / 79	55 / 60 / 83	55 / 60 / 76
	Defl:Shore removed+SD+LL=Total	5+4+6=15	5+5+6=16	5+5+6=16	5+5+6=16	5+5+6=16	5+4+5=14
	Steel mass / Cost per bay	1883 / 2408	2151 / 2708	2423 / 3007	2676 / 3284	3160 / 3794	3914 / 4294
10500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 4/1/7	13600 9/2/10	14180 18/4/15	15580 21/5/17	17660 8/3/14
	% Utilization: Mid span/Support	44 / 80	56 / 92	69 / 100	86 / 100	86 / 100	61 / 94
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W360X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 12/1/2	5280 13/2/3	6400 13/2/4	7820 14/3/6	8420 19/4/8	7340 25/4/7
	% Utilization: Mid span	54	56	61	74	86	80
	S-G bottom chord size (Studs)	W310X60 (64)	W310X67 (68)	W310X74 (76)	W310X79 (80)	W310X86 (84)	W310X86 (80)
	Length: Hole 1/Ext stub, (Studs)	750/1255 (26)	750/1255 (28)	640/1365 (32)	560/1445 (34)	400/1605 (36)	480/1525 (34)
	Length: Hole 3/Int stub, (Studs)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)	840/945 (6)
	% Utilization at: 1/2/3	84 / 90 / 97	82 / 91 / 97	69 / 90 / 97	61 / 93 / 99	42 / 92 / 99	53 / 92 / 98
	% Utilization at: 4/5/6	57 / 49 / 68	48 / 53 / 75	51 / 55 / 79	54 / 59 / 85	55 / 61 / 87	55 / 61 / 82
	Defl:Shore removed+SD+LL=Total	6+5+7=18	6+5+7=18	6+5+6=17	6+5+6=17	6+5+6=17	6+5+6=17
	Steel mass / Cost per bay	2007 / 2523	2268 / 2817	2571 / 3148	2779 / 3375	3294 / 3906	4050 / 4413
11000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/8	13420 10/3/11	14040 20/5/15	15420 23/6/18	17660 9/3/14
	% Utilization: Mid span/Support	46 / 83	58 / 95	73 / 100	90 / 100	90 / 100	63 / 98
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W360X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 13/2/3	6580 14/3/5	7960 15/4/7	8580 22/5/9	7340 26/4/7
	% Utilization: Mid span	57	59	67	81	93	84
	S-G bottom chord size (Studs)	W310X67 (68)	W310X74 (76)	W310X79 (80)	W310X86 (84)	W310X97 (100)	W310X97 (88)
	Length: Hole 1/Ext stub, (Studs)	780/1320 (28)	740/1360 (32)	660/1440 (34)	500/1600 (36)	420/1680 (40)	420/1680 (38)
	Length: Hole 3/Int stub, (Studs)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (6)	880/990 (10)	880/990 (6)
	% Utilization at: 1/2/3	81 / 89 / 96	76 / 90 / 96	69 / 93 / 100	51 / 93 / 100	42 / 91 / 97	43 / 91 / 96
	% Utilization at: 4/5/6	47 / 51 / 72	50 / 54 / 79	53 / 58 / 85	55 / 61 / 88	57 / 63 / 93	57 / 63 / 86
	Defl:Shore removed+SD+LL=Total	6+5+7=18	6+6+7=19	7+6+7=20	7+6+7=20	6+6+7=19	6+5+6=17
	Steel mass / Cost per bay	2129 / 2636	2397 / 2933	2676 / 3242	2917 / 3495	3474 / 4073	4249 / 4585

+ See Section 6.20 for explanation.

4-STUB CONFIGURATION

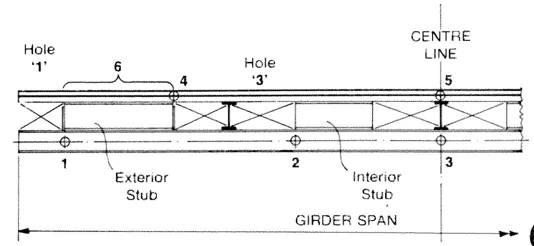


Table 6.5 (continued)
STUB-GIRDER FLOOR BAY+
Trial Selection Table

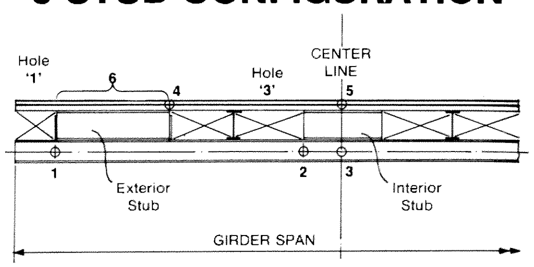
Live Load: 3.6 kPa
Cover slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
11500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/8	13260 12/3/11	13900 22/5/15	15280 25/6/18	17540 9/3/14
	% Utilization: Mid span/Support	47 / 86	60 / 99	76 / 100	94 / 100	94 / 100	65 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W360X33 (16)	W360X33 (18)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 14/2/3	6740 16/3/6	8100 17/4/8	8720 24/5/10	7460 28/4/8
	% Utilization: Mid span	59	62	74	87	97	90
	S-G bottom chord size (Studs)	W310X74 (72)	W310X79 (80)	W310X97 (92)	W310X97 (100)	W310X107 (116)	W310X107 (104)
	Length: Hole 1/Ext stub, (Studs)	820/1385 (30)	760/1445 (34)	600/1605 (36)	520/1685 (40)	440/1765 (42)	440/1765 (40)
	Length: Hole 3/Int stub, (Studs)	920/1035 (6)	920/1035 (6)	920/1035 (10)	920/1035 (10)	920/1035 (16)	920/1035 (12)
	% Utilization at: 1/2/3	79 / 88 / 95	75 / 92 / 99	53 / 85 / 91	50 / 92 / 99	41 / 91 / 97	42 / 90 / 96
	% Utilization at: 4/5/6	49 / 53 / 77	52 / 58 / 84	53 / 59 / 89	58 / 64 / 94	60 / 67 / 99	59 / 66 / 91
	Defl:Shore removed+SD+LL=Total	7+6+8=21	7+6+8=21	7+6+7=20	7+6+7=20	7+6+7=20	7+6+7=20
	Steel mass / Cost per bay	2258 / 2756	2507 / 3032	2950 / 3499	3104 / 3671	3655 / 4240	4431 / 4749
12000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12600 5/2/8	13120 13/3/11	14500 15/4/14	17120 7/3/13	17300 11/4/15
	% Utilization: Mid span/Support	49 / 90	62 / 100	79 / 100	81 / 100	62 / 100	68 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W250X33 (16)	W250X33 (16)	W250X33 (18)
	Length; Defl: Slab pour/SD/LL	4280 14/1/2	5400 15/2/4	6880 18/3/6	7500 26/5/9	6880 23/3/6	7700 32/5/10
	% Utilization: Mid span	62	67	80	95	80	96
	S-G bottom chord size (Studs)	W310X79 (80)	W310X86 (84)	W310X97 (100)	W310X107 (116)	W310X118 (120)	W310X118 (124)
	Length: Hole 1/Ext stub, (Studs)	850/1450 (34)	780/1520 (36)	620/1680 (40)	540/1760 (42)	460/1840 (42)	460/1840 (44)
	Length: Hole 3/Int stub, (Studs)	960/1080 (6)	960/1080 (6)	960/1080 (10)	960/1080 (16)	960/1080 (18)	960/1080 (18)
	% Utilization at: 1/2/3	79 / 90 / 97	72 / 92 / 99	56 / 92 / 98	48 / 91 / 97	40 / 87 / 92	42 / 90 / 95
	% Utilization at: 4/5/6	51 / 56 / 82	54 / 60 / 89	57 / 64 / 94	60 / 67 / 100	60 / 66 / 94	62 / 69 / 97
	Defl:Shore removed+SD+LL=Total	8+7+8=23	8+7+8=23	8+7+8=23	8+7+8=23	7+6+7=20	7+6+7=20
	Steel mass / Cost per bay	2370 / 2857	2644 / 3158	3010 / 3548	3556 / 4108	4548 / 4843	4628 / 4929
12500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W460X61 (30)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12440 6/2/8	13000 14/3/12	16160 4/2/10	16880 9/3/13	17100 13/4/15
	% Utilization: Mid span/Support	51 / 93	65 / 100	82 / 100	54 / 94	65 / 100	71 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W200X27 (14)	W250X33 (16)	W360X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 15/1/3	5560 17/2/4	7000 20/4/7	5840 24/3/5	7120 26/4/8	7900 21/4/8
	% Utilization: Mid span	64	74	86	82	89	90
	S-G bottom chord size (Studs)	W310X86 (84)	W310X97 (100)	W310X107 (116)	W310X118 (120)	W310X118 (128)	W310X129 (144)
	Length: Hole 1/Ext stub, (Studs)	870/1525 (36)	710/1685 (40)	630/1765 (42)	550/1845 (42)	470/1925 (46)	340/2055 (48)
	Length: Hole 3/Int stub, (Studs)	1000/1125 (6)	1000/1125 (10)	1000/1125 (16)	1000/1125 (18)	1000/1125 (18)	1000/1125 (24)
	% Utilization at: 1/2/3	76 / 90 / 97	61 / 91 / 97	54 / 91 / 97	46 / 87 / 92	43 / 94 / 99	29 / 89 / 94
	% Utilization at: 4/5/6	53 / 59 / 87	57 / 63 / 93	60 / 67 / 99	60 / 67 / 94	65 / 72 / 99	64 / 72 / 100
	Defl:Shore removed+SD+LL=Total	8+7+9=24	8+7+9=24	9+8+9=26	8+7+8=23	8+7+8=23	8+7+8=23
	Steel mass / Cost per bay	2513 / 2991	2848 / 3343	3202 / 3728	4348 / 4618	4613 / 4890	4857 / 5122

+ See Section 6.20 for explanation.

Table 6.6
STUB-GIRDER FLOOR BAY+
Trial Selection Table

3-STUB CONFIGURATION



Live Load: 2.4 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
8000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 10/2/7	14660 17/4/10	15860 21/5/12	17660 9/3/9
	% Utilization: Mid span/Support	37 / 71	48 / 81	59 / 91	73 / 100	72 / 95	52 / 84
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X31 (14)	W250X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 14/1/1	5280 14/2/2	6280 22/3/3	7340 24/4/5	8140 20/4/5	7340 29/4/5
	% Utilization: Mid span	50	51	61	71	72	71
	S-G bottom chord size (Studs)	W310X45 (34)	W310X45 (38)	W310X45 (42)	W310X45 (42)	W310X45 (46)	W310X45 (46)
	Length: Hole 1/Ext stub, (Studs)	570/1456 (14)	570/1456 (16)	570/1456 (18)	570/1456 (18)	570/1456 (20)	570/1456 (20)
	Length: Hole 3/Int stub, (Studs)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)
	% Utilization at: 1/2/3	53 / 62 / 49	59 / 69 / 54	65 / 76 / 59	71 / 83 / 65	77 / 89 / 70	79 / 88 / 67
	% Utilization at: 4/5/6	43 / 20 / 34	48 / 22 / 38	52 / 24 / 41	57 / 27 / 45	62 / 29 / 48	84 / 28 / 45
	Defl:Shore removed+SD+LL=Total	3+2+2=7	3+2+2=7	3+3+2=8	4+3+2=9	4+3+3=10	4+3+2=9
	Steel mass / Cost per bay	1420 / 1812	1558 / 1974	1701 / 2143	1834 / 2299	2155 / 2641	2728 / 3014
8500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 10/2/7	14860 14/3/9	16780 13/3/11	17660 10/3/9
	% Utilization: Mid span/Support	39 / 74	50 / 85	62 / 96	63 / 91	63 / 100	55 / 89
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X31 (14)	W250X33 (14)	W250X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 15/2/2	6280 23/3/4	7140 24/4/5	7220 28/4/5	7340 30/4/5
	% Utilization: Mid span	53	54	65	72	73	76
	S-G bottom chord size (Studs)	W310X45 (38)	W310X45 (42)	W310X45 (42)	W310X45 (46)	W310X60 (50)	W310X45 (50)
	Length: Hole 1/Ext stub, (Studs)	600/1553 (16)	600/1553 (18)	600/1553 (18)	600/1553 (20)	600/1553 (22)	600/1553 (22)
	Length: Hole 3/Int stub, (Studs)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)
	% Utilization at: 1/2/3	59 / 70 / 55	66 / 77 / 61	72 / 85 / 67	79 / 92 / 72	64 / 78 / 59	88 / 98 / 75
	% Utilization at: 4/5/6	47 / 22 / 37	53 / 24 / 41	58 / 26 / 45	63 / 28 / 48	62 / 31 / 51	100 / 30 / 49
	Defl:Shore removed+SD+LL=Total	3+3+2=8	4+3+3=10	4+3+3=10	4+3+3=10	4+3+3=10	5+4+3=12
	Steel mass / Cost per bay	1490 / 1838	1627 / 2000	1771 / 2169	2109 / 2520	2643 / 2839	2824 / 3040
9000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 6/2/5	13680 11/3/8	14860 15/3/9	16480 16/4/11	17660 10/3/10
	% Utilization: Mid span/Support	41 / 78	52 / 89	66 / 100	66 / 95	67 / 100	57 / 93
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (14)	W250X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 16/1/2	5280 16/2/2	6320 16/2/3	7140 25/4/5	7520 32/5/6	7340 32/4/5
	% Utilization: Mid span	56	57	60	76	84	80
	S-G bottom chord size (Studs)	W310X45 (42)	W310X45 (46)	W310X45 (46)	W310X52 (50)	W310X60 (54)	W310X52 (54)
	Length: Hole 1/Ext stub, (Studs)	640/1640 (18)	640/1640 (20)	640/1640 (20)	640/1640 (22)	640/1640 (24)	640/1640 (24)
	Length: Hole 3/Int stub, (Studs)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)
	% Utilization at: 1/2/3	67 / 77 / 61	74 / 86 / 68	81 / 94 / 74	75 / 89 / 69	72 / 86 / 66	84 / 95 / 72
	% Utilization at: 4/5/6	55 / 23 / 40	61 / 25 / 44	67 / 28 / 48	64 / 30 / 52	68 / 33 / 55	70 / 32 / 53
	Defl:Shore removed+SD+LL=Total	4+3+3=10	5+4+3=12	5+4+4=13	5+4+3=12	5+4+3=12	5+4+3=12
	Steel mass / Cost per bay	1489 / 1864	1627 / 2026	1790 / 2218	2170 / 2570	2631 / 2859	2876 / 3090

+ See Section 6.20 for explanation.

3-STUB CONFIGURATION

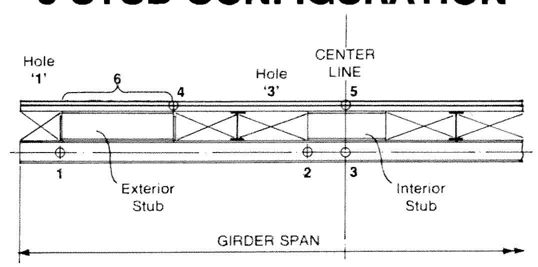


Table 6.6 (continued)
STUB-GIRDER FLOOR BAY+
Trial Selection Table

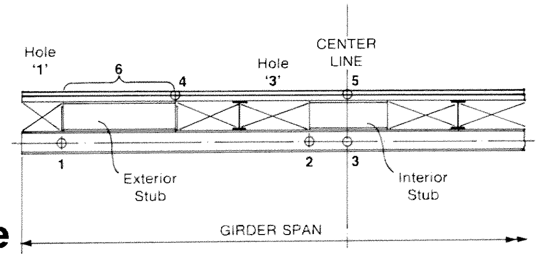
Live Load: 2.4 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
9500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X 46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 6/2/5	13460 13/3/8	14860 15/4/9	16220 18/4/11	17660 11/3/10
	% Utilization: Mid span/Support	43 / 82	55 / 94	70 / 100	69 / 100	71 / 100	60 / 98
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 17/1/2	5280 17/2/2	6540 18/3/4	7140 26/4/5	7780 22/4/5	7340 34/5/6
	% Utilization: Mid span	59	60	67	80	78	84
	S-G bottom chord size (Studs)	W310X45 (42)	W310X45 (46)	W310X52 (50)	W310X52 (54)	W310X60 (62)	W310X60 (58)
	Length: Hole 1/Ext stub, (Studs)	670/1736 (18)	670/1736 (20)	670/1736 (22)	670/1736 (24)	670/1736 (28)	610/1796 (26)
	Length: Hole 3/Int stub, (Studs)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)
	% Utilization at: 1/2/3	73 / 85 / 68	81 / 94 / 75	76 / 90 / 70	83 / 98 / 76	80 / 95 / 73	75 / 94 / 71
	% Utilization at: 4/5/6	77 / 24 / 43	86 / 26 / 47	63 / 29 / 52	68 / 32 / 56	72 / 35 / 59	72 / 33 / 56
	Defl:Shore removed+SD+LL=Total	5+4+4=13	6+4+4=14	5+4+4=13	6+5+4=15	6+5+4=15	5+4+3=12
	Steel mass / Cost per bay	1524 / 1890	1662 / 2052	1893 / 2266	2211 / 2597	2669 / 2882	3009 / 3197
10000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 3/1/4	12720 6/2/6	13860 9/2/7	15540 10/3/8	17160 9/3/9	17480 13/4/10
	% Utilization: Mid span/Support	45 / 85	57 / 98	59 / 94	60 / 100	57 / 98	63 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X27 (22)	W250X33 (14)	W250X33 (14)	W250X33 (26)
	Length; Defl: Slab pour/SD/LL	4280 18/1/2	5280 18/2/3	6140 31/4/5	6460 22/3/4	6840 28/4/5	7520 38/5/7
	% Utilization: Mid span	62	63	67	69	77	74
	S-G bottom chord size (Studs)	W310X52 (46)	W310X52 (50)	W310X52 (54)	W310X60 (62)	W310X60 (62)	W310X67 (62)
	Length: Hole 1/Ext stub, (Studs)	710/1823 (20)	710/1823 (22)	710/1823 (24)	710/1823 (28)	600/1933 (28)	580/1953 (28)
	Length: Hole 3/Int stub, (Studs)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)
	% Utilization at: 1/2/3	69 / 81 / 64	77 / 90 / 71	84 / 98 / 77	82 / 96 / 75	75 / 99 / 76	67 / 93 / 70
	% Utilization at: 4/5/6	55 / 25 / 46	61 / 27 / 51	70 / 30 / 55	70 / 33 / 59	79 / 33 / 58	73 / 35 / 59
	Defl:Shore removed+SD+LL=Total	5+4+4=13	6+5+4=15	7+5+4=16	7+5+4=16	6+5+4=15	6+5+4=15
	Steel mass / Cost per bay	1672 / 1942	1810 / 2104	2116 / 2410	2645 / 2778	3053 / 3149	3195 / 3293
10500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 3/1/4	12600 7/2/6	13860 10/3/7	16160 5/2/7	17020 10/3/9	17220 15/4/10
	% Utilization: Mid span/Support	46 / 89	60 / 100	61 / 98	49 / 92	60 / 100	67 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W200X27 (22)	W200X27 (12)	W250X33 (14)	W360X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 18/2/2	5400 20/2/3	6140 32/4/5	5840 30/3/4	6980 31/4/5	7780 26/4/5
	% Utilization: Mid span	65	70	70	81	85	86
	S-G bottom chord size (Studs)	W310X52 (50)	W310X52 (54)	W310X60 (62)	W310X60 (62)	W310X67 (66)	W310X74 (66)
	Length: Hole 1/Ext stub, (Studs)	750/1910 (22)	750/1910 (24)	750/1910 (28)	740/1920 (28)	750/1910 (30)	730/1930 (30)
	Length: Hole 3/Int stub, (Studs)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)
	% Utilization at: 1/2/3	76 / 88 / 70	85 / 98 / 78	83 / 97 / 76	89 / 100 / 77	87 / 98 / 74	78 / 92 / 69
	% Utilization at: 4/5/6	68 / 26 / 49	75 / 29 / 54	69 / 32 / 60	99 / 32 / 59	74 / 35 / 64	73 / 36 / 66
	Defl:Shore removed+SD+LL=Total	6+5+5=16	7+6+5=18	7+6+5=18	7+6+5=18	7+6+4=17	7+5+4=16
	Steel mass / Cost per bay	1670 / 1970	1805 / 2129	2196 / 2525	2822 / 2963	3094 / 3251	3235 / 3393

+ See Section 6.20 for explanation.

Table 6.7
STUB-GIRDER FLOOR BAY[+]
Trial Selection Table

3-STUB CONFIGURATION



Live Load: 2.4 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
8000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 4/1/5	13720 9/2/7	14720 15/4/10	15860 19/5/12	17660 8/3/9
	% Utilization: Mid span/Support	35 / 68	45 / 78	55 / 87	68 / 97	69 / 91	49 / 81
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 12/1/1	5280 13/2/2	6280 19/3/3	7280 21/4/5	8140 18/4/5	7340 25/4/5
	% Utilization: Mid span	46	48	57	66	68	67
	S-G bottom chord size (Studs)	W310X45 (38)	W310X45 (42)	W310X45 (42)	W310X45 (46)	W310X45 (50)	W310X45 (50)
	Length: Hole 1/Ext stub, (Studs)	570/1456 (16)	570/1456 (18)	570/1456 (18)	570/1456 (20)	570/1456 (22)	570/1456 (22)
	Length: Hole 3/Int stub, (Studs)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)
	% Utilization at: 1/2/3	51 / 60 / 46	57 / 66 / 51	62 / 73 / 56	68 / 79 / 61	73 / 85 / 66	76 / 84 / 64
	% Utilization at: 4/5/6	33 / 17 / 38	37 / 19 / 42	41 / 20 / 46	44 / 22 / 50	48 / 24 / 54	48 / 23 / 49
	Defl:Shore removed+SD+LL=Total	2+2+2=6	3+2+2=7	3+3+2=8	3+3+3=9	3+3+3=9	3+3+2=8
	Steel mass / Cost per bay	1420 / 1812	1558 / 1974	1701 / 2143	1834 / 2300	2155 / 2641	2728 / 3014
8500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/3	12720 5/1/5	13720 9/3/7	14620 17/4/10	15860 20/5/12	17660 9/3/10
	% Utilization: Mid span/Support	37 / 71	47 / 82	58 / 92	71 / 100	73 / 95	51 / 85
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 14/2/2	6280 20/3/4	7380 23/4/5	8140 19/4/5	7340 27/4/5
	% Utilization: Mid span	48	50	61	72	72	71
	S-G bottom chord size (Studs)	W310X45 (42)	W310X45 (46)	W310X45 (46)	W310X45 (50)	W310X45 (54)	W310X45 (54)
	Length: Hole 1/Ext stub, (Studs)	600/1553 (18)	600/1553 (20)	600/1553 (20)	600/1553 (22)	600/1553 (24)	600/1553 (24)
	Length: Hole 3/Int stub, (Studs)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)
	% Utilization at: 1/2/3	57 / 67 / 52	63 / 74 / 58	69 / 81 / 63	75 / 88 / 69	81 / 95 / 74	84 / 94 / 71
	% Utilization at: 4/5/6	37 / 18 / 41	41 / 20 / 45	45 / 22 / 50	49 / 24 / 54	53 / 26 / 58	53 / 25 / 54
	Defl:Shore removed+SD+LL=Total	3+3+3=9	3+3+3=9	4+3+3=10	4+4+3=11	4+4+3=11	4+4+3=11
	Steel mass / Cost per bay	1490 / 1838	1627 / 2000	1771 / 2169	1903 / 2325	2233 / 2667	2824 / 3040
9000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W410X54 (26)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 5/2/5	13720 10/3/8	14860 13/3/9	16760 12/4/11	17660 9/3/10
	% Utilization: Mid span/Support	38 / 75	49 / 86	61 / 96	63 / 91	62 / 100	54 / 89
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W200X31 (16)	W250X33 (16)	W250X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 14/1/2	5280 14/2/2	6280 22/3/4	7140 22/4/5	7240 26/4/5	7340 28/4/5
	% Utilization: Mid span	51	53	64	72	74	76
	S-G bottom chord size (Studs)	W310X45 (42)	W310X45 (46)	W310X45 (50)	W310X45 (54)	W310X60 (62)	W310X52 (58)
	Length: Hole 1/Ext stub, (Studs)	640/1640 (18)	640/1640 (20)	640/1640 (22)	640/1640 (24)	640/1640 (28)	640/1640 (26)
	Length: Hole 3/Int stub, (Studs)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)
	% Utilization at: 1/2/3	64 / 74 / 58	71 / 82 / 64	78 / 90 / 70	84 / 98 / 76	69 / 82 / 63	80 / 91 / 68
	% Utilization at: 4/5/6	40 / 19 / 45	45 / 21 / 49	49 / 23 / 54	53 / 25 / 59	53 / 28 / 61	55 / 26 / 59
	Defl:Shore removed+SD+LL=Total	4+3+3=10	4+4+3=11	5+4+4=13	5+4+4=13	4+4+3=11	4+4+3=11
	Steel mass / Cost per bay	1489 / 1864	1627 / 2026	1771 / 2195	2104 / 2546	2640 / 2869	2876 / 3090

[+] See Section 6.20 for explanation.

3-STUB CONFIGURATION

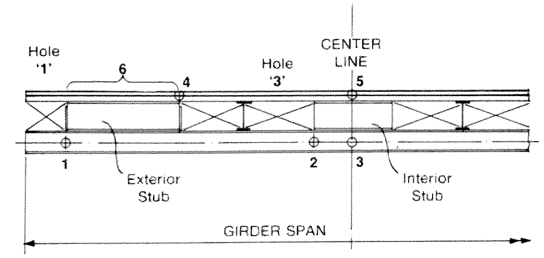


Table 6.7 (continued)
STUB-GIRDER FLOOR BAY[+]
Trial Selection Table

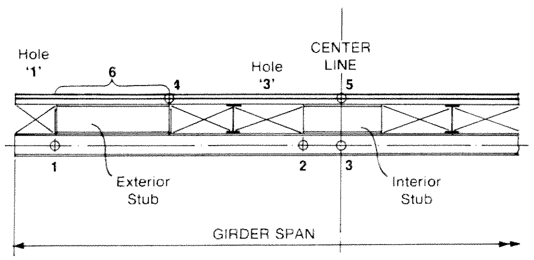
Live Load: 2.4 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
9500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W410X54 (26)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 5/2/5	13660 10/3/8	14860 14/4/9	16480 14/4/11	17660 10/3/10
	% Utilization: Mid span/Support	40 / 78	51 / 90	64 / 100	66 / 96	66 / 100	56 / 94
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W250X33 (16)	W250X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 15/2/2	6340 15/3/3	7140 23/4/5	7520 30/5/6	7340 30/5/6
	% Utilization: Mid span	54	56	60	76	84	80
	S-G bottom chord size (Studs)	W310X45 (46)	W310X45 (50)	W310X45 (54)	W310X52 (62)	W310X60 (66)	W310X52 (62)
	Length: Hole 1/Ext stub, (Studs)	670/1736 (20)	670/1736 (22)	670/1736 (24)	670/1736 (28)	670/1736 (30)	640/1766 (28)
	Length: Hole 3/Int stub, (Studs)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)
	% Utilization at: 1/2/3	70 / 81 / 64	78 / 90 / 71	85 / 99 / 78	79 / 93 / 72	76 / 91 / 69	84 / 100 / 75
	% Utilization at: 4/5/6	43 / 20 / 48	47 / 22 / 53	52 / 24 / 59	53 / 26 / 63	56 / 29 / 66	58 / 28 / 63
	Defl:Shore removed+SD+LL=Total	4+4+4=12	5+4+4=13	6+5+4=15	5+5+4=14	5+5+4=14	5+5+4=14
	Steel mass / Cost per bay	1524 / 1890	1662 / 2052	1825 / 2244	2211 / 2597	2678 / 2891	2925 / 3117
10000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 5/2/6	13460 12/3/8	14860 14/4/9	17160 8/3/9	17660 10/3/10
	% Utilization: Mid span/Support	42 / 82	53 / 94	68 / 100	69 / 100	54 / 93	59 / 98
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W250X33 (16)	W250X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5280 16/2/3	6540 17/3/4	7140 25/4/5	6840 25/4/5	7340 31/5/6
	% Utilization: Mid span	57	59	67	79	73	84
	S-G bottom chord size (Studs)	W310X45 (50)	W310X45 (54)	W310X52 (62)	W310X60 (66)	W310X60 (66)	W310X60 (70)
	Length: Hole 1/Ext stub, (Studs)	710/1823 (22)	710/1823 (24)	710/1823 (28)	680/1853 (30)	630/1903 (30)	610/1923 (32)
	Length: Hole 3/Int stub, (Studs)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)
	% Utilization at: 1/2/3	78 / 89 / 71	86 / 99 / 79	80 / 94 / 74	75 / 92 / 71	75 / 95 / 72	75 / 98 / 74
	% Utilization at: 4/5/6	53 / 21 / 52	59 / 23 / 58	51 / 25 / 63	54 / 28 / 69	57 / 28 / 66	59 / 29 / 68
	Defl:Shore removed+SD+LL=Total	5+5+4=14	6+5+5=16	6+5+5=16	6+5+4=15	6+5+4=15	6+5+4=15
	Steel mass / Cost per bay	1599 / 1916	1737 / 2078	1971 / 2294	2385 / 2708	3049 / 3149	3126 / 3228
10500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W460X61 (30)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/4	12720 6/2/6	13280 14/4/8	16160 4/2/7	17160 8/3/9	17500 12/4/11
	% Utilization: Mid span/Support	43 / 85	56 / 98	72 / 100	46 / 88	56 / 97	62 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W200X27 (14)	W250X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 16/2/2	5280 17/2/3	6720 19/4/5	5840 27/3/4	6840 26/4/5	7500 35/5/7
	% Utilization: Mid span	60	62	74	76	77	92
	S-G bottom chord size (Studs)	W310X45 (54)	W310X52 (62)	W310X60 (66)	W310X60 (66)	W310X67 (70)	W310X67 (74)
	Length: Hole 1/Ext stub, (Studs)	750/1910 (24)	750/1910 (28)	750/1910 (30)	750/1910 (30)	750/1910 (32)	750/1910 (34)
	Length: Hole 3/Int stub, (Studs)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)
	% Utilization at: 1/2/3	86 / 97 / 78	81 / 94 / 74	79 / 93 / 72	86 / 96 / 73	83 / 93 / 70	86 / 97 / 73
	% Utilization at: 4/5/6	71 / 21 / 56	49 / 24 / 62	53 / 27 / 69	56 / 27 / 67	58 / 29 / 72	60 / 30 / 75
	Defl:Shore removed+SD+LL=Total	7+6+5=18	6+6+5=17	7+6+5=18	6+6+5=17	6+6+5=17	7+6+5=18
	Steel mass / Cost per bay	1593 / 1942	1808 / 2132	2055 / 2407	2820 / 2963	3101 / 3258	3168 / 3329

[+] See Section 6.20 for explanation.

Table 6.8
STUB-GIRDER FLOOR BAY[+]
Trial Selection Table

3-STUB CONFIGURATION



Live Load: 3.6 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
8000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/7	13380 12/3/11	14000 23/5/15	15380 25/5/17	17660 9/3/14
	% Utilization: Mid span/Support	48 / 84	60 / 96	76 / 100	94 / 100	91 / 100	65 / 99
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W360X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 14/1/2	5280 14/2/3	6620 16/3/5	8000 17/4/7	8620 24/5/9	7340 29/4/7
	% Utilization: Mid span	59	61	70	83	96	85
	S-G bottom chord size (Studs)	W310X45 (38)	W310X45 (42)	W310X45 (46)	W310X45 (50)	W310X52 (50)	W310X52 (50)
	Length: Hole 1/Ext stub, (Studs)	570/1456 (16)	570/1456 (18)	570/1456 (20)	570/1456 (22)	570/1456 (22)	570/1456 (22)
	Length: Hole 3/Int stub, (Studs)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)
	% Utilization at: 1/2/3	62 / 73 / 57	69 / 80 / 63	75 / 88 / 69	82 / 95 / 75	75 / 89 / 68	78 / 88 / 66
	% Utilization at: 4/5/6	49 / 23 / 40	55 / 25 / 44	60 / 28 / 48	65 / 30 / 52	66 / 32 / 56	66 / 31 / 52
	Defl:Shore removed+SD+LL=Total	3+2+3=8	3+2+3=8	3+3+3=9	4+3+4=11	3+3+3=9	3+2+3=8
	Steel mass / Cost per bay	1420 / 1812	1558 / 1974	1718 / 2163	1827 / 2292	2204 / 2650	2787 / 3035
8500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12660 5/1/8	13160 14/3/11	14540 16/4/13	15900 19/4/16	17380 11/3/14
	% Utilization: Mid span/Support	50 / 89	63 / 100	81 / 100	79 / 100	82 / 100	69 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (14)	W360X33 (14)	W250X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 15/1/2	5340 16/2/3	6840 19/3/6	7460 27/4/8	8100 22/4/8	7620 33/5/9
	% Utilization: Mid span	63	66	79	94	90	98
	S-G bottom chord size (Studs)	W310X45 (42)	W310X45 (46)	W310X45 (50)	W310X52 (54)	W310X60 (58)	W310X52 (54)
	Length: Hole 1/Ext stub, (Studs)	600/1553 (18)	600/1553 (20)	600/1553 (22)	600/1553 (24)	600/1553 (26)	580/1573 (24)
	Length: Hole 3/Int stub, (Studs)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)
	% Utilization at: 1/2/3	69 / 81 / 64	77 / 90 / 70	84 / 98 / 77	77 / 92 / 71	74 / 89 / 68	83 / 98 / 74
	% Utilization at: 4/5/6	54 / 25 / 43	60 / 27 / 47	66 / 30 / 52	67 / 32 / 56	70 / 35 / 59	73 / 34 / 56
	Defl:Shore removed+SD+LL=Total	3+3+4=10	4+3+4=11	4+3+4=11	4+3+4=11	4+3+4=11	4+3+4=11
	Steel mass / Cost per bay	1490 / 1838	1626 / 1999	1785 / 2186	2165 / 2535	2614 / 2807	2877 / 3049
9000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12440 7/2/8	12980 16/3/11	14340 18/4/13	15680 22/5/16	17080 14/4/15
	% Utilization: Mid span/Support	52 / 93	67 / 100	85 / 99	84 / 100	86 / 100	73 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (16)	W360X33 (16)	W360X33 (14)
	Length; Defl: Slab pour/SD/LL	4280 16/1/2	5560 19/2/4	7020 22/4/7	7660 32/5/10	8320 25/5/9	7920 23/4/7
	% Utilization: Mid span	67	76	88	99	97	91
	S-G bottom chord size (Studs)	W310X45 (46)	W310X45 (50)	W310X52 (54)	W310X60 (58)	W310X60 (62)	W310X60 (62)
	Length: Hole 1/Ext stub, (Studs)	640/1640 (20)	640/1640 (22)	640/1640 (24)	640/1640 (26)	640X1640 (28)	640/1640 (28)
	Length: Hole 3/Int stub, (Studs)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)
	% Utilization at: 1/2/3	78 / 90 / 71	86 / 99 / 78	80 / 94 / 73	77 / 92 / 70	83 / 99 / 76	86 / 98 / 73
	% Utilization at: 4/5/6	63 / 26 / 46	70 / 29 / 51	67 / 32 / 56	71 / 35 / 61	77 / 38 / 64	77 / 36 / 61
	Defl:Shore removed+SD+LL=Total	4+3+5=12	5+4+5=14	4+3+5=12	4+3+4=11	5+4+5=14	5+4+4=13
	Steel mass / Cost per bay	1489 / 1864	1622 / 2020	1850 / 2234	2235 / 2633	2605 / 2831	2926 / 3138

[+] See Section 6.20 for explanation.

3-STUB CONFIGURATION

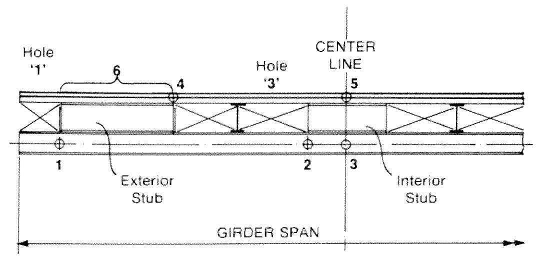


Table 6.8 (continued)
STUB-GIRDER FLOOR BAY[+]
Trial Selection Table

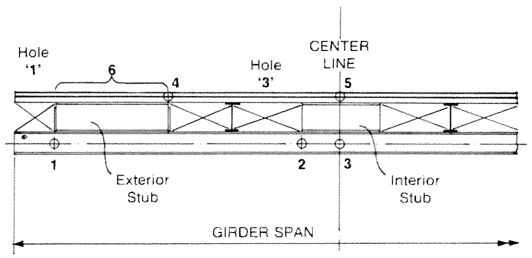
Live Load: 3.6 kPa
Cover Slab: 75 mm N.D. Concrete

Girder Span (mm)	Description	Beam Span (mm)					
		8 000	9 000	10 000	11 000	12 000	12 500
9500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12240 8/2/8	12820 18/4/12	14160 21/4/14	15480 24/5/17	16820 16/4/15
	% Utilization: Mid span/Support	54 / 97	71 / 100	90 / 99	89 / 100	91 / 100	78 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W360X33 (14)	W360X33 (20)	W360X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 17/1/3	5760 22/3/5	7180 25/4/8	7840 20/4/8	8520 28/6/10	8180 27/5/9
	% Utilization: Mid span	70	86	97	94	99	99
	S-G bottom chord size (Studs)	W310X45 (50)	W310X52 (54)	W310X60 (58)	W310X67 (62)	W310X67 (70)	W310X67 (66)
	Length: Hole 1/Ext stub, (Studs)	670/1736 (22)	670/1736 (24)	640/1766 (26)	590/1816 (28)	670/1736 (32)	530/1876 (30)
	Length: Hole 3/Int stub, (Studs)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)
	% Utilization at: 1/2/3	85 / 99 / 79	80 / 95 / 74	75 / 93 / 72	66 / 91 / 70	81 / 98 / 75	67 / 97 / 73
	% Utilization at: 4/5/6	88 / 27 / 50	65 / 30 / 55	69 / 34 / 60	72 / 36 / 64	78 / 39 / 68	78 / 38 / 63
	Defl:Shore removed+SD+LL=Total	5+4+5=14	5+4+5=14	5+4+5=14	5+4+5=14	5+4+5=14	5+4+5=14
	Steel mass / Cost per bay	1524 / 1890	1723 / 2066	1969 / 2339	2354 / 2730	2715 / 2925	3053 / 3227
10000	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W410X54 (22)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11640 3/1/6	12060 9/2/8	13380 12/3/10	14680 15/3/12	16320 13/3/13	16580 19/5/15
	% Utilization: Mid span/Support	57 / 100	75 / 99	75 / 100	79 / 100	74 / 100	82 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (12)	W250X33 (14)	W250X33 (18)	W250X33 (26)	W360X33 (22)
	Length; Defl: Slab pour/SD/LL	4360 18/2/3	5940 25/3/6	6620 20/3/6	7320 30/5/9	7680 38/6/11	8420 31/6/10
	% Utilization: Mid span	77	96	87	97	92	99
	S-G bottom chord size (Studs)	W310X52 (54)	W310X60 (58)	W310X67 (62)	W310X67 (70)	W310X74 (70)	W310X74 (70)
	Length: Hole 1/Ext stub, (Studs)	710/1823 (24)	670/1863 (26)	620/1913 (28)	710/1823 (32)	520/2013 (32)	500/2033 (32)
	Length: Hole 3/Int stub, (Studs)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)
	% Utilization at: 1/2/3	80 / 94 / 74	74 / 93 / 73	67 / 92 / 71	84 / 100 / 77	59 / 93 / 69	59 / 96 / 72
	% Utilization at: 4/5/6	62 / 28 / 53	66 / 32 / 59	69 / 34 / 63	75 / 37 / 68	75 / 38 / 65	78 / 39 / 67
	Defl:Shore removed+SD+LL=Total	5+4+6=15	5+4+6=15	5+4+5=14	6+5+6=17	5+4+5=14	5+4+5=14
	Steel mass / Cost per bay	1670 / 1940	1885 / 2174	2331 / 2624	2690 / 2821	3174 / 3256	3240 / 3324
10500	Cantilever seg. size (Studs)	W410X39 (16)	W410X39 (16)	W410X46 (20)	W460X61 (24)	W460X61 (24)	W460X61 (24)
	Length; Defl: Slab pour/SD/LL	11440 4/1/6	11920 10/2/9	13220 13/3/11	15660 7/2/10	16100 15/4/14	16380 21/5/16
	% Utilization: Mid span/Support	60 / 100	79 / 100	79 / 100	63 / 100	78 / 100	86 / 100
	Suspended seg. size (Studs)	W150X22 (20)	W200X27 (22)	W250X33 (14)	W250X33 (14)	W360X33 (18)	W410X39 (16)
	Length; Defl: Slab pour/SD/LL	4560 22/2/4	6080 29/4/7	6780 23/4/7	6340 21/3/5	7900 25/5/9	8620 21/5/9
	% Utilization: Mid span	88	83	96	84	98	97
	S-G bottom chord size (Studs)	W310X60 (58)	W310X67 (62)	W310X74 (70)	W310X74 (70)	W310X79 (74)	W310X79 (78)
	Length: Hole 1/Ext stub, (Studs)	750/1910 (26)	750/1910 (28)	750/1910 (32)	750/1910 (32)	750/1910 (34)	750/1910 (36)
	Length: Hole 3/Int stub, (Studs)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)
	% Utilization at: 1/2/3	79 / 92 / 72	77 / 92 / 71	76 / 91 / 70	82 / 94 / 71	83 / 96 / 72	86 / 100 / 74
	% Utilization at: 4/5/6	64 / 30 / 57	68 / 34 / 64	71 / 37 / 69	74 / 36 / 68	78 / 39 / 73	80 / 41 / 76
	Defl:Shore removed+SD+LL=Total	6+5+6=17	6+5+6=17	6+5+6=17	6+5+6=17	6+5+6=17	6+5+6=17
	Steel mass / Cost per bay	1751 / 2049	1958 / 2279	2396 / 2732	3008 / 3153	3186 / 3340	3332 / 3496

[+] See Section 6.20 for explanation.

Table 6.9
STUB-GIRDER FLOOR BAY+
Trial Selection Table

3-STUB CONFIGURATION



Live Load: 3.6 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
8000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 4/1/7	13540 9/3/11	14120 19/4/15	15540 21/5/17	17660 8/3/14
	% Utilization: Mid span/Support	45 / 81	57 / 93	70 / 100	87 / 100	87 / 100	61 / 95
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W360X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 12/1/2	5280 13/2/3	6460 13/2/4	7880 14/3/7	8460 20/5/9	7340 25/4/7
	% Utilization: Mid span	55	57	63	77	88	81
	S-G bottom chord size (Studs)	W310X45 (42)	W310X45 (46)	W310X45 (50)	W310X45 (54)	W310X45 (58)	W310X45 (54)
	Length: Hole 1/Ext stub, (Studs)	570/1456 (18)	570/1456 (20)	570/1456 (22)	570/1456 (24)	570/1456 (26)	570/1456 (24)
	Length: Hole 3/Int stub, (Studs)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)	760/1146 (6)
	% Utilization at: 1/2/3	60 / 70 / 54	66 / 78 / 60	73 / 85 / 66	79 / 92 / 71	85 / 99 / 77	88 / 98 / 74
	% Utilization at: 4/5/6	39 / 19 / 44	43 / 21 / 49	47 / 23 / 54	51 / 25 / 58	55 / 27 / 62	55 / 26 / 57
	Defl:Shore removed+SD+LL=Total	2+2+3=7	3+2+3=8	3+3+4=10	3+3+4=10	3+3+4=10	3+3+4=10
	Steel mass / Cost per bay	1420 / 1812	1558 / 1974	1719 / 2164	1829 / 2293	2149 / 2633	2728 / 3014
8500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12720 5/1/8	13320 11/3/11	13940 22/5/15	15320 24/6/18	17620 9/3/14
	% Utilization: Mid span/Support	47 / 85	59 / 97	75 / 100	92 / 100	93 / 100	64 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W360X33 (16)	W360X33 (16)	W250X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 13/1/2	5280 14/2/3	6680 15/3/5	8060 16/4/8	8680 23/5/10	7380 27/4/8
	% Utilization: Mid span	58	61	72	85	98	87
	S-G bottom chord size (Studs)	W310X45 (46)	W310X45 (50)	W310X45 (54)	W310X52 (58)	W310X52 (62)	W310X52 (62)
	Length: Hole 1/Ext stub, (Studs)	600/1553 (20)	600/1553 (22)	600/1553 (24)	600/1553 (26)	600/1553 (28)	600/1553 (28)
	Length: Hole 3/Int stub, (Studs)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)	800/2033 (6)
	% Utilization at: 1/2/3	66 / 78 / 61	74 / 87 / 67	80 / 95 / 74	74 / 89 / 68	80 / 96 / 73	83 / 95 / 71
	% Utilization at: 4/5/6	43 / 21 / 48	47 / 23 / 53	52 / 25 / 58	53 / 27 / 63	57 / 29 / 68	57 / 28 / 62
	Defl:Shore removed+SD+LL=Total	3+3+4=10	3+3+4=10	4+3+4=11	4+3+4=11	4+3+4=11	4+3+4=11
	Steel mass / Cost per bay	1490 / 1838	1627 / 2000	1787 / 2188	1959 / 2339	2285 / 2676	2885 / 3060
9000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W410X54 (26)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12600 5/2/8	13120 13/3/11	14500 15/4/14	15840 18/5/16	17300 11/4/15
	% Utilization: Mid span/Support	49 / 90	62 / 100	79 / 100	81 / 100	81 / 100	68 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W250X33 (16)	W360X33 (16)	W250X33 (18)
	Length; Defl: Slab pour/SD/LL	4280 14/1/2	5400 15/2/4	6880 18/3/6	7500 26/5/9	8160 21/4/8	7700 32/5/10
	% Utilization: Mid span	62	67	80	95	92	96
	S-G bottom chord size (Studs)	W310X45 (50)	W310X45 (54)	W310X52 (58)	W310X52 (62)	W310X60 (70)	W310X60 (66)
	Length: Hole 1/Ext stub, (Studs)	640/1640 (22)	640/1640 (24)	640/1640 (26)	640/1640 (28)	640/1640 (32)	640/1640 (30)
	Length: Hole 3/Int stub, (Studs)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)	850/1300 (6)
	% Utilization at: 1/2/3	75 / 87 / 68	82 / 96 / 75	77 / 91 / 70	83 / 98 / 75	80 / 95 / 72	83 / 94 / 70
	% Utilization at: 4/5/6	46 / 22 / 52	51 / 24 / 58	53 / 27 / 63	57 / 29 / 68	60 / 32 / 71	60 / 30 / 69
	Defl:Shore removed+SD+LL=Total	4+3+5=12	4+4+5=13	4+4+5=13	4+4+5=13	4+4+5=13	4+4+5=13
	Steel mass / Cost per bay	1489 / 1864	1625 / 2024	1851 / 2235	2163 / 2561	2610 / 2837	2936 / 3148

+ See Section 6.20 for explanation.

3-STUB CONFIGURATION

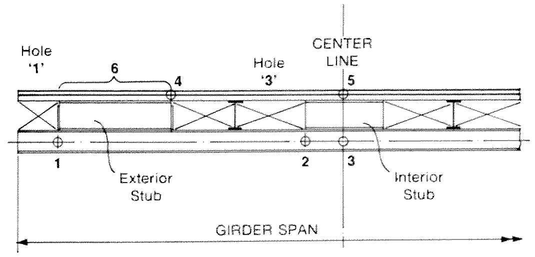


Table 6.9 (continued)
STUB-GIRDER FLOOR BAY+
Trial Selection Table

Live Load: 3.6 kPa
Cover Slab: 85 mm S.L.D. Concrete

Girder Span (mm)	Description	Beam Span (mm) 8 000	9 000	10 000	11 000	12 000	12 500
9500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W410X54 (26)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/5	12400 6/2/8	12960 15/4/12	14320 17/4/14	15640 20/5/17	17040 13/4/15
	% Utilization: Mid span/Support	51 / 94	66 / 100	84 / 100	85 / 100	86 / 100	72 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W250X33 (20)	W360X33 (18)	W360X33 (16)
	Length; Defl: Slab pour/SD/LL	4280 15/1/3	5600 18/2/5	7040 20/4/7	7680 30/5/10	8360 23/5/10	7960 22/4/8
	% Utilization: Mid span	65	76	89	97	98	93
	S-G bottom chord size (Studs)	W310X45 (54)	W310X52 (58)	W310X60 (66)	W310X60 (70)	W310X67 (74)	W310X67 (70)
	Length: Hole 1/Ext stub, (Studs)	670/1736 (24)	670/1736 (26)	670/1736 (30)	620/1786 (32)	670/1736 (34)	550/1856 (32)
	Length: Hole 3/Int stub, (Studs)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)	900/1366 (6)
	% Utilization at: 1/2/3	82 / 95 / 75	77 / 91 / 71	75 / 90 / 69	75 / 97 / 74	78 / 95 / 72	66 / 94 / 69
	% Utilization at: 4/5/6	49 / 23 / 56	51 / 25 / 62	54 / 28 / 69	59 / 30 / 74	61 / 33 / 77	61 / 32 / 72
	Defl:Shore removed+SD+LL=Total	4+4+6=14	4+4+5=13	5+4+5=14	5+4+5=14	5+4+5=14	4+4+5=13
	Steel mass / Cost per bay	1524 / 1890	1726 / 2070	1968 / 2341	2285 / 2664	2720 / 2931	3060 / 3237
10000	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W410X46 (22)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11720 2/1/6	12220 8/2/9	12820 16/4/12	14140 19/5/14	16540 11/3/13	16800 15/4/15
	% Utilization: Mid span/Support	53 / 98	69 / 100	88 / 100	89 / 100	69 / 100	76 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (16)	W360X33 (16)	W250X33 (20)	W360X33 (18)
	Length; Defl: Slab pour/SD/LL	4280 15/1/3	5780 20/3/6	7180 23/4/8	7860 18/4/8	7460 31/5/10	8200 25/5/10
	% Utilization: Mid span	69	85	97	95	96	100
	S-G bottom chord size (Studs)	W310X52 (58)	W310X52 (62)	W310X60 (70)	W310X67 (74)	W310X67 (74)	W310X74 (78)
	Length: Hole 1/Ext stub, (Studs)	710/1823 (26)	700/1833 (28)	640/1893 (32)	590/1943 (34)	50/1983 (34)	530/2003 (36)
	Length: Hole 3/Int stub, (Studs)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)	950/2383 (6)
	% Utilization at: 1/2/3	77 / 90 / 71	84 / 100 / 78	75 / 98 / 76	67 / 96 / 73	67 / 99 / 74	60 / 93 / 68
	% Utilization at: 4/5/6	48 / 24 / 61	53 / 26 / 67	57 / 29 / 73	59 / 32 / 78	62 / 32 / 74	61 / 33 / 77
	Defl:Shore removed+SD+LL=Total	5+4+6=15	5+5+6=16	5+5+6=16	5+5+6=16	5+5+6=16	5+4+5=14
	Steel mass / Cost per bay	1672 / 1942	1801 / 2093	2055 / 2371	2453 / 2765	3106 / 3193	3246 / 3335
10500	Cantilever seg. size (Studs)	W410X39 (20)	W410X39 (20)	W410X39 (20)	W460X61 (30)	W460X61 (30)	W460X61 (30)
	Length; Defl: Slab pour/SD/LL	11620 3/1/6	12060 9/2/9	12680 18/4/13	15900 5/2/10	16300 12/4/14	16580 17/5/16
	% Utilization: Mid span/Support	56 / 100	73 / 100	92 / 100	59 / 100	73 / 100	80 / 100
	Suspended seg. size (Studs)	W150X22 (24)	W200X27 (14)	W250X33 (20)	W200X27 (14)	W250X33 (32)	W360X33 (26)
	Length; Defl: Slab pour/SD/LL	4380 17/2/3	5940 23/3/6	7320 26/5/9	6100 30/4/7	7700 36/6/11	8420 28/6/11
	% Utilization: Mid span	75	94	97	100	90	97
	S-G bottom chord size (Studs)	W310X52 (62)	W310X60 (70)	W310X67 (74)	W310X67 (78)	W310X74 (82)	W310X79 (86)
	Length: Hole 1/Ext stub, (Studs)	750/1910 (28)	750/1910 (32)	750/1910 (34)	750/1910 (36)	750/1910 (38)	750/1910 (40)
	Length: Hole 3/Int stub, (Studs)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)	990/1520 (6)
	% Utilization at: 1/2/3	85 / 98 / 77	84 / 98 / 76	81 / 97 / 75	89 / 100 / 75	85 / 98 / 73	82 / 96 / 71
	% Utilization at: 4/5/6	51 / 25 / 65	55 / 28 / 73	58 / 30 / 80	60 / 30 / 77	62 / 33 / 83	63 / 34 / 86
	Defl:Shore removed+SD+LL=Total	6+5+7=18	6+5+7=18	6+5+7=18	6+5+6=17	6+5+6=17	6+5+6=17
	Steel mass / Cost per bay	1667 / 1967	1883 / 2205	2127 / 2478	2883 / 3025	3140 / 3294	3259 / 3418

+ See Section 6.20 for explanation.

CHAPTER 7

7.0 FLOOR VIBRATION – COMPOSITE CONSTRUCTION

7.1 INTRODUCTION

The assessment of vibration characteristics of a floor from a serviceability standpoint is a complex subject. It involves the assessment of certain dynamic properties of the floor system such as the natural frequency, mass, damping, and dynamic response in terms of acceleration. It also requires consideration of the occupants' threshold of annoyance to vibrations. The threshold of annoyance generally varies depending on the types of floor vibration and the types of occupancy.

A guide for assessing floor vibrations is outlined in Appendix G of CAN3-S16.1-M84 which provides a method of evaluating serviceability and acceptance criteria. The authors have based the following commentary and the following design examples on Appendix G, and on their experience.

There is an increasing trend toward large column-free "office landscaped" floor spaces, and an increasing trend to compositely designed floor systems, using lighter and shallower floor construction with less inherent damping. Superimposed loads are also a component of a structure's performance under vertical impact loading. This aspect is now addressed mathematically in Appendix G, in that, a designer is able to include an appropriate amount of the design loads in his calculations of acceleration and frequency. Since office occupancy space is rarely subjected to its design load, it is one of the more susceptible design cases, and one must ensure that objectionable floor vibrations will not occur during normal occupancy activity.

In estimating appropriate damping values for various types of construction, Appendix G differentiates between composite and non-composite assemblies, assigning a lesser value to compositely designed floors.

Any steel/concrete floor assembly usually has some means of positive connection between concrete deck and beams (e.g. arc spot welds through the steel deck). Since very small deflections are involved in vibrations due to normal human activity, there may indeed be sufficient interaction to prevent distinction between composite and non-composite construction in many instances. Therefore it is the authors' recommendation that in borderline cases, damping should be assumed to be equivalent to composite construction, regardless of the actual construction used. However it is also suggested that this subject is worthy of further investigation.

7.2 TYPES OF FLOOR VIBRATION

Floor vibrations can be divided into two categories, namely continuous or steady state source vibrations and transient vibrations. ***Continuous vibrations*** are usually caused by periodic forces of machinery, vehicles or human group activities. If the dynamic force frequency approaches the natural frequency of the floor, the magnitude of vibration can be considerably magnified and a resonance state is reached unless there is a large amount of damping. Human group activities, such as dancing or gymnastics, can generate periodic forces with frequencies as high as about 4 Hz. Therefore, in general, floors used for such occupancies are designed with natural frequencies of greater than 5 hertz (8 hertz for very repetitive activities such as jumping exercises). The

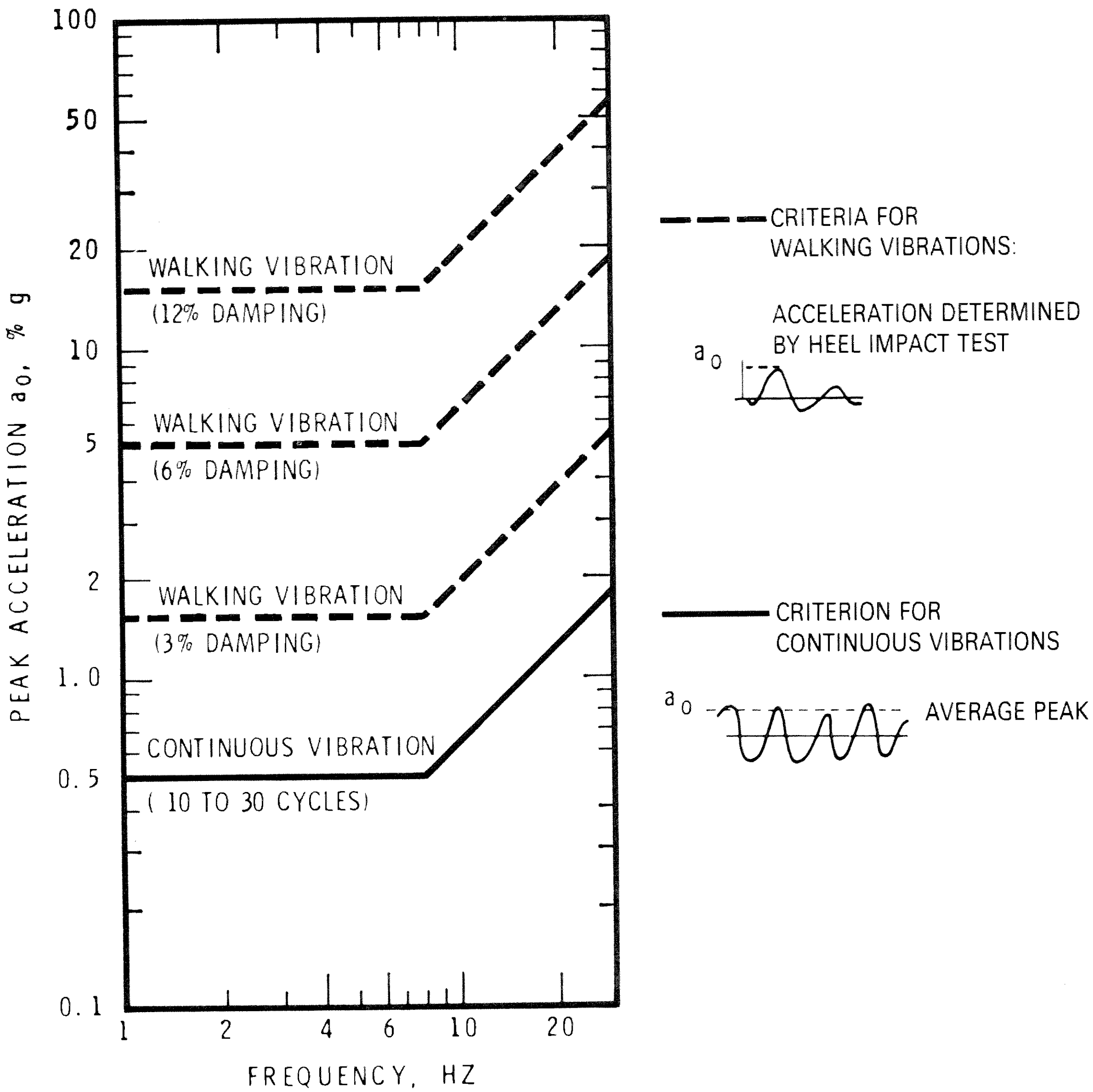


Figure 7.1
Annoyance Thresholds for Floor Vibrations due to Footstep (Residential, School, Office Occupancies) (as per S16.1-M84, Appendix G)

Commentary on Serviceability Criteria for Deflection and Vibrations, as published in Ref.(7.1), provides more specific guidance on vibrations due to rhythmic human activities. ***Transient vibrations*** arise due to impulses caused by footsteps or other impacts. Floor vibrations caused by such impulses decay at a rate dependent on the amount of available damping in (and on) the floor system.

7.3 TYPES OF OCCUPANCY

Human sensitivity to floor vibration differs in areas of different occupancies. Graphical representation of human annoyance criteria for floor vibrations, as proposed for use in residential, school, and office occupancies, is shown in Fig. 7.1. These annoyance thresholds, approximated by thresholds of definite perception, are expressed in terms of peak acceleration due to the standard heel impact test for various critical damping ratios and natural frequencies of floor assemblies.

Annoyance threshold design levels may be lowered for occupancies such as operating rooms and certain laboratories and may be raised for industrial occupancies where floor vibrations are less objectionable. Recent test results(7.2) also suggest a higher threshold level for walking areas of shopping centres.

Continuous vibrations caused by machines in operation can best be minimized by isolating this equipment from 'quiet' occupancies such as office areas. This is because most people find continuous vibration to be much more annoying than transient vibration which dies out quickly, that is, generally speaking, within 5 cycles(7.1). For continuous sinusoidal vibration, a threshold level of 0.5 percent of gravity acceleration is suggested by Appendix G. See Fig. 7.1. It should be noted that design and analysis of floor systems dealing with continuous floor vibration is generally applied to a very specific and isolated area of the total structure. Hence, in the following sections of this chapter, the question of continuous floor vibration is totally ignored while the design procedure dealing with the serviceability of composite steel floors under the subject of transient floor vibration due to footsteps for residential, school, and office occupancies is illustrated in accordance with Appendix G.

7.4 WALKING VIBRATION

Vibration due to footstep impact is considered the most common source of annoyance in lightly damped floors (e.g. open floor plan without partitions, bookshelves, file storage, etc.) of office, school and residential occupancies. One-way floor systems with spans less than 8 metres, and that satisfy live load deflection limitations, usually possess acceptable vibration characteristics.

The characteristics of transient vibrations due to footsteps on floor systems with spans greater than 8 metres and with natural frequencies of 4 to 15 hertz may be evaluated during a structural design using the procedures provided by Appendix G. These procedures are illustrated in the following text. Using this method, it can be shown that, for a floor system with natural frequencies less than 8 hertz, an increase in stiffness in the floor system causes an increase in the peak acceleration computed using the same heel-drop impact. When these results are plotted on the graph shown in Figure 7.1, it can be seen that, in such a case, the floor will be more susceptible to vibrations in the perceptible range, provided that damping of the floor assembly remains unchanged.

7.5 MATHEMATICAL SIMULATION OF VIBRATION CHARACTERISTICS

One-way Floor System

The performance of a floor can be determined by means of a performance test as proposed in Appendix G. In the absence of such a test, the vibration characteristics of a one-way system may be assessed by using the simple computational approach outlined in Appendix G. The frequency in hertz is given by,

$$f_1 = 156\sqrt{\frac{E\ I_T}{w\ L^4}} \qquad 7.1$$

where E = 200 000 MPa,

I_T = moment of inertia of the transformed T-section, in mm^4, (concrete transformed to steel with slab width equal to spacing of steel members, and slab thickness equal to t_e as defined in equation 7.3),

w = total dead load (less movable partitions load allowance) per unit length, in N/mm,

L = span, in mm.

Where the span is greater than 8 m and the fundamental frequency is less than 10 Hz, Allen and

Rainer[7.3] have shown that the peak acceleration due to heel-drop for a one-way system can be approximated by,

$$a_o = 0.9\,(2\pi f)J/M \tag{7.2}$$

in which,
f = fundamental frequency,
J = impulse due to a "standard heel-drop",
M = mass of an equivalent simple oscillator.

Assuming a "standard heel-drop" impulse of 70 N·s and a mass of an equivalent simply supported beam vibrating in the fundamental mode representing the floor system, a_o, in percent of gravitational acceleration, is then rewritten in Appendix G as,

$$a_o = \frac{60f}{q\,B\,L} \tag{7.3}$$

in which,

q = load due to floor plus contents (kPa), as appropriate,
B = 40 t_e, in metres,
L = span of beam, in metres,
t_e = average thickness of concrete deck-slab (m).

Following the assessment of f_1 and a_o, Fig. 7.1, representing annoyance criteria for floor vibrations, can then be used to determine the amount of required damping in (and on) the floor system. Note again that Fig. 7.1 is intended for use in residential, school, office and similar occupancies. Recent research data[7.4] have shown that damping is the most influential parameter. Transient vibration can be more effectively controlled by increasing damping than by reducing acceleration or by altering the frequencies of the floor.

Floor finishing, carpet, furniture, ceiling, ducts and fire protection materials contribute considerably to total damping of a floor system. Partitions, either above or below the floor (in certain situations), if favourably located and oriented, also provide a substantial amount of damping. The following values are suggested in Appendix G for design calculations:

	Damping in Percent Critical
– Bare floor (fully composite construction)	2
– Finished composite floor (with ceiling, ducts, flooring, furniture)	5
– Finished floor with partitions.	12

Two-way Interaction

Often, trusses or beams are supported by girders (such as those in an interior floor bay) instead of rigid supports. The floor frequency, f, of a two-way system is smaller than f_1 or f_2, where f_2 is the frequency of floor in the girder direction. An approximation for f, using Dunkerley's formula, is suggested in Appendix G:

$$f = (f_1^{-2} + f_2^{-2})^{-0.5} \tag{7.4}$$

If f_1 is much smaller than f_2 (say $f_1 < 0.5\,f_2$), the floor behaves like a one-way ***beam*** system. Hence the equivalent vibrating floor area, BL, shall be computed as noted in equation 7.3. When f_2 is much less than f_1, one-way ***girder*** behaviour prevails. In this case, the product BL may be estimated as the tributary floor area supported by the girder. For cases where f_1 and f_2 are close, the formula

provided by Appendix G for calculating equivalent vibrating floor area of the two-way system may be used:

$$BL = \left(\frac{f}{f_1}\right)^2 B_1\,L_1 + \left(\frac{f}{f_2}\right)^2 B_2\,L_2 \tag{7.5}$$

where the subscripts 1 and 2 denote the beam and girder systems respectively.

Often, in practice, a true two-way beam and girder interaction is prevented by certain non-structural features in the building. A spandrel girder or a girder at the location of a full height partition is in effect so stiff that one-way beam vibration may prevail. When it becomes doubtful as to whether a true two-way behaviour exists, one may investigate one-way behaviour in addition to that of the two-way system, keeping in mind that, in any event, damping is the dominant factor and field test data for two-way systems are still lacking. The simple-approach calculation that is outlined in this Chapter is illustrated in the examples in Section 7.6.

The stub-girder floor system is clearly a two-way beam/girder system. The design method recommended in the previous paragraphs of this section to determine fundamental frequency and peak acceleration of floor systems is also applicable to a stub-girder floor [7.5]. Also see Section 7.6 for worked example.

7.6 DESIGN EXAMPLES

Since floor vibration due to one or more occupants walking is the most common source of annoyance, this section demonstrates how the simple procedure described in this chapter is used to assess the vibration characteristics of a floor due to a "standard heel-drop". This approximate method may be useful in deciding whether there is likely to be a problem sufficient to warrant further investigation. In view of its limitations however, this simple approach does not always provide a definitive conclusion to this highly complex and somewhat subjective matter.

Example 1: Assess the "standard heel-drop" vibration characteristics of the typical floor designed in Chapter 4. Figure 7.E1 shows the plan of this hollow composite beam and girder floor.

Since the vibration characteristics of typical bay 'A' and typical bay 'B' (both identified in Fig. 7.E1) can be quite different, the acceptability of each is investigated.

– *Bay 'A'*

Floor beams, B1, span between the rigid core wall and the spandrel girders which also provide fairly stiff supports. Therefore, one-way beam behaviour likely prevails.

Find I_T

$$t_e = \frac{\text{slab dead load}}{w_c\,g} = \frac{2.40 \times 10^6}{2\,300(9.81)} = 106 \text{ mm} \; ; \; n = 9.43$$

Element	Transformed Area (mm²)	Distance from top of slab y (mm)	Ay 10^3 mm³	Ay² 10^6 mm⁴	I_{local} 10^6 mm⁴
Concrete	33 722	53.0	1 787	94.7	31.6
W410×60	7 580	344.5	2 611	899.6	216
Total	41 302		4 398	994.3	247.6

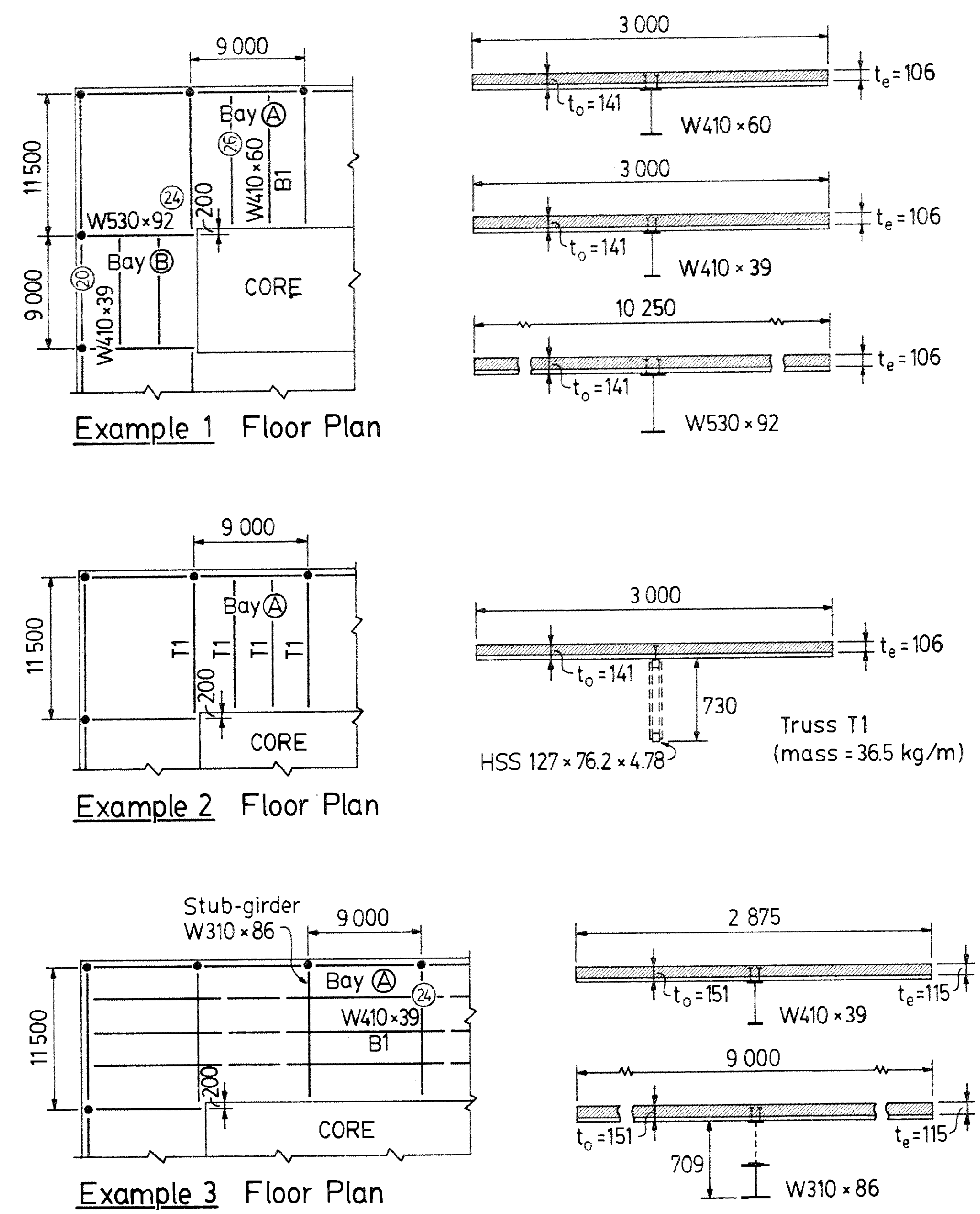


Figure 7.E1
Floor Design Example Key Plans
and Member Sizes
(for Assessment of Standard Heel-Drop
Vibration Characteristics)

$$\bar{y} = \frac{4\ 398 \times 10^3}{41\ 302} = 106.5 \text{ mm}$$

$$I_T = (994.3 + 247.6) \times 10^6 - 41\ 302\ (106.5)^2 = 773 \times 10^6 \text{ mm}^4$$

From Example in Section 4.14

$$w = 0.584 + (2.4 + 0.2 + 0.5)(3) = 9.88 \text{ N/mm} \qquad \text{(or kN/m)}$$

$$f = 156\sqrt{\frac{E\ I_T}{w\ L^4}} \qquad \text{(Eq. 7.1)}$$

$$= 156\sqrt{\frac{200\ 000\ (773 \times 10^6)}{9.88\ (11\ 300)^4}} = 4.83 \text{ Hz}$$

$$q = (9.88)/(3) = 3.29 \text{ kPa (floor plus contents)}$$

$$a_o = \frac{60\ f}{q\ BL} \qquad \text{(Eq. 7.3)}$$

$$= \frac{(60)(4.83)}{(3.29)(40)(0.106)(11.3)}$$

$$= 1.8\%\ g$$

A point representing f = 4.8 Hz and $a_o \simeq 1.8\%$ g is labelled as '1a' in Fig. 7.E2, for which a critical damping ratio somewhere between 3% and 4% is required. Damping amounting to approximately 5% of critical damping (see Section 7.5) is available in the finished floor *per se* (assuming worst condition of no partition, bookcase or other form of on-the-floor damping). Bay 'A' is therefore acceptable.

—Bay 'B'

Consider W410×39 composite beams supported by W530×92 composite girders.

Find I_{T1} (beam)

Element	Transformed Area (mm²)	Distance from top of slab y (mm)	Ay 10^3 mm³	Ay² 10^6 mm⁴	I_{local} 10^6 mm⁴
Concrete	33 722	53.0	1 787	94.7	31.6
W410×39	4 990	340.5	1 699	578.5	127
Total	38 712		3 486	673.2	158.6

$$\bar{y} = \frac{3\ 486 \times 10^3}{38\ 712} = 90.05 \text{ mm}$$

$$I_{T1} = (673.2 + 158.6) \times 10^6 - 38\ 712\ (90.05)^2 = 518 \times 10^6 \text{ mm}^4$$

$$w_1 = 0.384 + (2.4 + 0.2 + 0.5)(3) = 9.68 \text{ N/mm}$$

$$f_1 = 156\sqrt{\frac{200\ 000\ (518 \times 10^6)}{9.68\ (9\ 000)^4}} = 6.30 \text{ Hz}$$

Find I_{T2} (girder)

Element	Transformed Area (mm^2)	Distance from top of slab y (mm)	Ay 10^3 mm^3	Ay^2 10^6 mm^4	I_{local} 10^6 mm^4
Concrete	115 217	53.0	6 106	323.6	107.9
W530×92	11 800	407.5	4 808	1 959	552
Total	127 017		10 914	2 283	659.9

$$\bar{y} = \frac{10\ 914 \times 10^3}{127\ 017} = 85.93 \text{ mm}$$

$$I_{T2} = (2\ 283 + 659.9) \times 10^6 - 127\ 017\ (85.93)^2 = 2\ 005 \times 10^6 \text{ mm}^4$$

$$w_2 = 0.907 + (2.4 + 0.2 + 0.5)(10.25) = 32.68 \text{ N/mm}$$

$$f_2 = 156\sqrt{\frac{200\ 000\ (2\ 005 \times 10^6)}{32.68(9\ 200)^4}} = 6.46 \text{ Hz}$$

Therefore, floor frequency can be computed as:

$$f = (f_1^{-2} + f_2^{-2})^{-0.5} \qquad \text{(Eq. 7.4)}$$

$$= (6.3^{-2} + 6.46^{-2})^{-0.5}$$

$$= 4.51 \text{ Hz}$$

Since f_1 (= 6.3) $\simeq$ f_2 (=6.46), two-way interaction exists.
B_1L_1 (of the beam system) = (40)(0.106)(9) = 38.2 m^2
B_2L_2 (of the girder system) $\simeq$ (10.25)(6.1) = 62.5 m^2

Hence, the area of equivalent vibrating floor is:

$$BL = \left(\frac{f}{f_1}\right)^2 B_1L_1 + \left(\frac{f}{f_2}\right)^2 B_2L_2 \qquad \text{(Eq. 7.5)}$$

$$= \left(\frac{4.51}{6.3}\right)^2 (38.2) + \left(\frac{4.51}{6.46}\right)^2 (62.5)$$

$$= 50 \text{ m}^2$$

Peak acceleration, a_o, in terms of percent of gravitational acceleration,

$$a_o = \frac{60f}{q\ BL}$$

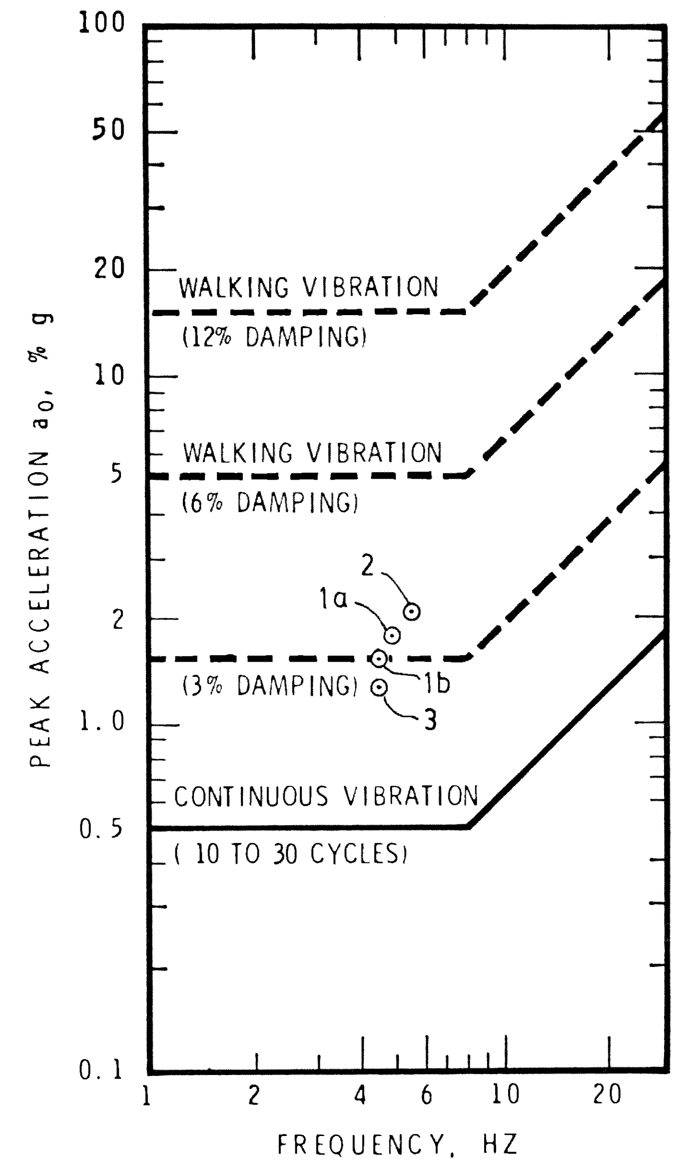


Figure 7.E2
Results of the Standard Heel-Drop Vibration Characteristic Assessments

$$= \frac{(60)(4.51)}{(3.29)(50)}$$

$$= 1.6\% \text{ g}$$

On Fig. 7.E2, a point is labelled as '1b' corresponding to a frequency of 4.51 Hz and a_o of 1.6 % g. Approximately 3% of critical damping is required, and hence bay B is considered to be satisfactory.

Example 2: Using the "standard heel-drop" simulation, assess the vibration characteristics of the typical floor bay 'A' as identified in Fig. 7.E1 using the HSS truss designed in Chapter 5 (see also Fig. 5.E3).

Find I_T $t_e = 106$ mm   (see Example 1)

Element	Transformed Area (mm^2)	Distance from top of slab y (mm)	Ay 10^3 mm^3	Ay^2 10^6 mm^4	I_{local} 10^6 mm^4
Concrete	33 722	53.0	1 787	94.7	31.6
HSS127×76.2 ×4.78	1 790	807.5	1 445	1 167	3.8
Total	35 512		3 232	1 261.7	35.4

$$\bar{y} = \frac{3\ 232 \times 10^3}{35\ 512} = 91.0 \text{ mm}$$

$$I_{chords} = (1\ 261.7 + 35.4) \times 10^6 - 35\ 512\ (91)^2$$
$$= 1\ 003 \times 10^6 \text{ mm}^4$$

I_{wr} $= 51 \times 10^6$ mm^4 see P. 206 (I-reduction due to open webs)

$$I_T = (1\ 003 - 51) \times 10^6 = 952 \times 10^6 \text{ mm}^4$$

Compute frequency, $f = 156 \sqrt{\dfrac{E\ I_T}{w\ L^4}}$ (Eq. 7.1)

$w = 37.1\ (9.81)/1\ 000 + (2.4 + 0.2 + 0.5)(3)$ see P. 210
$= 9.66$ N/mm (or kN/m)

$$f = 156\sqrt{\frac{200\ 000\ (952 \times 10^6)}{9.66\ (11\ 300)^4}} = 5.42 \text{ Hz}$$

$$q = (9.66)/(3) = 3.22 \text{ kPa}$$

$$a_o = \frac{60\ f}{q\ BL}$$

$$= \frac{(60)(5.42)}{(3.22)(40)(0.106)(11.3)}$$

$$= 2.1\%\ g$$

A point labelled '2' is plotted in Figure 7.E2 using the computed values, f = 5.42 Hz and a_o = 2.1% g. The resulting plot shows that a critical damping ratio of about 4% is needed to satisfy the annoyance-threshold for floor vibration due to footsteps. Using Appendix G, and assuming that the finished composite truss floor is mostly open and without partitions, the available floor damping in percent of critical may be estimated at around 5 percent, which is greater than the required 4 percent. Therefore, the design is acceptable.

Example 3: Floor plan and member sizes for a stub-girder floor bay 'A' are shown in Fig. 7.E1. Using the "standard heel-drop" simulation, assess the vibration characteristics of the floor bay.

$$t_e = \frac{2.6 \times 10^6}{2\ 300\ (9.81)} = 115 \text{ mm} \qquad n = 8.44$$

I_{T1}, (for floor beams B1) may be computed using procedure as shown in Example 1.

$$I_{T1} = 550 \times 10^6 \text{ mm}^4.$$

For stub-girder moment of inertia, I_{T2}, the following procedure may be followed.

From section 6.19, the elastic maximum deflection of stub-girder (with no consideration of concrete creep) is given as 0.1168P (mm), where P is given as a point load from member B1, in kN.

Assume girder span = 11 500 mm

Assume equivalent composite (prismatic) girder moment of inertia = I_t

Girder deflection at mid-span (3 equally spaced point loads),

$$\Delta = \frac{19}{384}\frac{P\ L^3}{E\ I_t}$$

Equating deflections,

$$0.1168\ P = \frac{19}{384}\frac{P\ L^3}{E\ I_t}$$

solving for I_t,

$$I_t = \frac{19}{384}\frac{(11\ 500)^3}{(200)(0.1168)} = 3\ 221 \times 10^6 \text{ mm}^4$$

From Table 4.6, a prismatic member of W610×155, with a deck-slab of 76 mm deck and 75 mm slab (of effective width = 2 556, as shown in Section 6.19), would provide a moment of inertia of 3 146 × 10^6 mm^4 ($\simeq$ 3 221 × 10^6 mm^4).

Using this equivalent section, and a slab width of 9 000 mm, and t_e of 115 mm, the value of I_{T2} may be obtained as 4 126 × 10^6 mm^4.

$$w_1 = 0.384 + (2.6 + 0.2 + 0.5)(3) = 10.3 \text{ N/mm}$$

$$w_2 \simeq (0.847) + (9)(0.384/3) + (2.6 + 0.2 + 0.5)(9) = 31.7 \text{ N/mm}$$

$$q = w_2/(9) = 3.52 \text{ kPa}$$

$$f_1 = 156\sqrt{\frac{200\ 000\ (550 \times 10^6)}{10.3\ (9\ 000)^4}} = 6.3 \text{ Hz}$$

$$f_2 = 156\sqrt{\frac{200\ 000\ (4\ 126 \times 10^6)}{31.7\ (11\ 300)^4}} = 6.2 \text{ Hz}$$

$$f = \left(f_1^{-2} + f_2^{-2}\right)^{-0.5} = 4.4 \text{ Hz}$$

$$B_1L_1 = (40)(0.115)(9) = 41.4 \text{ m}^2$$

$$B_2L_2 = (9)(11.3)(3)/(4) = 76.3 \text{ m}^2 \text{ (tributary floor area supported by the girder)}$$

$$BL = \left(\frac{4.4}{6.3}\right)^2 (41.4) + \left(\frac{4.4}{6.2}\right)^2 (76.3)$$

$$= 58.6 \text{ m}^2$$

$$a_o = \frac{60 \text{ f}}{\text{q BL}}$$

$$= \frac{(60)(4.4)}{(3.52)(58.6)}$$

$$= 1.3\% \text{ g}$$

A critical damping ratio of between 2 to 3 percent is required (see point '3' in Figure 7.E2). This is less than the estimated available floor damping of about 5 percent of critical. The stub-girder floor design is acceptable.

REFERENCES

7.1 Supplement to the National Building Code of Canada, 1985 "Commentary on Serviceability Criteria for Deflection and Vibrations".

7.2 Pernica, G., and Allen, D.E., "Floor Vibration Measurements in a Shopping Centre", Canadian Journal of Civil Engineering, June 1982.

7.3 Allen, D.E., and Rainer, J.H., "Vibration Criteria for Long-Span Floors", Canadian Journal of Civil Engineering, June 1976.

7.4 Murray, T.M., "Acceptability Criterion for Occupant-Induced Floor Vibrations", AISC Engineering Journal, Second Quarter, 1981.

7.5 Matthews, C.M., Montgomery, C.J., Murray, D.W., "Designing Floor Systems for Dynamic Response", Structural Engineering Report No. 106, Department of Civil Engineering, University of Alberta, October 1982.

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