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# Design and Construction of COMPOSITE FLOOR SYSTEMS

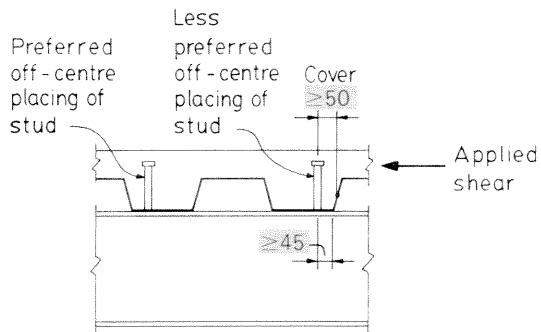
## ERRATUM 1

January 7, 1985

1. Please replace the body of Table 1.1, page 5, with the material below.

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**

Chien  
Ritchie



Canadian Institute of Steel Construction

Design and Construction of  
**COMPOSITE FLOOR SYSTEMS**

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1984

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Design and  
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**COMPOSITE  
FLOOR  
SYSTEMS**

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## **PREFACE**

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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

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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<sup>(1.1)</sup>, 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.

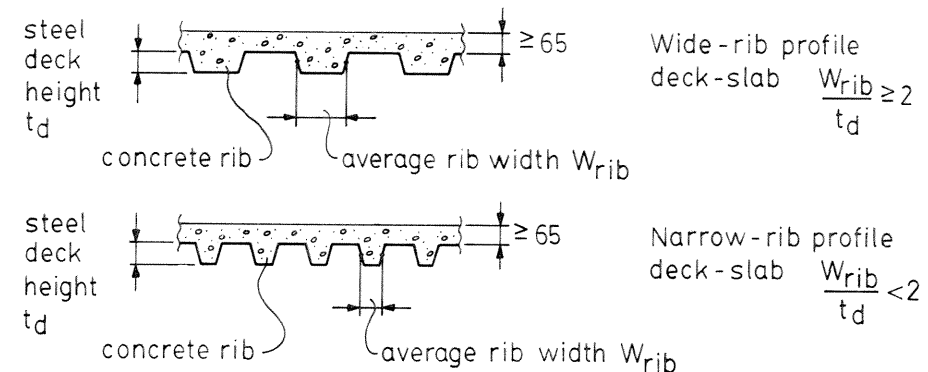


Figure 1.1  
Concrete Ribbed Slab Formed by Steel Decks

## 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”<sup>(1.2)</sup>. 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<sup>(1.3)</sup>. 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.

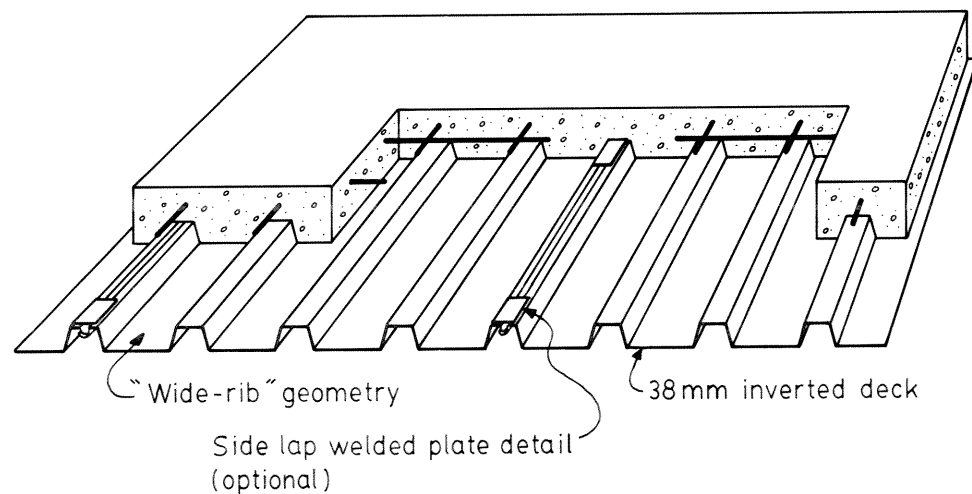
Steel deck profile and thickness govern spanning capability of the deck-slab system either under the fresh-concrete condition load or under occupancy load. Thus, a deeper deck profile and/or a thicker deck sheet would permit wider support spacing. This would permit a reduction in the number of steel 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+		Sheet Width (mm)	Thickness Range (mm)	F.P. Spray Contact Area (m <sup>2</sup> /m <sup>2</sup> )	Profile# Geometry
		A	B				
		Grade					
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<sup>(1,4)</sup>. 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<sup>(1.5 to 1.8)</sup>. 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<sup>(1.18)</sup>.

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<sup>(1.9)</sup>. Recent research projects on the behaviour of composite deck-slabs under repeated load<sup>(1.10,1.11,1.12)</sup> 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<sup>(1.13 to 1.17)</sup>. 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  $P-\Delta$  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.

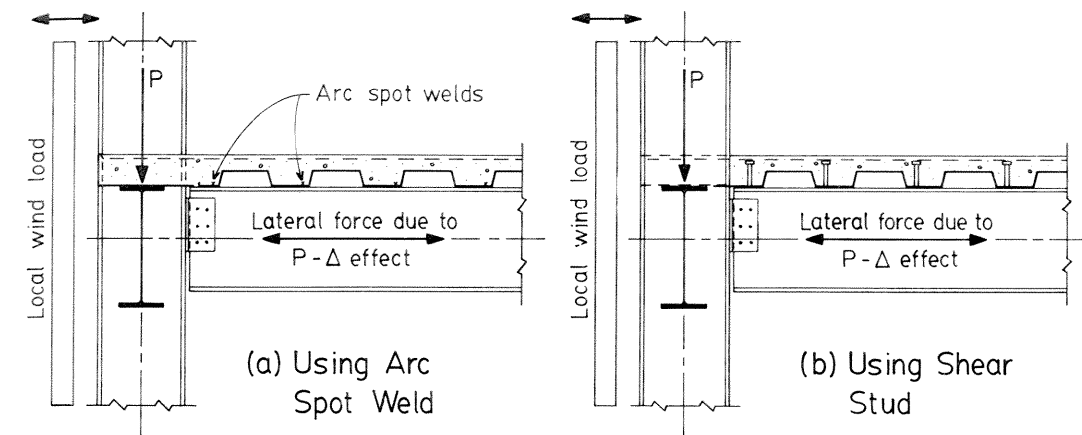
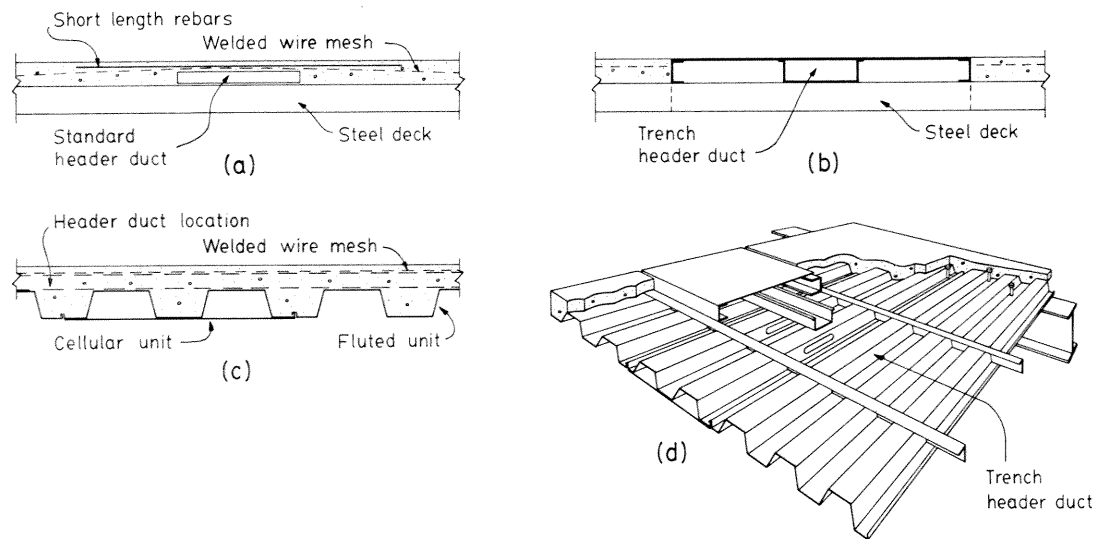


Figure 1.3  
Deck-Slab Shear Diaphragm  
Acting as Column Lateral Support

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**

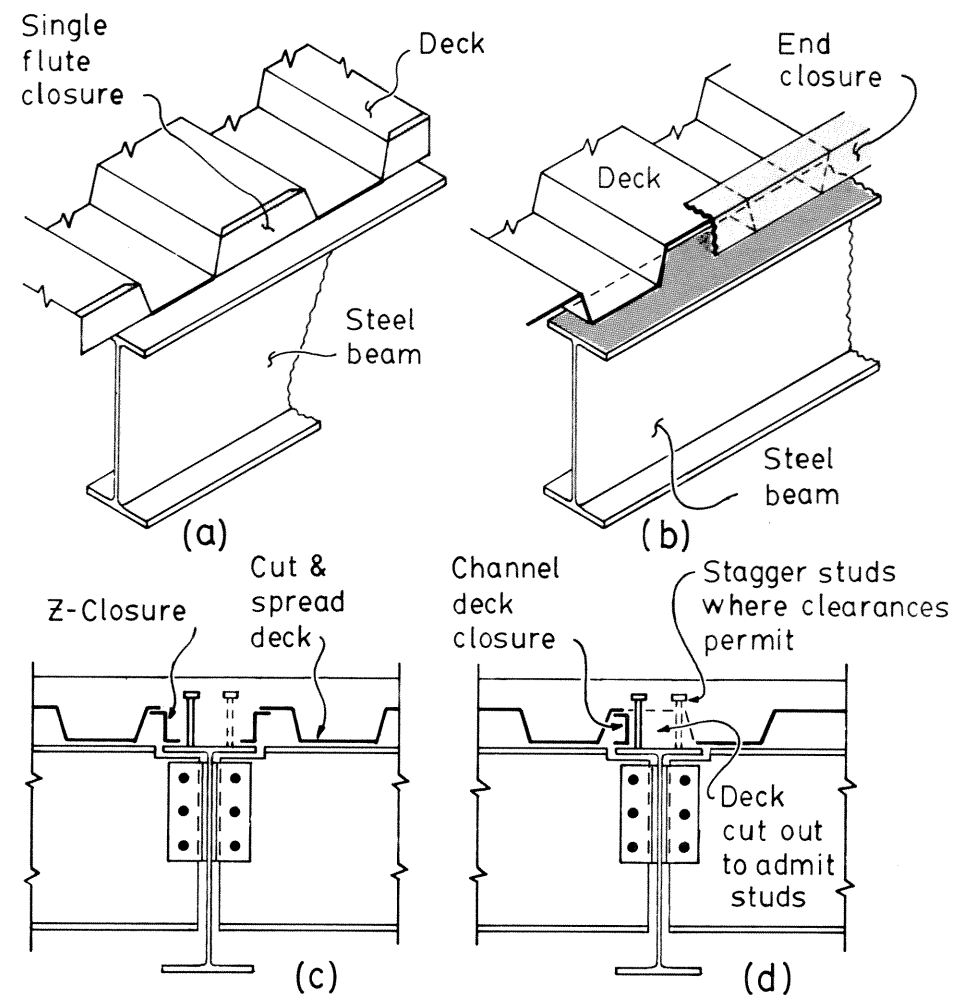
provide the shear transfer mechanism for transmitting external lateral forces and column stability forces into the shear core. At column locations, closure plates may be required between flanges and trim members to pick up the deck edge around large columns. Figures 1.5 to 1.11 suggest typical details. 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.**

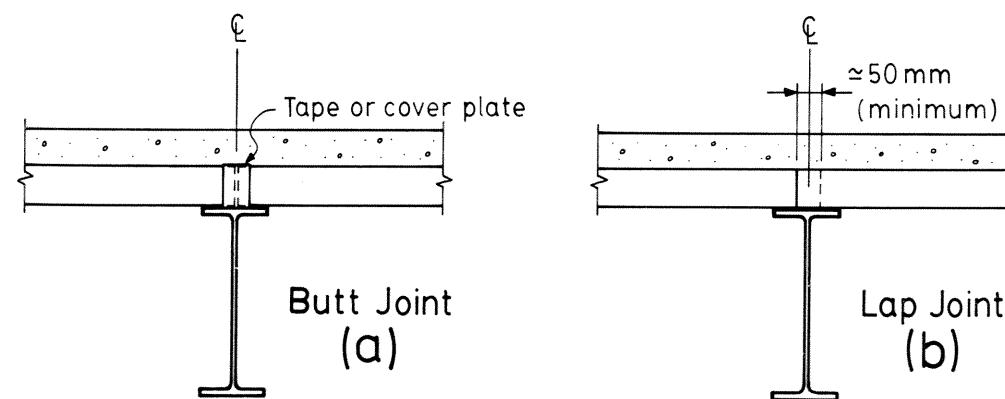
- Skew cutting: This involves the cost of waste and cost for labour hours.
- Length of panels: Preferred lengths are in the range of 9 metres. Lengths beyond 12 metres are rarely handled.
- Width of Coverage: Wider units generally require fewer man-hours for installation and shipment, but width is usually 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**

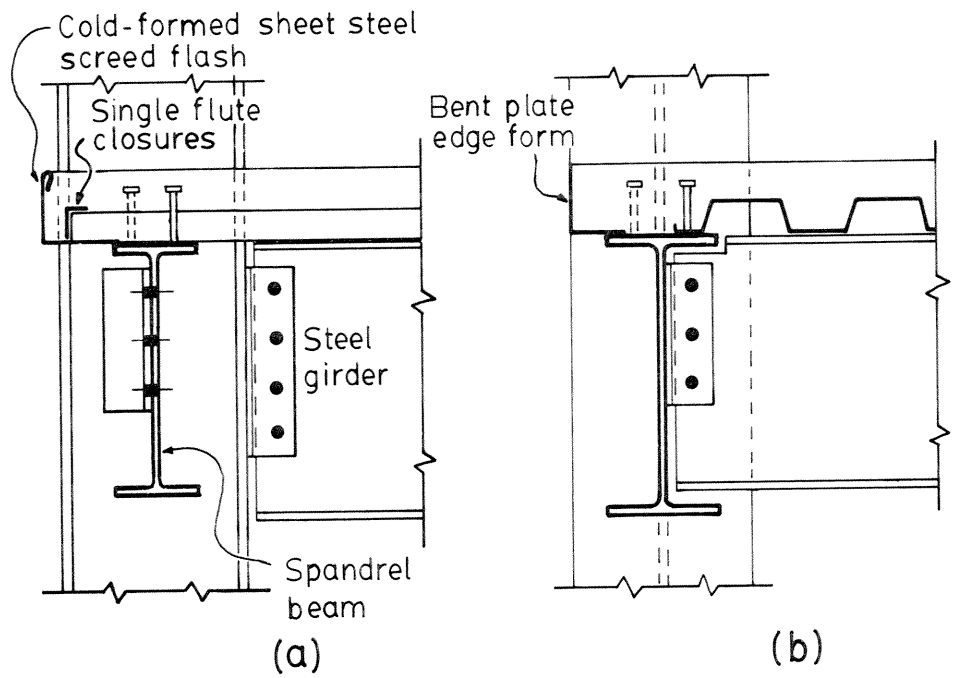
In composite floor or roof design, concrete slabs or cover slabs play an important role in providing structural strength to the overall framing system. In addition to structural functions, slabs or cover slabs provide a working surface for other trade work as well as a sub-floor for floor finishes such 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.



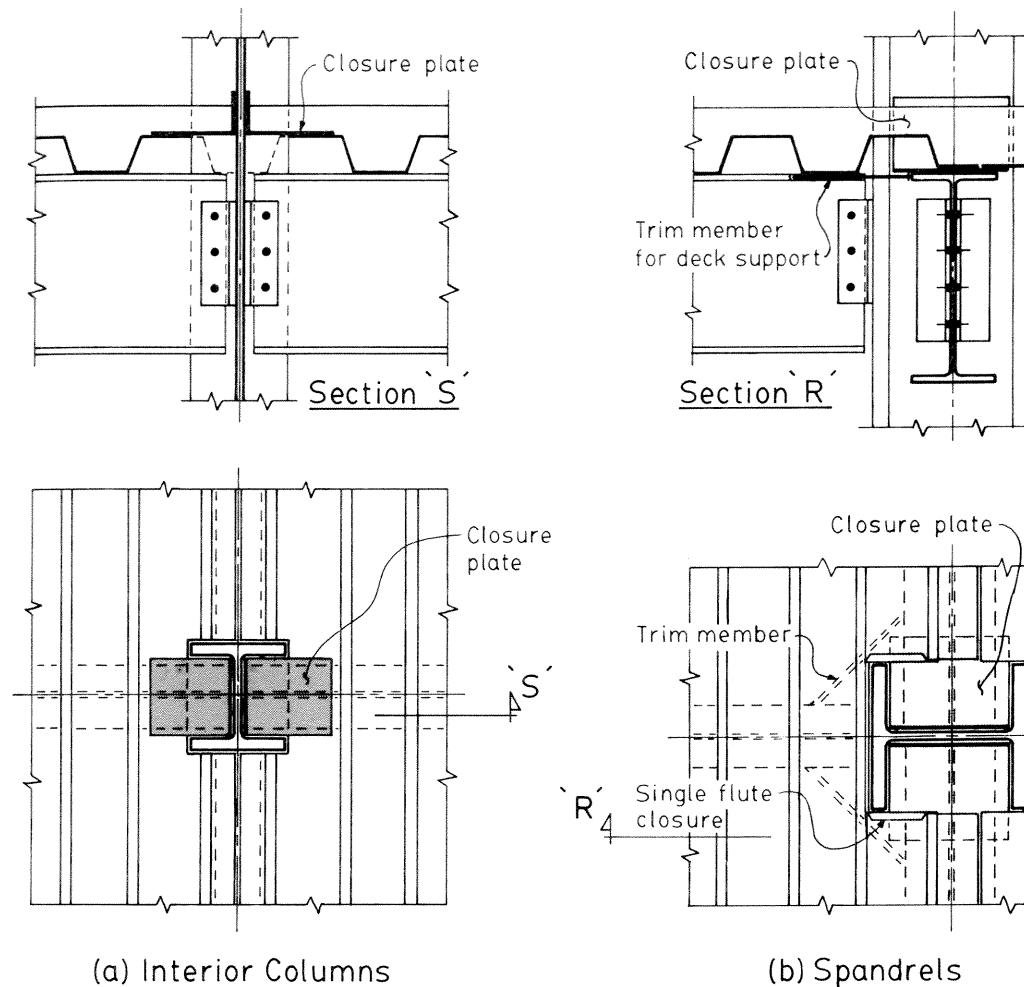
**Figure 1.5**  
**Deck Flute Closure Details**



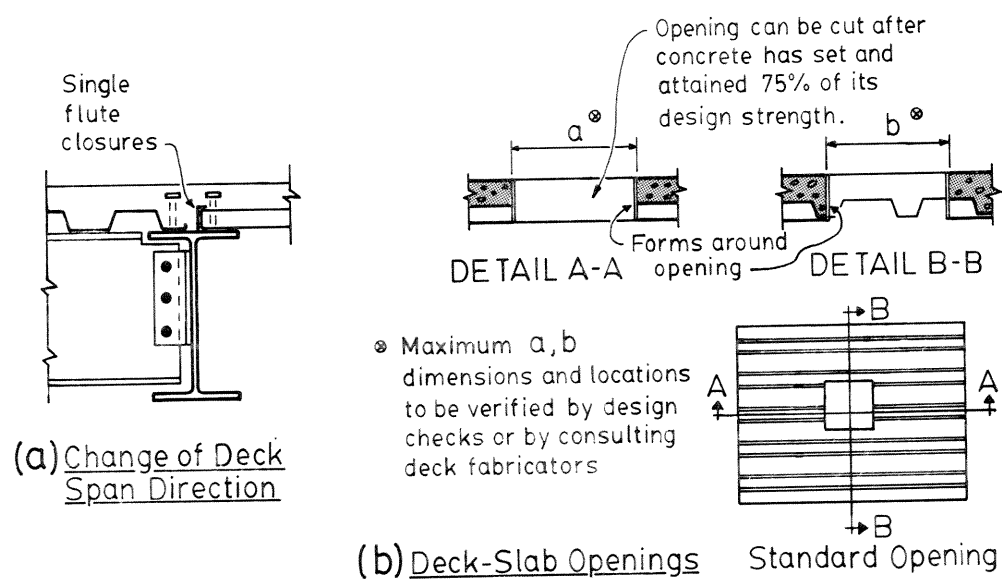
**Figure 1.6**  
**Deck End-Joint Details**



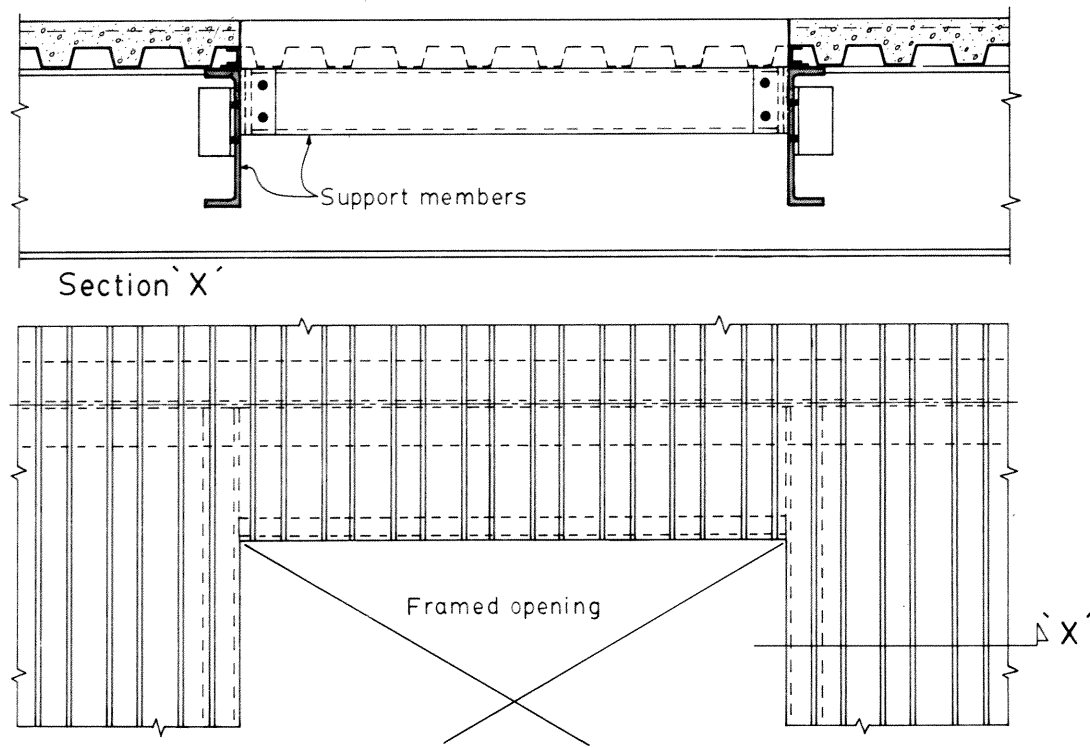
**Figure 1.7**  
Spandrel Edge Details Showing  
'Screed Flash' and 'Edge Form' Angles



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



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



**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.

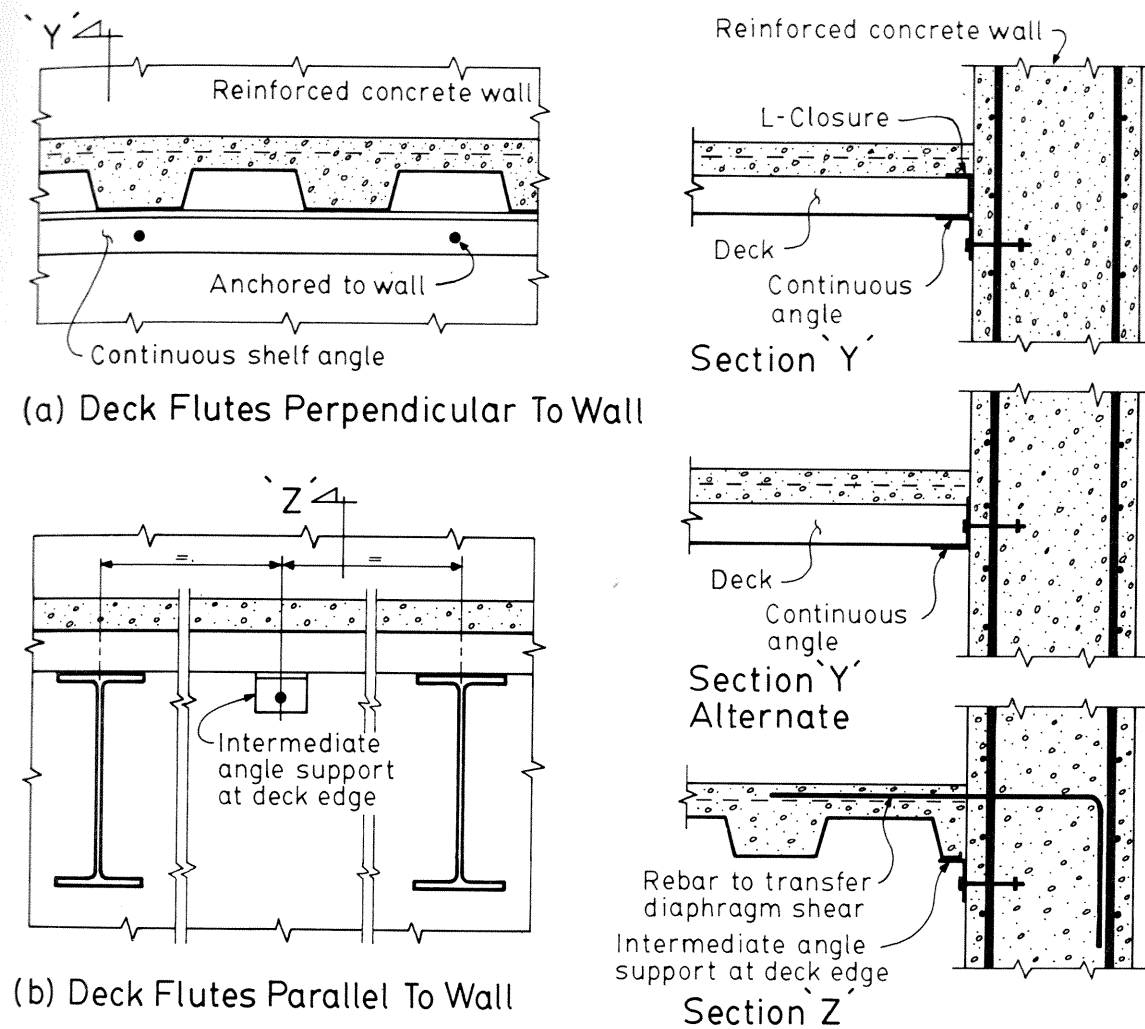
**Concrete** for slab or cover slab construction shall conform to Portland cement concrete in accordance with CSA Standard CAN3-A23.1-M Concrete Materials and Methods of Concrete Construction.

**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**,  $t_e$ , 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 times 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

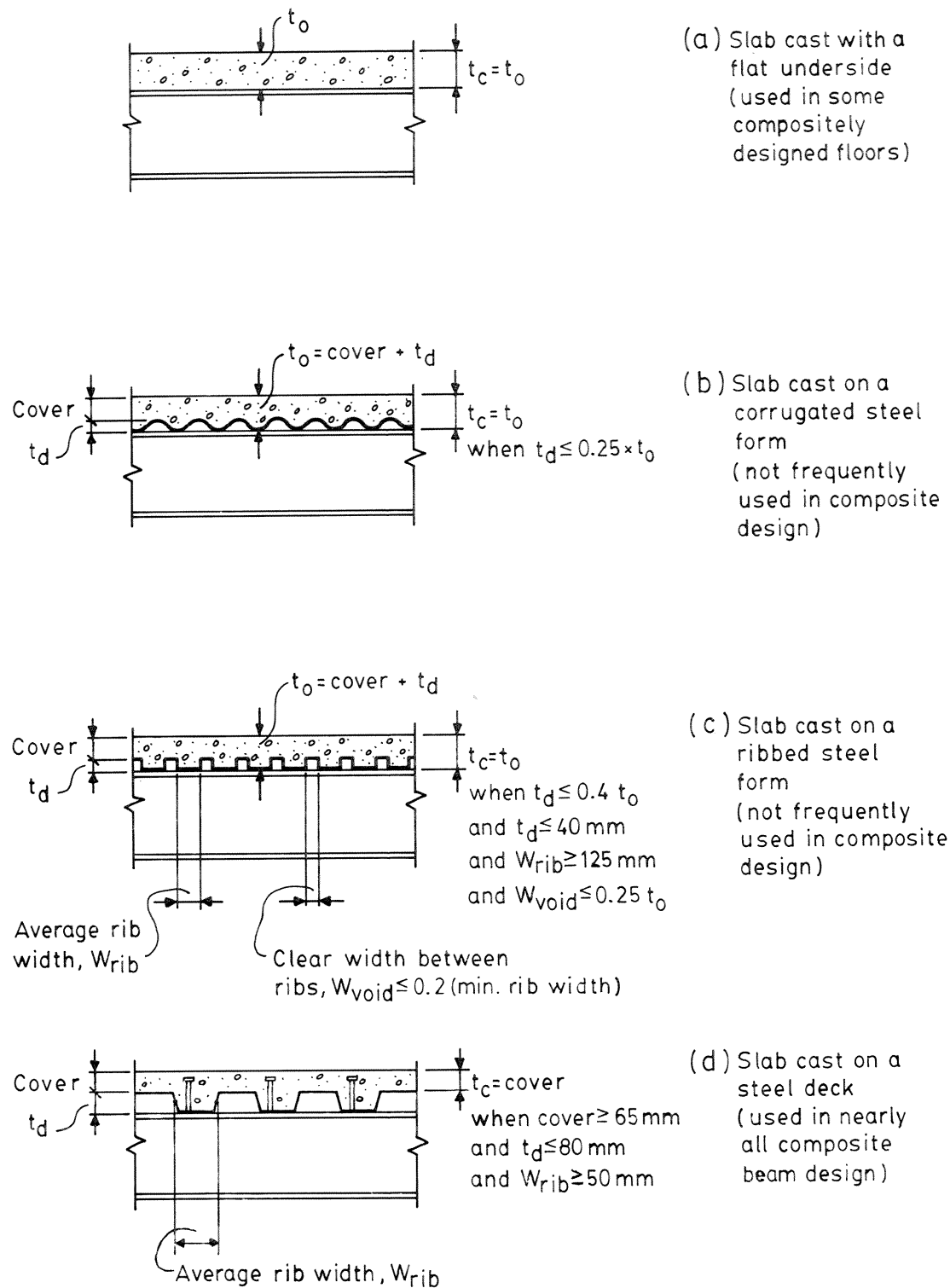


Figure 1.12  
Effective Cover Slab Thickness,  $t_c$   
for Composite Floor Member Design

- 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:

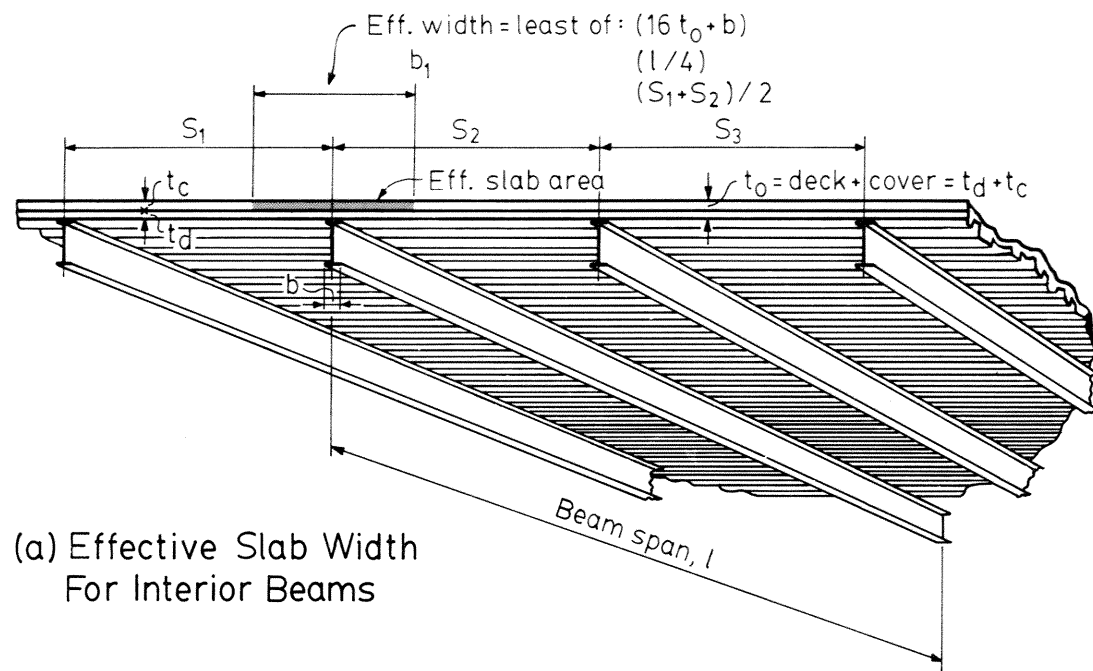
a) *Slab Thickness or Cover Slab Thickness*

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

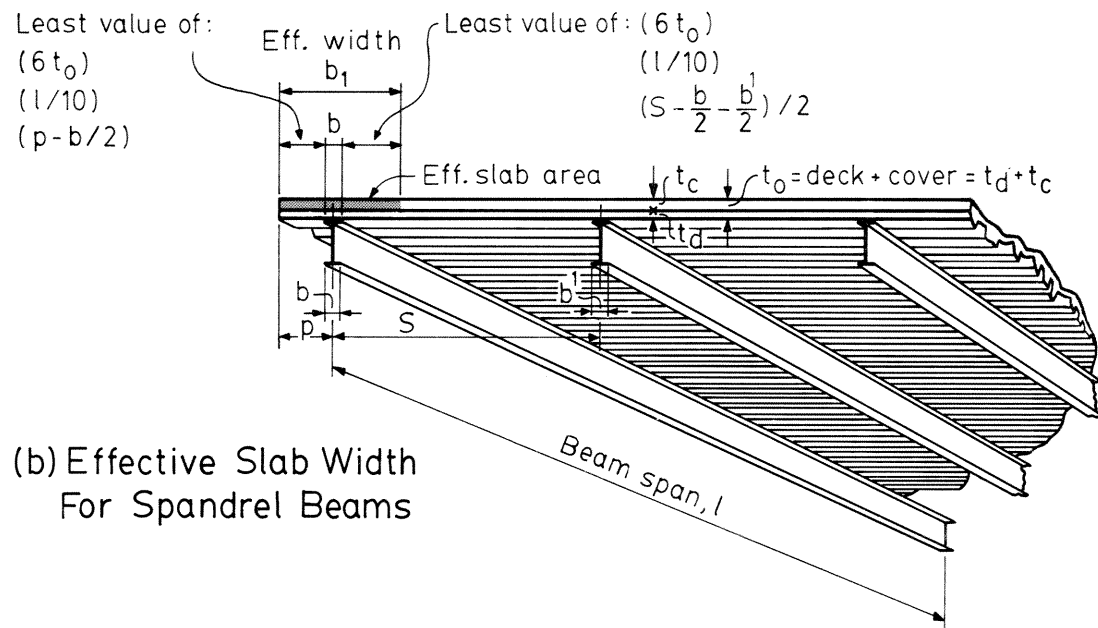
Items governing the selection of slab type and thickness include:

- 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<sup>(1.19,1.20,1.21)</sup>,
- the sound transmission and impact noise rating, and
- the floor to floor height and/or clear height of floors.





(a) Effective Slab Width For Interior Beams



(b) Effective Slab Width For Spandrel Beams

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.

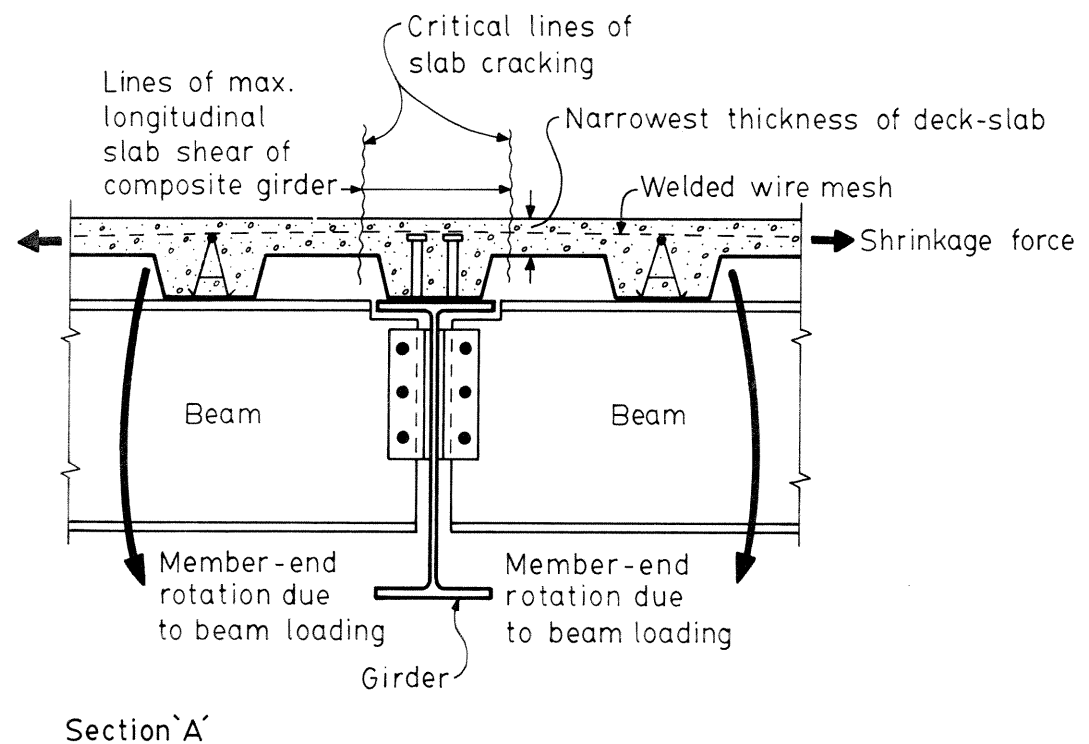
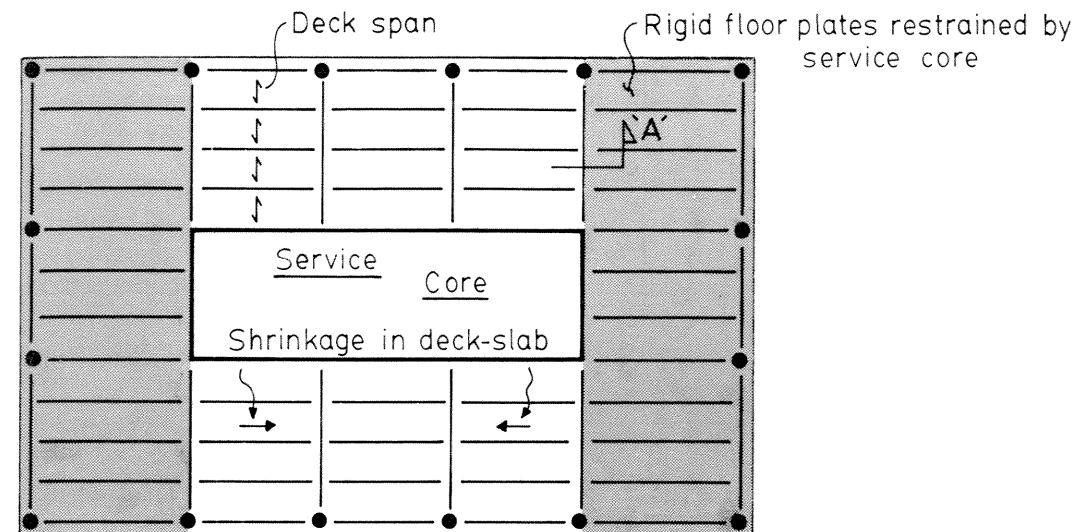
In a structural framing plan as shown in Fig. 1.17, composite deck supplemented by chaired mesh provides good crack control in the long direction of the structure. The relatively short dimension from core to exterior free edge (commonly 10-12 m) usually allows sufficient unrestrained shrinkage to avoid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

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

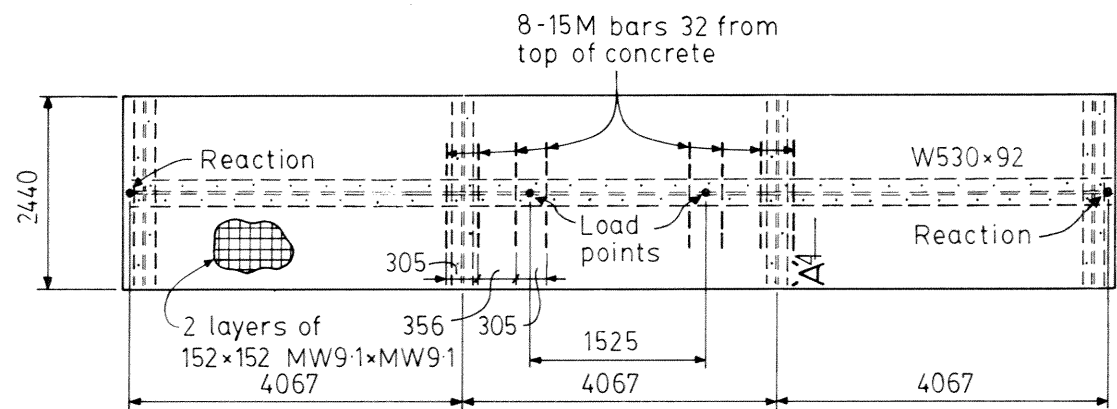
- the concrete cover slab is treated as structural concrete,
- there is no segregation of mixed concrete during slab pour,
- the concrete surface is not impaired by over-working during floating and trowelling by causing excessive amount of fines and water to be brought up to the surface, and
- the concrete 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*

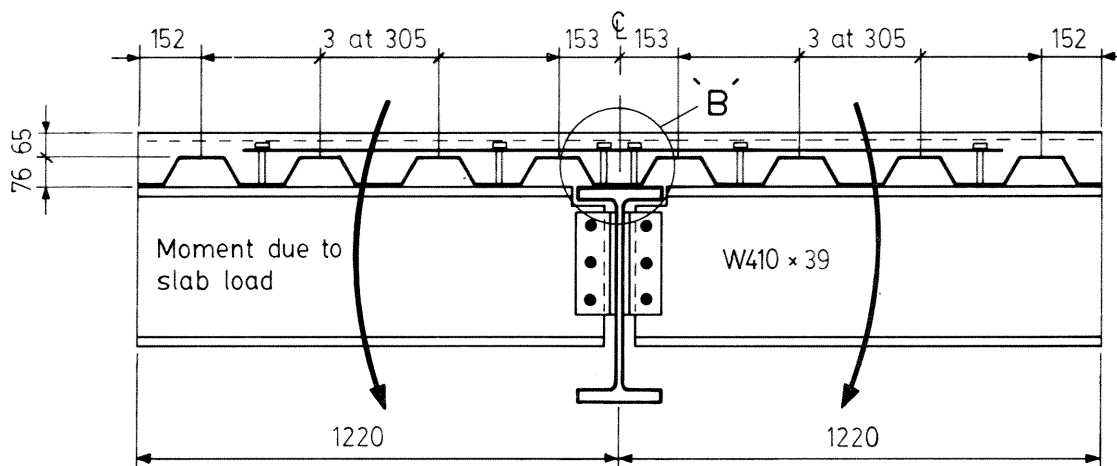
Two-directional considerations of composite construction incorporating a deck-slab system warrant some discussion. It is appropriate at this point to note the definition of a **girder** as a principal framing member normally supporting secondary **beams** and with the deck normally paralleling the girder. 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



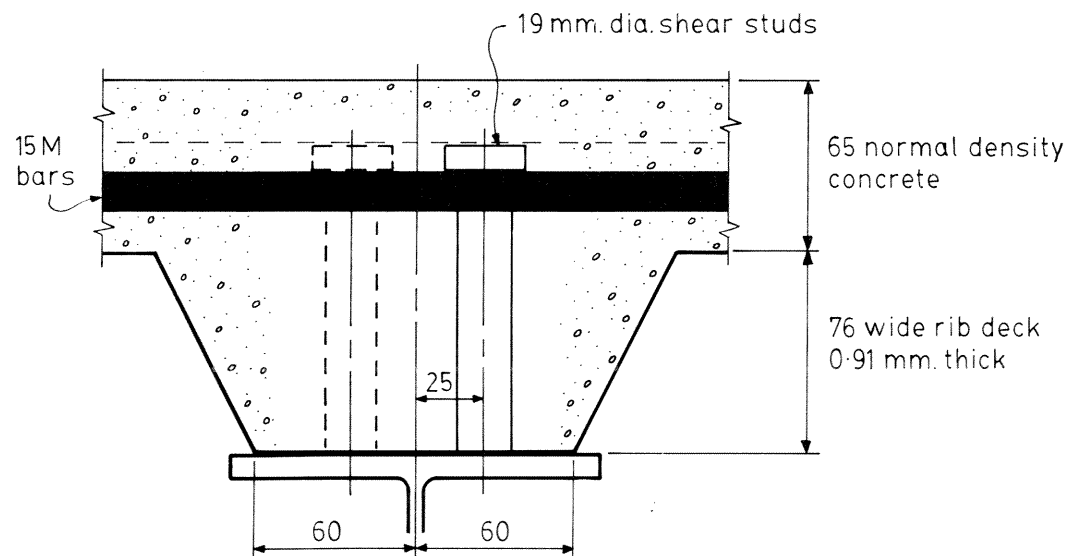
**Figure 1.14**  
Example Floor Plan Showing Locations  
of Stress Concentration in Deck-Slab



Reinforcement in Slab of Girder



Section A



Detail B

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<sup>(1.22)</sup> 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<sup>(1.19)</sup>.

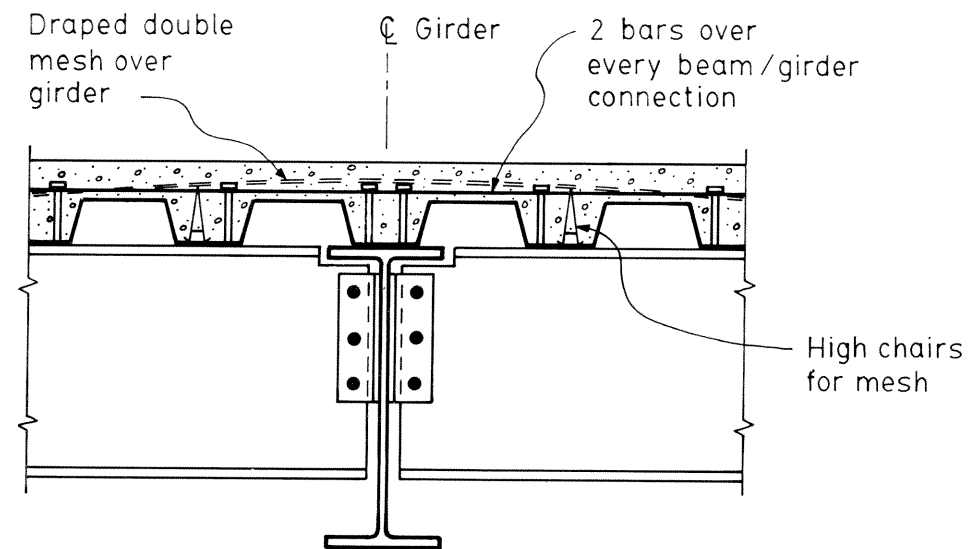


Figure 1.16  
Proposed Deck-Slab Reinforced at  
Beam-to-Girder Joints

– 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.

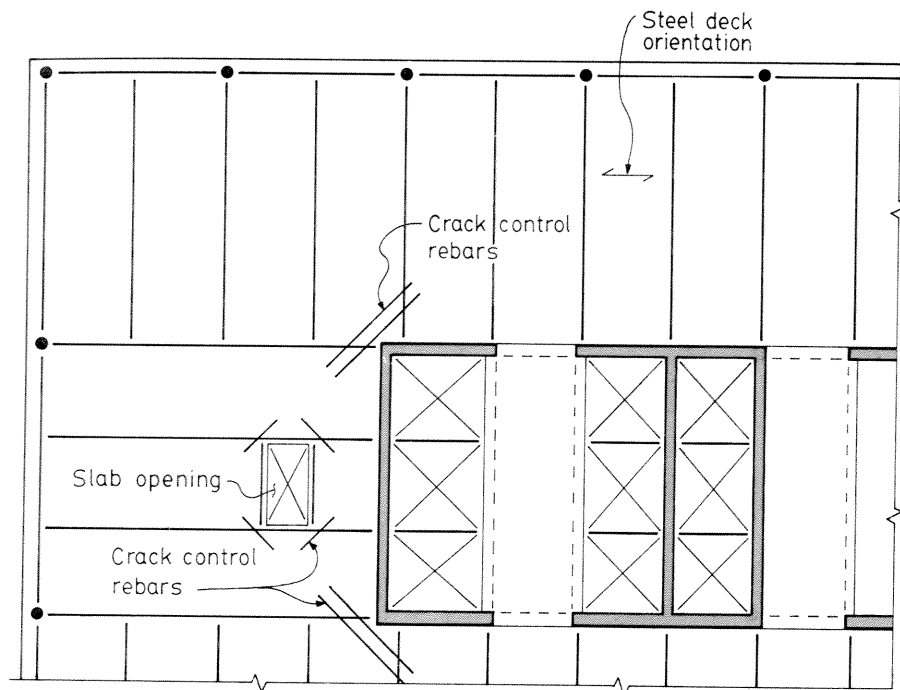


Figure 1.17  
Crack Control Rebars in Deck-Slab Floors

e) Other Considerations

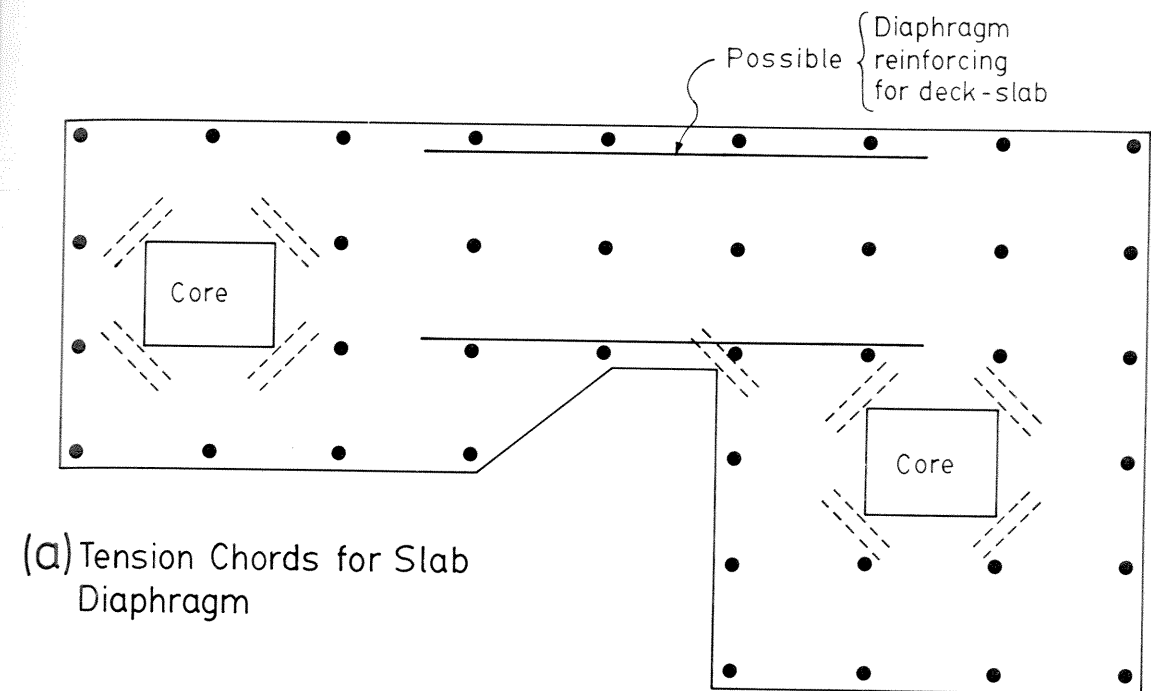
– Expansion (and contraction) joints in composite steel-concrete structures

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.

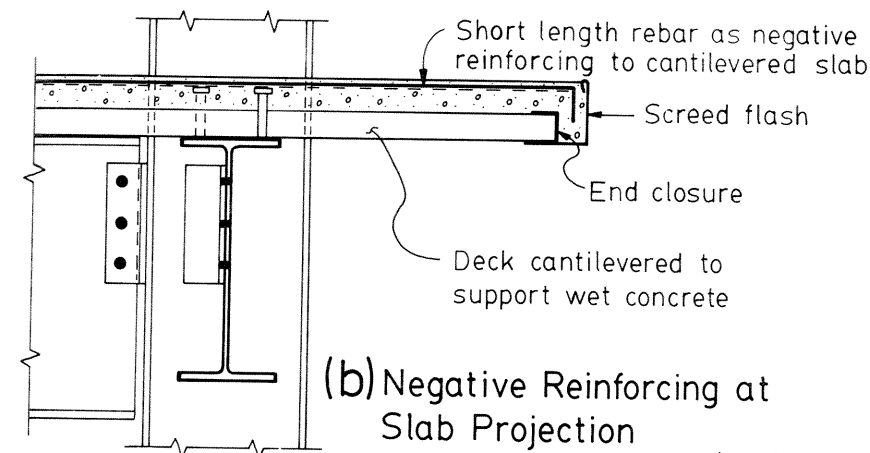
The introduction of suspended floors and an insulated roof membrane, as in more common-place structures, simplifies joint requirements in some respects and adds complexity in others. A steel skeletal frame to be fully enclosed and occupied at constant temperature will only be subjected to significant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

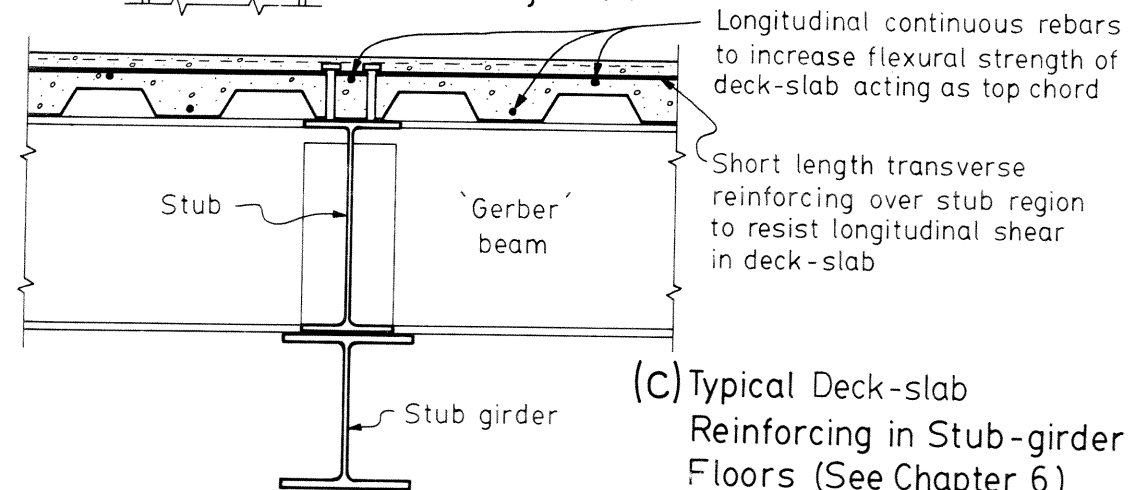
The steel skeletal frame is a known stable material with predictable thermal characteristics and with no time dependent dimensional changes. However, the introduction of concrete slabs or cover slabs adds a new dimension. Concrete shrinkage is inevitable, as discussed earlier. Some composite action in 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.



(a) Tension Chords for Slab Diaphragm



(b) Negative Reinforcing at Slab Projection



(c) Typical Deck-slab Reinforcing in Stub-girder Floors (See Chapter 6)

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  $600 \times 10^{-6}$  to a restrained shrinkage strain of about  $200 \times 10^{-6}$  to  $300 \times 10^{-6}$ . 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.

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## 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<sup>(2.1)</sup>. 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<sup>(2.2,2.3,2.4)</sup>, 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<sup>(2.5)</sup>, 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<sup>(2.6)</sup>. The following equation was derived:

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

where  $q_u$  = ultimate strength of a stud connector (N)  
 $A_{sc}$  = normal area of stud shear connector (mm<sup>2</sup>)  
 $E_c$  = 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

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

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<sup>(2.7)</sup>; therefore, a limiting value of  $415 \phi_{sc} A_{sc}$  (the tensile strength of the commonly available stud is 415 MPa), is given in S16.1.

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

where  $E_c$  may be expressed as

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

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

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		Mass Density of Concrete kg/m <sup>3</sup>	Factored Shear Resistance <sup>+</sup> , $q_r$ in kN for Various Concrete Strengths, $f'_c$		
in	mm		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

<sup>+</sup>  $q_r = 0.5 \phi_{sc} A_{sc} \sqrt{f'_c E_c} \leq 415 \phi_{sc} A_{sc}$   
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<sup>(2.9)</sup>. This work led to the publication of some comprehensive and design-oriented references<sup>(2.10,2.11)</sup>.

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<sup>(2.25)</sup>. 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<sup>(2.12)</sup>. 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<sup>(2.13,2.14,2.15,2.16)</sup> 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<sup>(2.17)</sup> through tests by Inland-Ryerson<sup>(2.18)</sup>.

### 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<sup>(2.17)</sup>. Using a factor of safety of 2.0 against flexural failure of test beams, Fisher proposed a stud shear connector allowable load formula,

$$Q_{rib} = 0.5 \frac{W_{rib}}{t_d} Q_{sol} \sqrt{\frac{E_{c-1}}{E_{c-n}}} \quad 2.5$$

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  
 $E_{c-1}$ ,  $E_{c-n}$  = 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,

$$q_{u(rib)} = 0.5 \frac{W_{rib}}{t_d} q_u \sqrt{\frac{E_{c-1}}{E_{c-n}}} \quad 2.6$$

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  $2 \times$  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<sup>(2.19)</sup>.

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

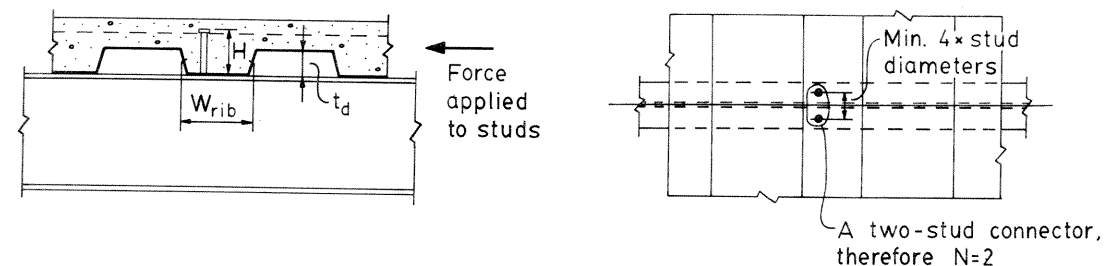
where  $N$  = number of stud connectors embedded in a concrete rib,  
 $H$  = height of stud connector,  
 $t_d$  = height of steel deck.

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<sup>(2.20)</sup> and further tests carried out at University of Texas<sup>(2.21)</sup> and at Lehigh University<sup>(2.22)</sup> verified the effects of multiple-stud grouping and stud embedment lengths.

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

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

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:

$$N = 1, (W_{rib}/t_d) = 2, \text{ wide-rib profile, } H = 120, t_d = 76 \text{ mm}$$

SOLUTION:

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

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

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.

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

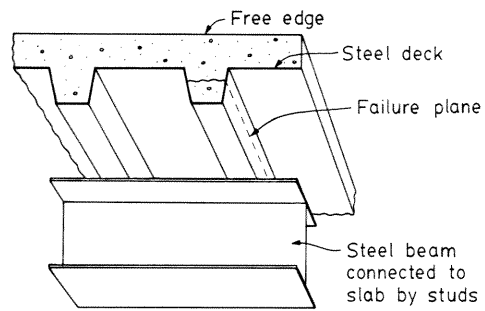


## 2.4 SLAB EDGE DISTANCES AND DISTANCES BETWEEN STUDS IN PAIRS

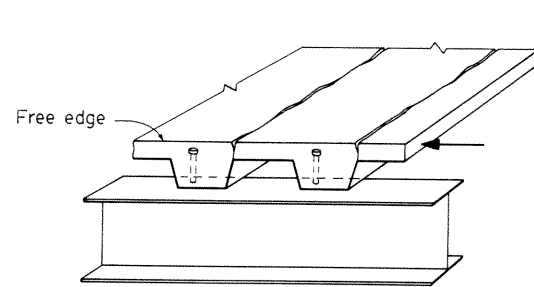
The discussion to this point has centered on studs applied singly on interior framing members. Every building has edge conditions, at the perimeter, at atriums, and/or interior stairwells, and at other conventional openings for vertical services. When compositely designed members are used under such 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

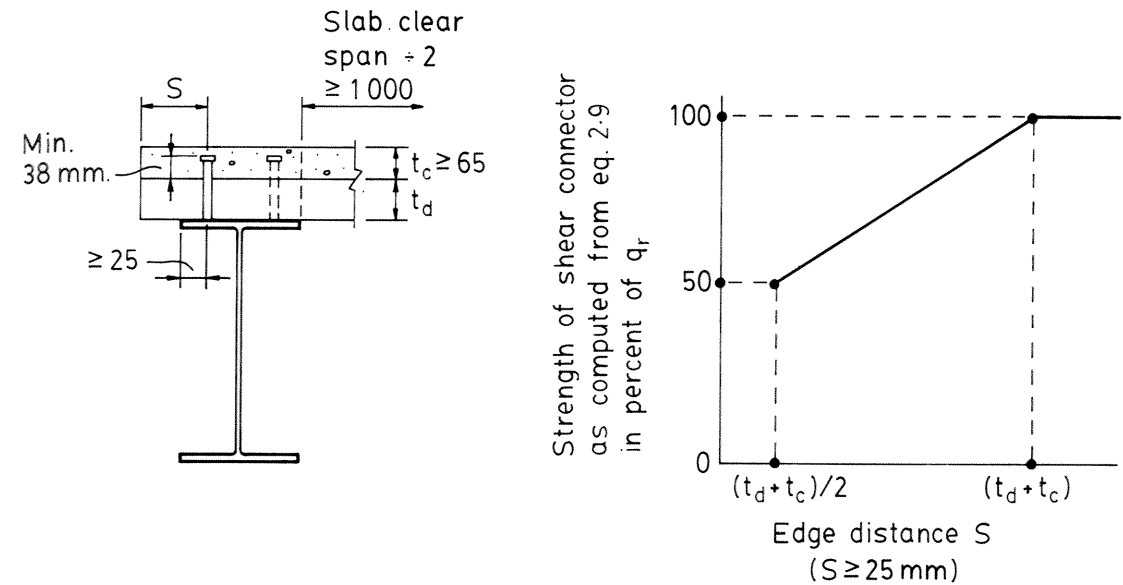


**Figure 2.3**  
Effect of Free Edge on Cover-Slab Strength

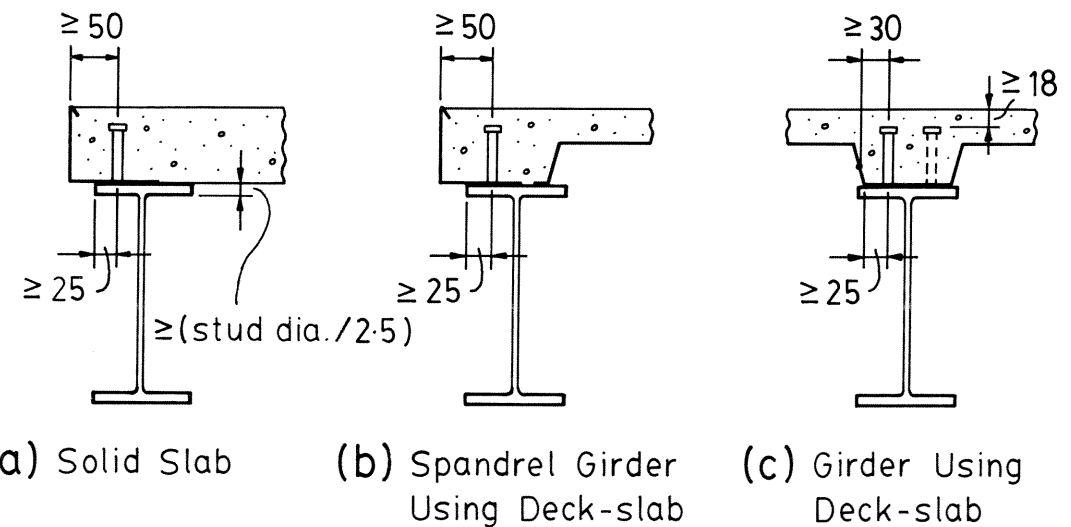
A comprehensive test program conducted by McMackin et al<sup>(2.9)</sup> to determine the behaviour of headed studs, when embedded in plain unreinforced concrete, provided sufficient data to reliably forecast anchor strengths of studs in both shear and tension. Design aids<sup>(2.10,2.11)</sup> provided by stud producers, facilitate the computation of ultimate anchor strengths for studs embedded in plain solid concrete. Recent push-out tests by Robinson<sup>(2.23)</sup> 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<sup>(2.30)</sup>. 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



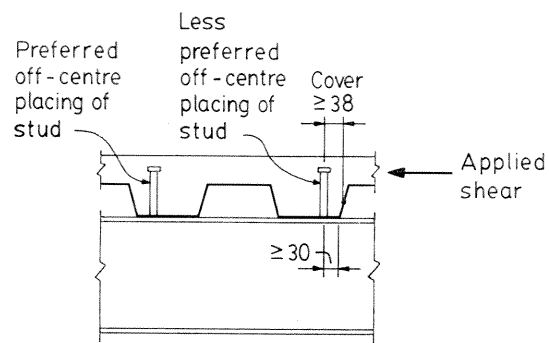
**Figure 2.4**  
Proposed Stud Shear Resistance in Hollow Composite Spandrel Beams



**Figure 2.5**  
Proposed Minimum Edge Distances for Stud Shear Connectors

tested by Grant et al<sup>(2.19,2.24)</sup> 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<sup>(2.23)</sup>. 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.



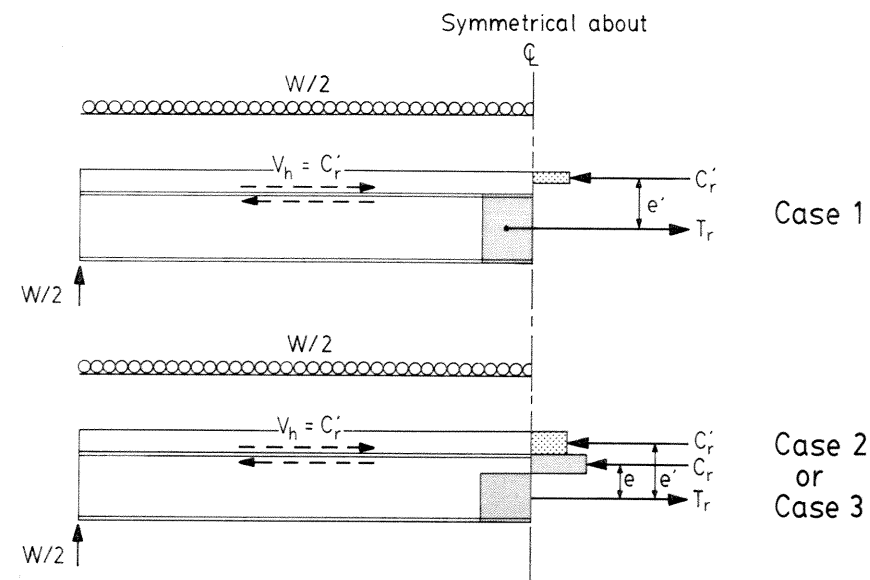
**Figure 2.6**  
Minimum Cover to Resist Punch-Through Failure of Stud Connection (for Single Stud per Rib Connections)

## 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<sup>(2.5)</sup> 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  $n'$ .

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

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.

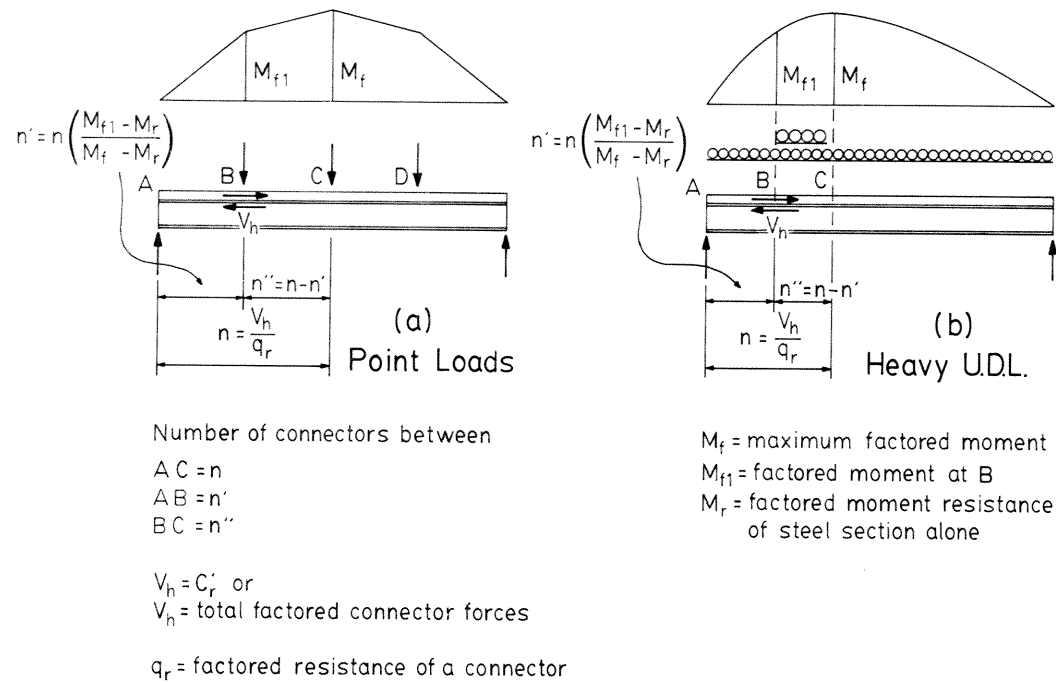
$M_f$  = maximum positive factored bending moment.

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)<sup>(2,29)</sup>.

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<sup>2</sup>, procedures recommended by the stud manufacturer shall be followed<sup>(2,27)</sup>.
- 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<sup>2</sup>). 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<sup>(2,28)</sup> 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.

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## 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:

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

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.

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

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

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

Where W = total slab load per span including ponding per metre width of deck (kN)

$w_c$  = mass density of concrete ( $\text{kg/m}^3$ )

s = deck span (m)

$I_d$  = moment of inertia per metre width of steel deck ( $\text{mm}^4$ )

q = Theoretical uniformly distributed slab load on unsagged steel deck (kPa)

Note: <sup>+</sup> 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<sup>(3.1)</sup>. 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.

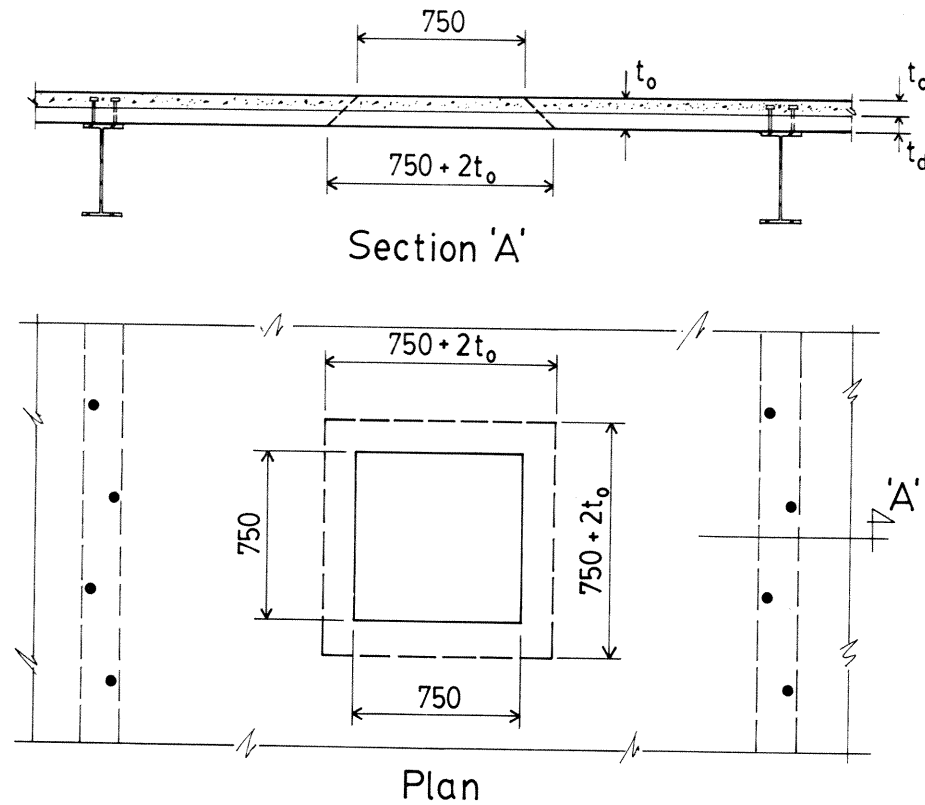
### 3.3 LOADING CONSIDERATIONS DURING CONSTRUCTION

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<sup>+</sup>**

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 $\leq 27 \text{ m}^2$ – for tributary area $\geq 54 \text{ m}^2$ – when tributary area is $> 27$ and $< 54 \text{ m}^2$ – concentrated load over an area $0.3 \times 0.3 \text{ m}$ , for beam area $< 16 \text{ m}^2$
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 $\leq 27 \text{ m}^2$ – for tributary area $\geq 54 \text{ m}^2$ – when tributary area is $> 27$ and $< 54 \text{ m}^2$ – concentrated load over an area $0.3 \times 0.3 \text{ m}$ , for beam tributary area $< 8 \text{ m}^2$

<sup>+</sup> 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 <sup>+</sup> - steel member (fresh-concrete condition) loading	- deck - concrete slab <sup>+</sup> - steel member  - other building finishing components if applicable	- deck - concrete slab <sup>+</sup> - 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**

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