Chapter 5

5.0 COMPOSITE OPEN WEB STEEL JOISTS AND TRUSSES

5.1 INTRODUCTION

Compositely designed trusses for floor framing systems have gradually developed into one of the "menu" selections available to the structural designer. They are particularly attractive in spans greater than 10 metres where members appropriately sized for strength and stiffness offer "free" web openings large enough to accommodate a substantial amount of air ducting as well as other services.

Their origin stems from a non-composite member, the open web steel joist(5.1) (OWSJ, or simply "joist") which may also be viable as a composite member. These members were developed as alternatives to solid web members and for reasons of economy. S16.1 defines OWSJ as simply supported steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems, proportioned to span between structural supporting members, providing direct top chord support for floor or roof deck. Joists were first fabricated in 1923(5.2), using top and bottom chords of double round bars and webs composed of single round continuous bent bars to form a Warren truss type of web. Since then, various other joist chord types, web types and web configurations have been developed as proprietary products and manufactured by individual joist manufacturers.

Composite Open Web Steel Joists

The structural action of an OWSJ acting compositely with a 1 640 mm wide concrete slab on corrugated steel forms was first studied and reported by Lembeck(5.3). The slab to joist shear connection in his test specimens was achieved by lowering the double angle top chord so that the web member ends extended above the top chord to be embedded in the concrete slab.

More studies on composite joists were reported by Wang and Kaley(5.4). The researchers evaluated two composite joists fabricated according to a structural system devised by the K-System, Inc., of New York City. Performance of the composite joists was compared with the results of two additional non-composite OWSJ tests. Composite action of the joists was achieved in this case by embedment of the top chords in solid concrete slabs, without the use of specific shear connectors.

Later work reported by Tide and Galambos(5.5), was initiated to determine the behaviour of stud shear connectors in composite open web steel joist framing systems incorporating solid concrete slabs 75 mm thick. Five different OWSJ specimens were obtained from several steel joist manufacturing plants. Stud shear connectors were welded to the top chords using standard installation equipment and inspection procedures. Many factors governing the behaviour of stud shear connectors during composite action of joists were identified. The study also showed that only the web system of OWSJs should be counted on to carry the total design shear, a fundamental concept also applicable to the design of non-composite joists.

The composite joist tests described above were similar in that the joist spans were short (approximately 4 to 6 metres) and the joists were shallow in depth (250 to 400 mm). In addition, the spacing of the composite joists tested was generally no more than 700 millimetres. The researchers verified that reliable composite interaction with concrete slabs could be achieved by various means.

Tests on composite long-span OWSJs utilizing deck-slab systems incorporating a 38 millimetre deep steel deck were reported by Stelco[5.6] and by Atkinson and Cran[5.7]. A two part report on the same research[5.7] reviews the results of two long-span OWSJ tests as well as several push-out tests of shear connections connected to typical joist chord members. More information on design considerations, and cost comparisons of composite versus non-composite joist floors, is provided in further documentation of the same research. As indicated in both references, there are several building design considerations which demonstrate that composite OWSJ floor systems in certain span ranges should be considered equally feasible as other steel framed floor systems. The reporters found that with joists spaced 1.5 metres apart and joist spans of more than 11 metres, compositely designed joists were more economical than non-compositely designed joists. Other building design considerations include:

- ability to provide long column free spans,
- ability to permit relocation of heating, ventilation and air conditioning ducts and sprinkler pipes,
- ease in accommodating new electrical and communication requirements,
- better plenum space utilization, and
- reduced storey heights.

Tests on six joists (composite with a deck-slab system) were carried out during the early seventies by Azmi[5.8]. The joists spanned 15.24 metres and two specimens were obtained from each of three different joist manufacturers. The joist chords selected included a cold formed hat shape, hot rolled double angles, and a hot rolled hat shape. The overall composite depth of the system measured approximately 960 millimetres, with a deck-slab composed of a 38 millimetre deep deck plus a 65 millimetre thick normal density cover slab. The width of deck-slab, acting as the top chord of the composite OWSJ, was measured at 1 524 millimetres. Different types of shear connectors and different degrees of connection were used. Test results correlated well with analytical results and an ultimate design method was derived which gave good agreement with the experimental results.

Two additional composite OWSJ tests were reported by Fahmy[5.9] and results were compared to the previous six test specimens reported by Azmi. Based on those eight tests, Robinson[5.10] found that composite joists with a deck-slab system not only gave better stiffness but also offered better ductility than a comparable non-composite joist. Researchers[5.11] also pointed out the need to check compressive resistances of steel top chords of composite joists when the amount of supplied shear connection falls below 75% of the full connection case. They had determined that with lower levels of connection, the compressive capacity of a top chord could be governed by the buckling strength of the steel chord.

Composite Truss Evaluation

Like a composite OWSJ, a composite truss is normally designed as a non-shored member during construction, again with composite action achieved through direct connection of steel top chord to concrete top flange by means of shear connections. The viability of OWSJ is based on mass production of a standardized product in jigs. To justify the use of composite trusses (or non-composite for that matter) as the floor framing component of a structural system, a project should be framed to permit a large number of similar trusses which can then be built using a jig, without the significant cost premium which would result from "short run" or "one-off" production. The use of simple joist-type connection details, without gusset plates, is essential for economy. A major difference between an OWSJ and a floor truss of this type lies in the type of structural components selected to manufacture the chord and web members. Floor trusses for composite design are manufactured using members selected by the designer from a relatively wide range of steel products available. Although the design approach used for composite interaction of deck-slabs with

OWSJs and trusses is similar, design applications for trusses are frequently beyond the capacity of conventional joists listed in a joist manufacturer's catalogue. Wider spacings and larger spans are the main features offered by a compositely designed truss system. The availability of wide-rib profile decks also makes a composite truss design a more viable solution for long span floor construction than that offered by joists.

A number of full scale composite truss tests have contributed to the development of a design methodology for this structural component. These tests were tied to specific projects and are reported in the following paragraphs.

As part of the development of an optimum floor system for the 110 storey Sears Tower in Chicago, composite trusses were tested to validate this method of construction. Iyengar and Zils[5.12] credit this floor system with a reduction in floor steel, increased flexural stiffness as well as with providing the long-span feature unique to this bundled-tube structure, without inhibiting the passage of mechanical ducts within the floor-ceiling sandwich. The composite trusses tested were spaced 4 570 millimetres apart with a span of 22.85 metres. The deck-slab system was constructed with a 76 millimetre deep composite wide-rib profile steel deck and topped with a 65 millimetre thick structural semi-low density concrete. A comparison of the test results with a theoretical analysis showed good correlation. The structural analysis was performed in two stages. For the wet concrete condition prior to the hardening of concrete, the truss was analysed non-compositely with all joints considered rigid. Effective design depth of the non-composite trusses was based on C.G. distances between the top and bottom tee chords. For the superimposed dead and live loads, composite properties of the truss with the deck-slab system were used.

A research test on a typical full-scale composite floor truss intended for use on a highrise project in downtown Edmonton, was conducted and reported on by Bjorhovde[5.13]. The span of the truss tested was 12 metres, with an out-to-out depth of 850 millimetres. The deck-slab system, connected to the truss by means of stud shear connectors, was composed of a 76 millimetre deep composite wide-rib profile steel deck and a 65 millimetre normal density concrete cover slab. The test showed full elastic response of the truss in the service load range, and the ultimate load exceeded the limit states design prediction by about 7%. Overall failure was precipitated by buckling of a compression diagonal and recommendations were drawn with regard to design and construction procedures.

Today, a composite open web steel joist (or truss) design is a fully recognized form of construction, with general slab design rules covered by the "composite beam" section of S16.1. A list of some U.S. and Canadian buildings using this form of construction is included in Table 5.1. For more information about Canadian examples, the reader is referred to a paper by Ritchie[5.14]. The following sections of this chapter are devoted to the specific design considerations and methodology of composite OWSJ and truss floor systems.

5.2 CHORD AND WEB STEEL SHAPES AND WEB FRAMING CONFIGURATIONS

Whether one is designing a joist or a floor truss (either composite or non-composite), the key to selection of economical and compatible chord and web member types, and web framing configuration usually lies in the ability to produce a "gussetless" truss assembly, suitable for a jigged production process during fabrication. (See Section 5.6 for information on connections and details.)

For composite truss design, chord members may be selected from steel products such as angles, tees, and rectangular or square hollow structural sections (HSS). At the same time, several steel products are offered for web member selection, i.e. angles, various types of HSS shapes, flats, and others. Only certain combinations of chord and web member types may be considered compatible for ease of jigging, ease of welding and therefore overall economy of production. See Table 5.2 (also see Section 5.6 for more details).

TABLE 5.1 SOME IMPORTANT CANADIAN AND U.S. COMPOSITE OWSJ AND TRUSS BUILDINGS

Project Name	No. of Storeys	Total Steel Content* GFA (kg/m²)	Remark
Oxford Square Towers, Calgary	33 and 37	45.0 (gravity steel)	Comp. OWSJ 11.8 m at 2030 c/c
Stelco Tower, Hamilton	26	36.8 (gravity steel)	Comp. OWSJ 12.4 m at 1520 c/c
Guardian Royal Exchange Tower, Toronto	23	47.8 (gravity steel)	Comp. OWSJ 12.5 m at 2290 c/c
Campeau Corp. Principal Plaza, Edmonton	29	40.4 (gravity steel)	Comp. Truss 12.0 m at 3000 c/c
Edmonton Centre 3, Edmonton	29	39.3 (gravity steel)	Comp. Truss 10.7 m at 2950 c/c
Edmonton Centre 5, Edmonton	32	40.8 (gravity steel)	Comp. Truss 10.8 m at 3050 c/c
The World Trade Centre, Twin Office Towers, New York	110	180 (gravity plus lateral steel)	Comp. OWSJ 18.3 m at 1006 c/c
Sears Tower, Chicago	109	161 (gravity plus lateral steel)	Comp. Truss 22.9 m at 4573 c/c

*Columns and supporting steel girders, etc, are included.

Three types of web-framing configurations are common in floor truss and joist designs. These are:

- Pratt,
- Warren, and
- Modified Warren.

A truss or joist design may be affected structurally and non-structurally by the selection of its web-framing configuration in the following ways:

- The efficiency of various web members in resisting vertical shear forces may be affected by the choice of a web-framing configuration, e.g., the selection of Pratt web over Warren web may effectively shorten compression diagonals resulting in more efficient use of these members.
- Web configuration may also influence the unsupported panel length of the compression top chord, affecting its ability to carry local bending moments, e.g.,the middle post in a modified Warren configuration provides mid-panel support to the top chord.
- Web to chord connection details may be partly resolved by selecting an appropriate web framing configuration, although in most cases also by selecting a deeper chord member.

TABLE 5.2 RECOMMENDED COMBINATIONS OF WEB AND CHORD MEMBER TYPES FOR TRUSSES OF A PRE-SELECTED WEB FRAMING CONFIGURATION

		WEB CONFIGURATION		
Truss* Type	Name of Component Member	Pratt	Warren	Modified Warren
1	– Top and bottom chords	SHS, RHS	SHS, RHS	SHS, RHS
	– Diagonals and Verticals	2L's	2L's	2L's
	– Modified Warren Web Posts			L, SHS, Round
2	– Top and bottom chords	Tees	Tees	Tees
	– Diagonals and Verticals	L or 2L's	L or 2L's	L or 2L's
	– Modified Warren Web Posts			L, SHS, Round
3	– Top and bottom chords	2L's	2L's	2L's
	– Diagonals and Verticals	SHS, RHS	SHS, RHS	SHS, RHS
	– Modified Warren Web Posts			SHS, RHS
4	– Top and bottom chords	SHS, RHS	SHS, RHS	SHS, RHS
	– Diagonals and Verticals	HSS	HSS	HSS
	– Modified Warren Web Posts			HSS

* Arranged in the ascending order of costs.
(Gussetless trusses are assumed)

- Availability of adequate web openings to accommodate ducts and other services may also be affected by this choice. Warren and modified Warren will usually permit larger web openings.
- The number of web component pieces handled during fabrication is yet another result of the web-framing-configuration selection.

Experience has shown that both Pratt and Warren configurations of web framing are suitable for short span trusses and joists with shallow depths. For truss or joist members with spans greater than 10 metres, or effective depths larger than 700 millimetres, a modified Warren configuration is generally more appropriate. Table 5.2 outlines the possible choices of compatible chord and web member types in relation to web framing configurations of composite truss members.

5.3 PROPOSED DESIGN CRITERIA FOR COMPOSITE OWSJ AND TRUSS MEMBERS

Unless otherwise noted, the following proposed design criteria are applicable to both composite OWSJs and composite trusses under generally uniform floor loading. These proposed design criteria, embodied in Sections 5.4 and 5.5, are in the form of descriptive discussions relating to design considerations concerning strength and serviceability requirements of such structural members. Basic structural assumptions and computational methods dealing with analysis (for the computation of factored design web or chord forces) are discussed and illustrated in an example in Section 5.8. Detailed formulas will not be given; designers are referred to the current relevant design clauses of S16.1 for the assessment of factored structural resistance of individual components.

To facilitate the steel cost comparison of a composite truss or joist floor with other floor framing systems, information provided by Reference 5.15 may prove to be useful.

5.4 STRENGTH DESIGN CONSIDERATIONS

Like an unshored hollow composite beam, a composite truss or joist must provide adequate factored resistance against collapse under loading conditions as follows:

– Deck placement,
– Concrete placement, and
– Occupancy loading.

Unlike a hollow composite beam, the factored resistance against collapse of a composite truss or joist depends on the factored resistance against failure of each individual component. During construction stages, particularly prior to concrete placement and again prior to full curing of concrete, the strength of the top steel chord must be evaluated. Under the occupancy load condition, the concrete top flange is assumed to participate structurally (in lieu of steel top chord) as required by S16.1. Although detailed design rules for web to chord connections have not been provided in this publication, the compatibility of web and chord members for the development of desired connection forces and for the development of economical connection details must not be overlooked. See worked example, Section 5.8.

Itemized discussions relating to strength design of the following component members are provided in the remainder of this section.

– Steel top chord member,
– Concrete deck-slab as a top flange,
– Steel bottom chord member,
– Web framing members, and
– Stud shear connectors.

a) Steel Top Chord Member

In composite design, the steel top chord member of a truss or joist is designed to carry construction loads prior to composite action. For OWSJ members, design of the top chord is governed by S16.1, Clauses 16.5.8, and 16.7; and for trusses, each panel of top chord steel member must be designed (under factored construction load condition) for axial and local bending effects based on actual conditions of restraint (provided by either additional steel support members, or through deck-steel direct connections). Structural analysis of construction load effects for truss types as described in Table 5.2 should be carried out assuming either pin-jointed or rigid-jointed web to chord framing.

Rules provided in Chapter 2, for stud shear connector design, require thickness of steel top chord material to be no less than stud diameter divided by 2.5. Otherwise, reduced stud shear resistance should be used during design.

Under occupancy loading, the steel top chord of a truss or joist is neglected in strength evaluations, except that the top chord should be treated as a horizontal shear transfer medium which collects horizontal components of web forces to be transmitted to the concrete top flange via shear connectors (normally studs).

To ensure overall member stability during construction, top chords of joists or trusses must be restrained against lateral movement through provision of direct structural support such as bridging or other means. Some truss chord configurations may have sufficient lateral stability to support steel deck and construction loads, permitting erection without bridging; however, adequate welding of the deck to the truss chord, before additional loads are carried, is essential.

b) Concrete Deck-Slab

The factored ultimate bending resistance of a composite joist or truss is computed using only the effective slab area of deck-slab system (excluding the steel top chord). Only the full shear connection case is to be used in computing the moment resistance. Refer to S16.1 Clause 17.4.3(a).

The effective width of concrete top flange for the analysis and design of a composite joist or truss

should be calculated using S16.1 rules (also as shown in Section 1.4 and Figure 1.13 of this publication).

Figure 5.1, illustrates the force equilibrium of a composite joist or truss cross section, assuming full shear connection. The plastic neutral axis (P.N.A.), in this case, lies within the depth of the cover slab. The factored compressive resistance (newtons) of the effective concrete slab is computed as $0.85\,\phi_c\,f'_c\,a\,b_1$, and the factored tensile resistance (newtons) of the steel bottom chord is computed as $\phi\,A_sF_y$, where,

ϕ_c = performance factor for concrete in flexure, 0.60
ϕ = performance factor for steel in flexure, 0.90
b_1 = effective width of concrete slab, mm
a = depth of concrete slab under ultimate compression, mm
f'_c = specified compressive strength of concrete at 28 days, MPa
A_s = cross sectional area of steel bottom chord; mm^2
F_y = specified minimum yield strength of steel bottom chord, MPa

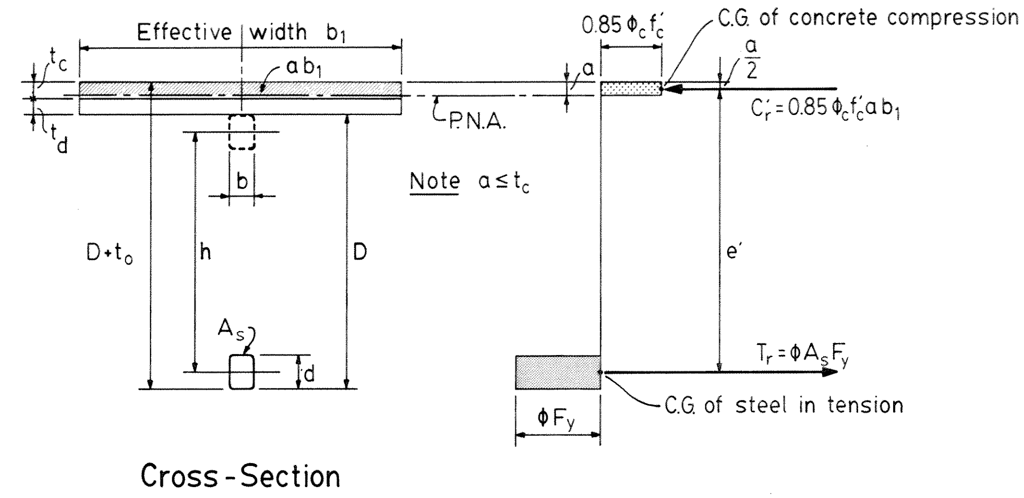


Figure 5.1
Force Equilibrium of Composite Truss (or Joist) Section

Equating factored compression to factored tension,

$$0.85\,\phi_c\,f'_c\,a\,b_1 = \phi\,A_s\,F_y$$

and solving for 'a',

$$a = \frac{\phi\,A_s\,F_y}{0.85\,\phi_c\,f'_c\,b_1} \qquad 5.1$$

and 'a' must not exceed t_c, i.e. Case 1, Clauses 17.4.2, 17.4.3(a)

The factored moment resistance of the composite section may be computed as

$$M_{rc} = e'\,\phi\,A_s\,F_y \qquad 5.2$$

where e′ represents the distance from C.G. of steel bottom chord to C.G. of concrete in compression.

c) Steel Bottom Chord Member

The prime function of the steel bottom chord is to provide the "tensile" component of moment resistance to a composite joist or truss under occupancy loading. However, during the selection of such a member, one must also provide adequate member stiffness to facilitate handling and erection.

Rules for strength and stability design of OWSJ bottom chords are provided by S16.1, Clauses 16.5.7, 16.6 and 16.7. Other strength considerations, when selecting chord members, include the availability of adequate depth of chord sections for the development of connection forces.

d) Web Framing Members

In composite design, web framing members of joists or trusses are proportioned to carry the total vertical shear as required by Cl. 17.3.1.2 of S16.1. In other words, factored design axial forces in web framing members under occupancy load may be conservatively analysed by resolving a statically determinate (pin-jointed) truss model neglecting any shear contribution from the concrete cover slab and steel chords. To illustrate the above, an example is provided in Section 5.8, showing the analysis method and rationale as well as the applications of S16.1 design clauses.

A joist or truss member rarely possesses true pin-joints, and local moment occurring at web-to-chord joints can be redistributed into web members. Strength and stability of compression web members may be affected, and it is prudent for a designer to include such design forces during the selection of web compression members. There are four main causes of local bending in web members:

- Floor loadings on equal or unequal top chord panels (Fig. 5.2).
- Web to chord joint eccentricity (Fig. 5.3).
- Connection eccentricity (Fig. 5.4).
- Localized overturning due to steel-to-concrete shear connection (Fig. 5.5).

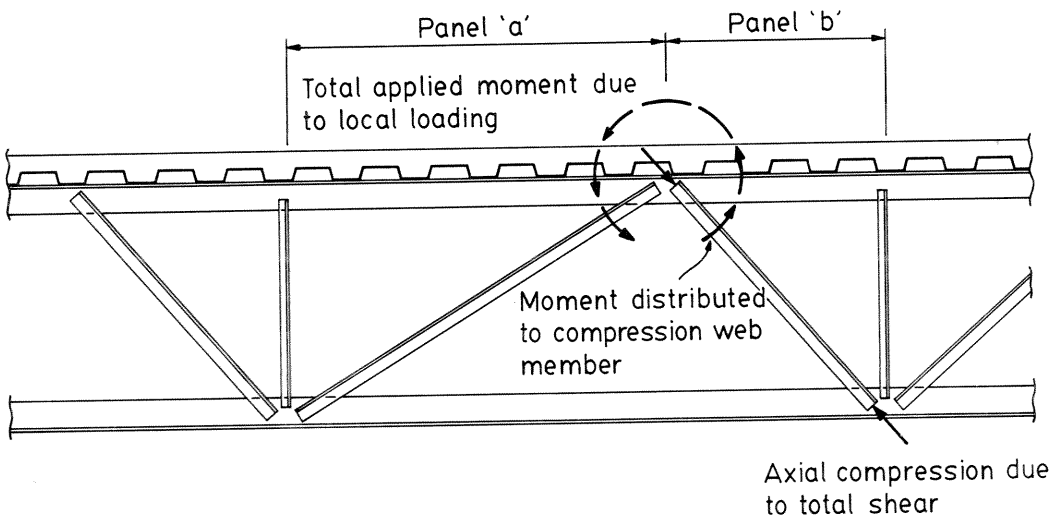


Figure 5.2
Induced Bending due to Floor Loads Acting at Top Chord

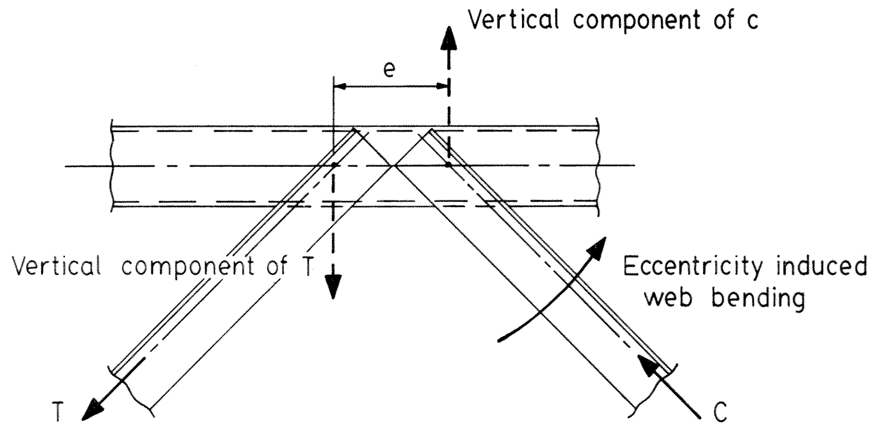


Figure 5.3
Bending due to Joint Eccentricity

The effects of local bending are generally neglected in the design of tensile web members due to the fact that such effects are often too small to affect member sizing. It is proposed that both OWSJ and truss web members in tension are required to meet a limiting slenderness ratio of 300 to facilitate handling and erection. In addition, analysis should be made to provide adequate compressive strength to tension members to account for stress reversal under patterned loading conditions and during handling and erection.

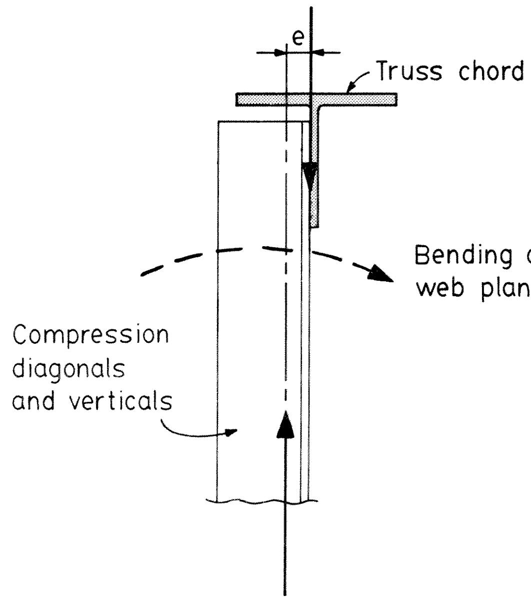


Figure 5.4
Induced Bending due to Connection Eccentricity

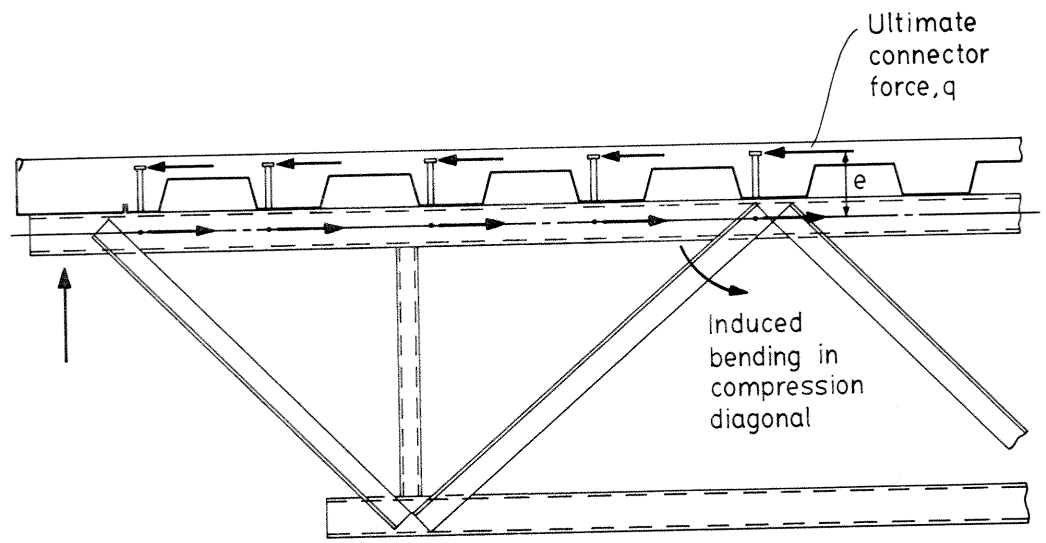


Figure 5.5
Induced Bending due to Localized Overturning of Stud Connections

e) Stud Shear Connections

S16.1 requires compositely designed joists or trusses to have full shear connectors between concrete top flange and steel top chord. Therefore the total horizontal factored shear of a composite joist or truss between the point of maximum bending and each adjacent point of zero moment can be represented as,

$$V_h = \phi A_s F_y \qquad 5.3$$

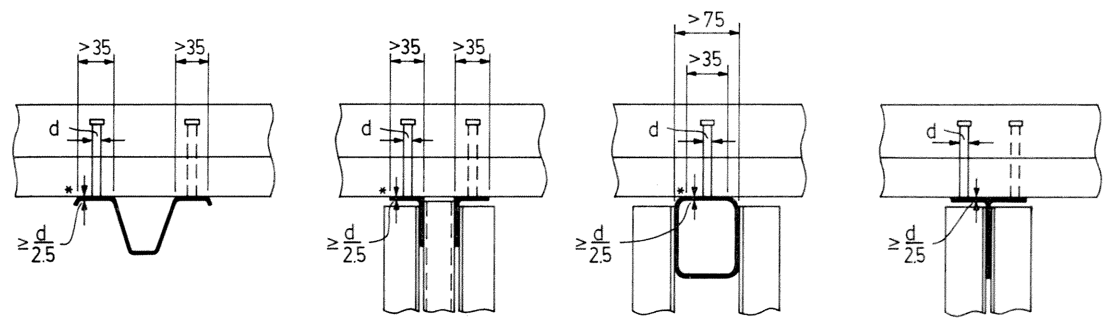
Stud shear connectors are commonly used to provide shear transfer at interfaces of steel and concrete. Factored shear resistances of studs in solid slab or deck-slab systems may be computed using the information provided in Chapter 2. When joist top chord segments between stud connectors are slender, it may be desirable for the designer to check the buckling strength of the top chord segments between shear connectors under the fresh-concrete condition loading.

Stud shear connectors must be distributed along the top chord member to enable a smooth transfer of interface shear (produced by the horizontal components of web forces transmitted through the steel top chord) to be carried by the concrete compression flange. Field welding of studs to steel joists or trusses through steel decks may be greatly facilitated if the stud-receiving flat width of a top chord component member is limited to not less than 35 mm, and the out to out width of a top chord is limited to not less than 75 mm. Figure 5.6 illustrates proposed limits of top chord dimensions for field welded stud application to OWSJs and trusses.

5.5 SERVICEABILITY DESIGN CONSIDERATIONS

The serviceability limit states design of a composite floor member often includes the following important considerations:

- deflection of steel member under fresh-concrete condition load,
- deflection of the composite member under occupancy live load and part of the superimposed dead load,



* Thinner flanges may be used, provided that stud shear resistance is reduced in design calculations and that flange materials are thick enough to prevent weld-through of studs during application.

Figure 5.6
Proposed Top Chord Selection Criteria to Facilitate Shear Stud Application

- floor vibration due to occupancy activity,
- deflection of floor member due to slab shrinkage,
- deflection of floor member due to creep of concrete top flange.

Member deflection under fresh-concrete load

The traditional method of fabrication for OWSJ is to provide camber by setting the manufacturing jig to an amount specified by Clause 16.5.15 of S16.1 unless otherwise specified by the building designer. The amount of camber suggested, in this case, is generally appropriate for joists of relatively light non-composite construction such as in floors where joists are closely spaced, or in non-composite roof construction, and would normally be inappropriate for compositely designed members.

In specifying compositely designed trusses or OWSJ for floor construction, it is advisable for the designer to indicate the amount of camber (usually equal to the elastic deflection of the non-composite truss or joist under the concrete slab load) to the fabricator. Thus, a "flat" floor is achieved, avoiding additional loads created by concrete slab "ponding" on deflected steel members (See Section 4.8).

Joist or truss deflection under fresh-concrete condition loads may be computed using sophisticated computer programs assuming rigidly connected web-chord members, or by a process of manual computation using a commonly accepted approximation method, such as the method described in S16.1 Cl. 16.5.14.2. For more detail, see worked example in Section 5.8.

Composite member deflection

Even though the deflection of composite joists or trusses under live loads is rarely critical when compared to the S16.1 recommended maximum values for deflections of floor members, it is still considered good practice to complete calculations to substantiate this fact.

Deflection of composite joists or trusses under occupancy loads may be computed manually or with the aid of a computer in the following manner:

Manual computation procedure:

Step 1 Compute effective slab area of the concrete top flange and transform into equivalent steel area.

Step 2 Compute gross moment of inertia of the composite member, I_t, using only the steel bottom chord and the transformed concrete top chord.

Step 3 Estimate reduction of moment of inertia of the steel member, I_r, using I computed from steel chord members and multiply the result by 0.15. (This is an estimated reduction of moment of inertia to account for the web's contribution of member deflection.)

Step 4 Subtract value obtained in Step 3 from the value obtained in Step 2. The resulting value is then divided by $(1+0.15+0.15)$ to allow for the effect of increased flexibility due to slip and creep.

Step 5 Compute composite joist (or truss) deflection using the moment of inertia value obtained in Step 4.

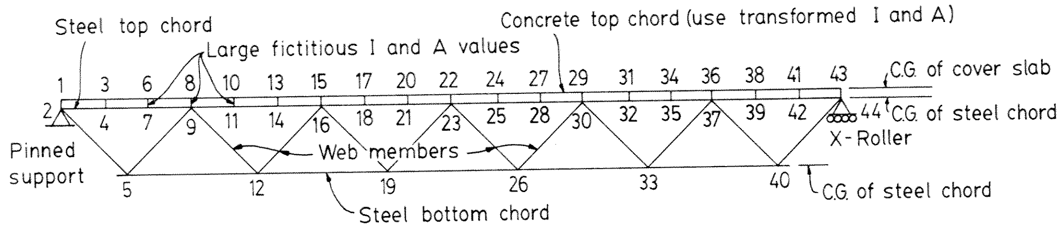


Figure 5.7
Composite Truss Modelling Technique for "Detailed" Structural Analysis

Computer stiffness analysis procedure:

Step 1 A composite truss (or joist) may be modelled as shown in Fig. 5.7. The effective concrete top flange is transformed to an equivalent steel area. The interconnection of concrete slab and steel top chord is modelled by short bar members having large moments of inertia and area.

Step 2 Analyse the model using a 'plane frame' type of computer program, to obtain member deflection under the specified floor loads. Note that the computed truss deflection need not be arbitrarily increased by a further 15 percent to account for slip and flexibility of concrete ribs formed by steel deck, due to the fact that some vierendeel effect was accounted for through the modelling technique. Likewise the computed truss deflection also includes its web contribution.

Floor vibration due to occupancy

Composite truss (or joist) floor systems provide strong and stiff floors in most instances. When such a system is used to support a large open area, free of partitions or other natural damping features, special consideration should be given to susceptibility to walking vibration to ensure that vibration characteristics are acceptable for the intended use and occupancy. The reader is referred to Appendix G of S16.1 for design information. A sample calculation of a composite truss floor vibration evaluation is demonstrated in Section 7.6.

Deflections due to creep and shrinkage

The latter two serviceability design considerations are often considered to be not critical in the design of composite joists and trusses for the following reasons:

- The composite joist and trusses covered by this chapter are generally deep, with the out-to-out steel member depths falling within the range of 1/17 to 1/11 of span. With such steel member depths, shrinkage and creep deflections of composite assemblies tend to be insignificant compared with the values one would find with a similar deck-slab on a shallower solid web steel beam.
- The composite joist or truss floors described herein are normally used for office occupancy. In such cases, the amount of sustained live load does not represent a significant portion of the total specified design live load. Hence, creep deflections also tend to be not critical.

5.6 TYPICAL CONNECTIONS AND DETAILS FOR TRUSSES

The key to economical production of trusses lies in the selection of compatible web and chord members to permit simple and direct connection details. Without the use of loose connection materials or gusset plates, members are produced on a mass production basis using a jig. When using a jig, the chord members are positioned with the desired camber to permit placement of one half of the web members on one side of the truss. Then, the entire assembly is turned over 180 degrees for placement of the other half of the web members.

In the case of trusses with WT chords and single angle web members, the process of web installation may be simplified, permitting a moderate saving in fabrication costs. For example, all web members may be placed on one side, then the truss "flipped" to add end diagonals, thus avoiding undue eccentricity at the truss end connection. Alternatively, the web system may be staggered. For example, in a Pratt configuration, all verticals may be placed on one side with diagonals on the other (but usually paired at the end) allowing easier fit-up and welding.

Although it is desirable that component members, connected at a joint, have their centroidal axes intersecting at a point, it is frequently unavoidable during shop-detailing. The joint eccentricity which is thus introduced, permits the production of an inexpensive joint fabrication detail. Such details could consist of web members free of angular clipped ends along with an easy-to-fabricate welding detail. Additional structural analysis and design are required to assure the adequacy of member and connection resistances, including the effects of joint eccentricity in the analysis. The joint eccentricity illustrated in Fig. 5.3 could be more readily tolerated at mid span of a compositely designed truss, whereas, use of a similar detail near the ends of the truss, could create greater concern of a shear failure.

The design of double angle web compression members often includes the consideration of whether battens or spacers are required. Heavy unbattened members or lighter battened members may be considered in selecting the most economical solution.

Truss types 1 and 2 (shown in Table 5.2) utilizing HSS top and bottom chords with double angle webs or WT top and bottom chords with single and/or double angle webs are generally considered to be best suited to the fabrication criteria as described in the last few paragraphs. Nevertheless, trusses fabricated with double angle top and bottom chords and an HSS web system continue to be strongly recommended by some fabricators from an economic standpoint.

Some typical web-to-chord connection details are shown in Figure 5.8. Typical end-support details for flange-top connections and web-framed connections are shown in Figure 5.9. To illustrate the capability of cantilever extensions using a compositely designed truss, the reader is referred to Figure 5.10. In Figure 5.11, typical details at a vierendeel opening are also provided.

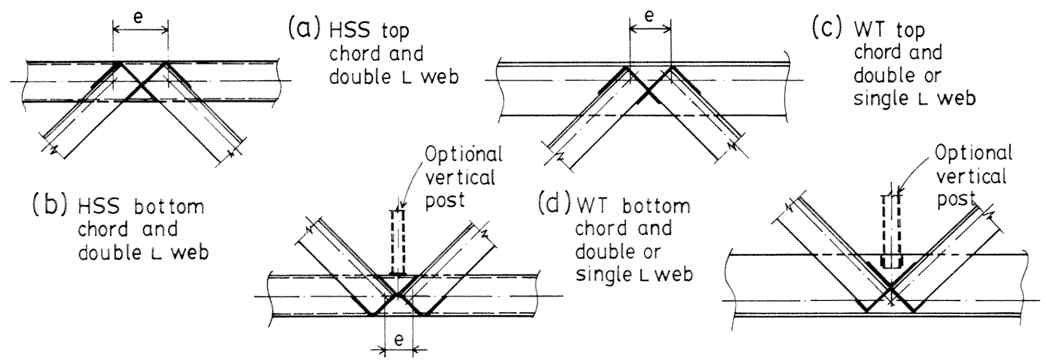


Figure 5.8
Typical Web-to-Chord Connection Details

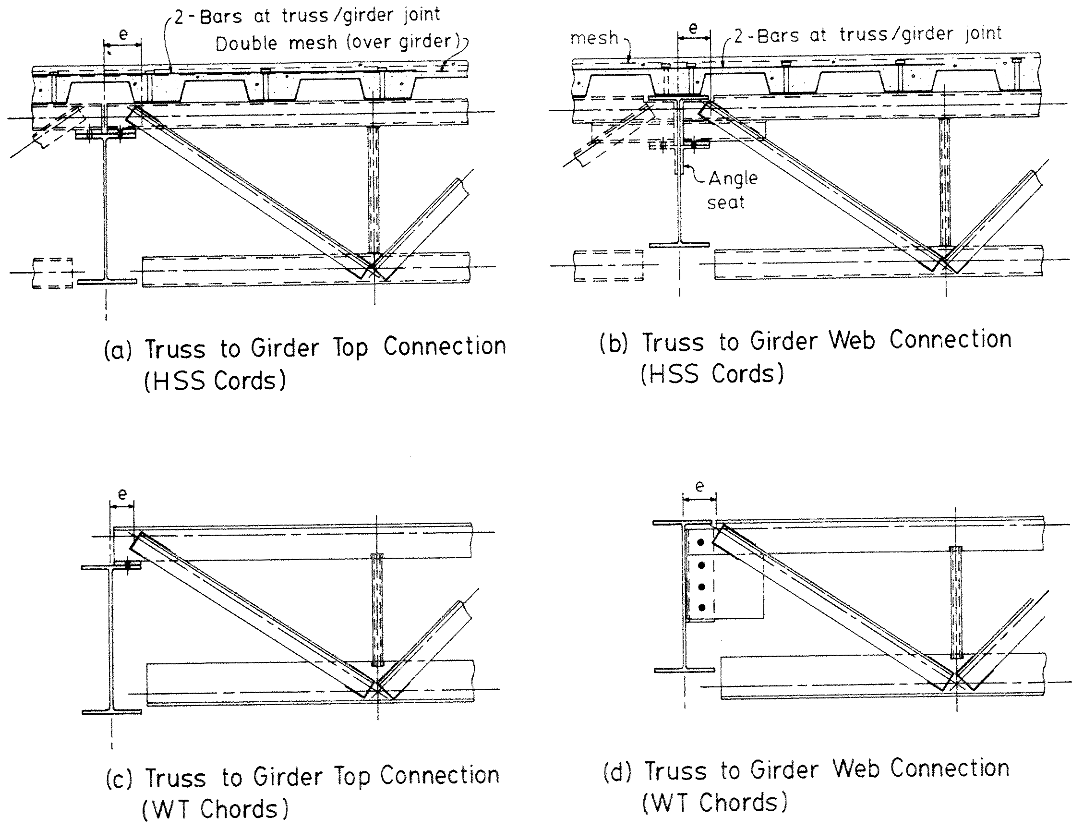


Figure 5.9
Typical Truss-to-Girder Connections

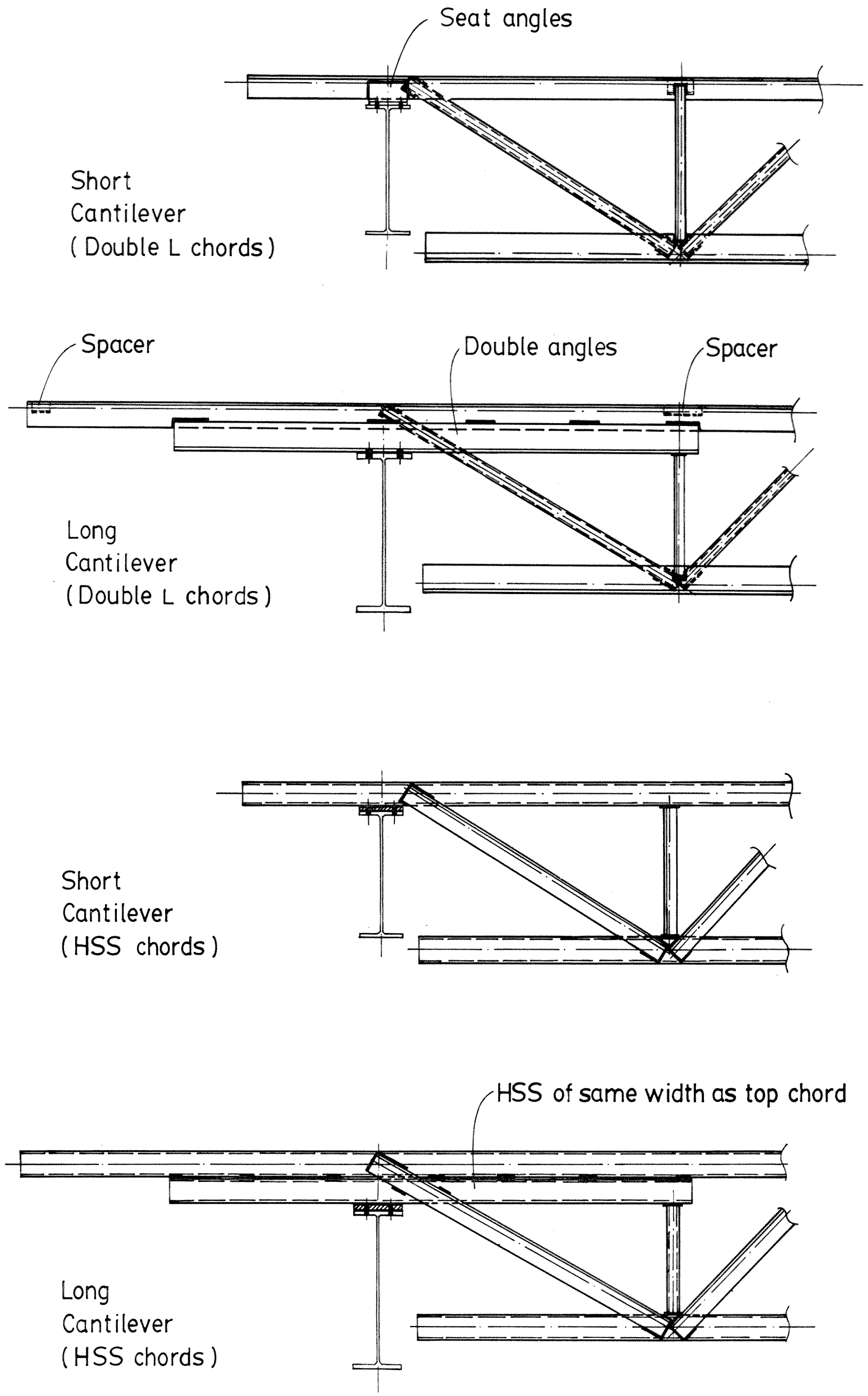


Figure 5.10
Typical Cantilever End Details for Composite Truss Design

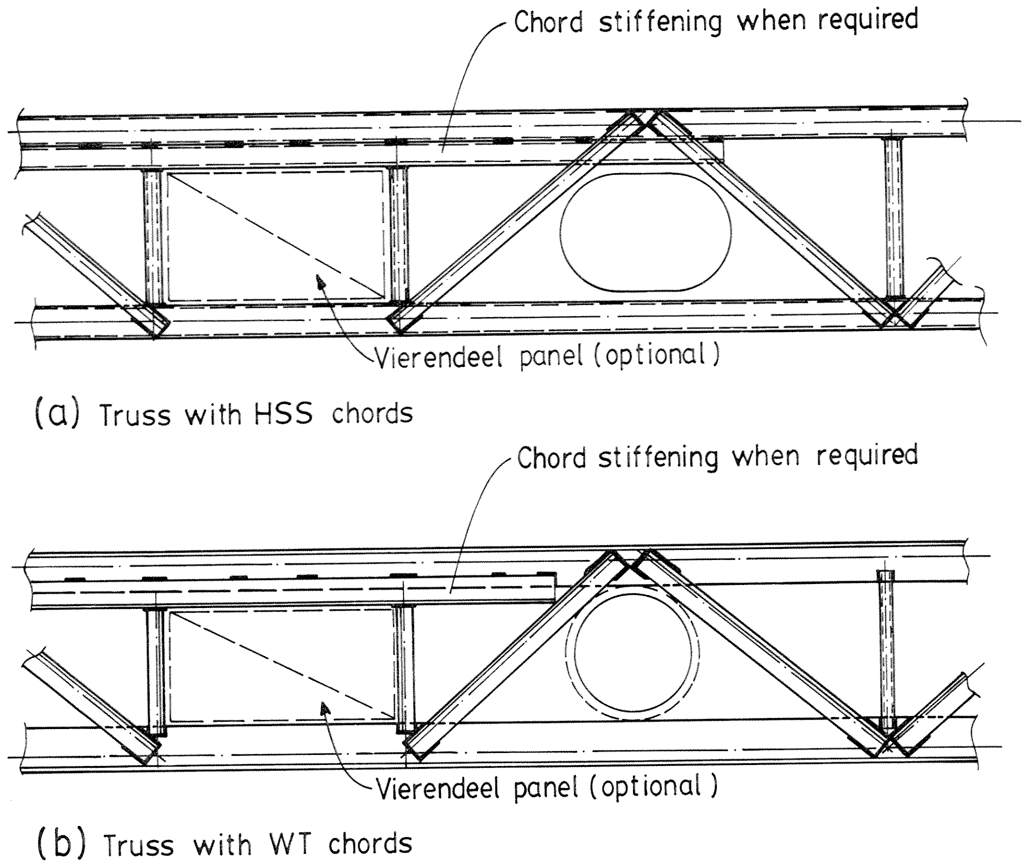


Figure 5.11
Typical Vierendeel Opening Details

5.7 COMPOSITE TRUSS MEMBERS – TRIAL SELECTION TABLES.

To facilitate the trial selection of truss component members, the following design tables are provided:

Table 5.3 HSS Bottom Chords
Table 5.4 WT Bottom Chords
Table 5.5 Class C HSS Top Chords
Table 5.6 Class H HSS Top Chords
Table 5.7 WT Top Chords
Table 5.8 Double Angle Tension Members
Table 5.9 Single Angle Tension Members
Table 5.10 Single Angle Struts* - with one leg welded to chords
Table 5.11 Double Angle Struts* - interconnected at mid length
Table 5.12 Class C HSS Warren Posts
Table 5.13 Class H HSS Warren Posts
Table 5.14 I_s/h^2 values – HSS chords
Table 5.15 I_s/h^2 values – WT chords

*Note: Angles in compression included in design tables belong to section classification up to 3 only.

The following is a detailed explanation of symbols which are used in the above mentioned design tables.

b_l = Effective width of concrete slab (Clause 17.3.2 of S16.1) used in computing values of M_{rc} and I_g, in millimetres.
C_r = Factored axial compressive resistance of a concentrically loaded web member, in kilonewtons. (Resistance to torsional-flexural instability of a double angle strut has been computed by means of an equivalent radius of gyration method.)
C_{re} = Factored axial compressive resistance of a single angle web member with one leg welded to chords, in kilonewtons (force resultant assumed to act at centroid of attached leg but half the moment caused by such eccentricity is included in the computation, K = 0.9) (for explanation, see 5.8 Floor Design Example).
D = Overall depth of steel truss, in millimetres.
h = Effective depth of steel truss (vertical distance between centroids of steel chords), in millimetres.
I_s = Moment of inertia of steel truss, 15% reduction due to open web included, in 10^6 mm^4.
I_g = Gross moment of inertia of composite truss, neglecting effects due to open web, deck profile and concrete creep, in 10^6 mm^4.
L = Truss span, in millimetres.
L′ = Laterally unsupported length of steel top chord while placing deck, in millimetres.
M_{rc} = Factored moment resistance of composite truss, in kilonewton metres (refer to Clause 17.4.3(a) of S16.1).
M_{rx} = Factored moment resistance about the x-x axis (as identified in Table 5.11), in kilonewton metres.
p = Top chord panel width defined as the horizontal distance between panel points (where top chord to 'Warren Post' intersection is also a panel point), in millimetres.
r_y = Radius of gyration about the y-y axis (as identified in the diagrams), in millimetres.
T_r = Factored axial tensile resistance, $T_r = \phi A F_y$, in kilonewtons.
V_h = Total horizonal shear to be resisted by shear connectors distributed between point of maximum moment and each adjacent point of zero moment, for full shear connection, in kilonewtons ($V_h = \phi A_b F_y$, where A_b is the bottom chord area).
V_r = Factored shear resistance, in kilonewtons ($V_r = 0.66 F_y \phi A_w$, where A_w = shear area).
w_{r1} = Factored top chord resistance to total u.d.l. while placing steel deck, in kilonewtons per metre.
w_{r2} = Factored top chord resistance to total u.d.l. while placing concrete, in kilonewtons per metre.

5.8 FLOOR DESIGN EXAMPLE

The designs of two composite truss configurations are illustrated in this section to represent a typical floor truss (T1 as indicated in the typical floor plan), shown in Figure 5.E1. To best illustrate the mechanics of the design process, the loadings and criteria are assumed to be the same as described in Section 4.14 except as noted below:

Storey heights given

floor to floor height	= 3 640 mm
floor to ceiling height	= 2 590 mm
plenum depth	= 1 050 mm
maximum horizontal duct depth (after deducting depths of chord members and stiffening member and sprayed fire protection material, Fig. 5.E1)	= 390 mm
Out to out depth of steel truss, D	= 730 mm

Solution:

Deck-slab system selected:

76 mm wide-rib profile deck manufactured by a different deck producer than the deck shown in Example 4.14. A normal density (20 MPa) cover slab of 65 mm thick is assumed.

$q_d = 0.1$ kPa $I_d = 1.25 \times 10^6$ mm^4 $q = 2.4$ kPa

$w = (1 + 0.2\, w_c s^4 / I_d)\, s\, q$ (see Table 3.1)

$= [1 + 0.2\,(2\,300)\,(3)^4 / (1.25 \times 10^6)]\, s\, q$

$= 1.03\,(3)\,(2.4)$

$= 7.42$ kN/m

$W_c = (7.42 + 0.6)\,(11.5) = 92.2$ kN (assumed steel member = 0.6 kN/m)

$W_L = 68.7$ kN ; see section 4.14

$W_p = 41.4$ kN ; see section 4.14

$W_{OD} = 24.2$ kN ; see section 4.14

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

$= 1.5(68.7) + 1.25\,(92.2 + 41.4 + 24.2)$

$= 300$ kN

$M_f = W_f L/8 = 300\,(11.5)/8 = 431$ kN·m

$V_f = W_f/2 = 150$ kN

Composite Truss with HSS Chords and Angle Webs

(a) Trial Section:

Bottom chord (HSS system)

Compute effective slab width (assuming width of top chord = 76 mm)

$L/4$	$= 11\,500/4$	$= 2\,875$ mm
$16t_o + b$	$= 16\,(141) + 76$	$= 2\,332$ mm (governs)
beam spacing		$= 3\,000$ mm

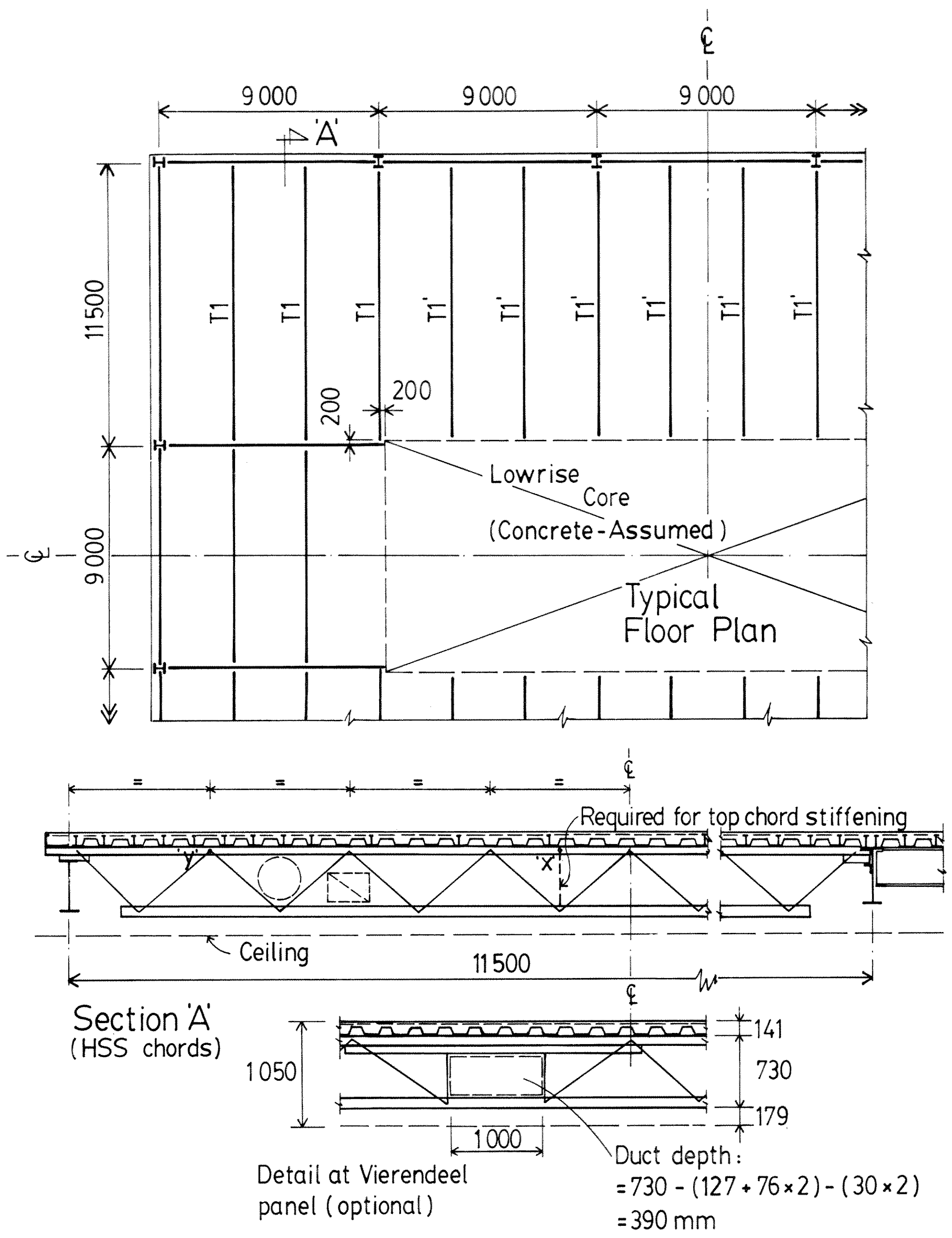


Figure 5.E1
Floor Design Example Key Plan
(Composite Truss Floor)

Therefore effective slab width, $b_l = 2\ 332$ mm

Using Table 5.3 – Composite Truss Bottom Chord
– trial selection table (HSS Bottom Chords),

for $D = 730$ mm $t_d = 76$ mm $t_c = 65$ mm
$b_l = 2\ 330$ mm and using **HSS127×76.2×4.78,**

and since tables do not provide a value for D = 730,

$$M_{rc} \text{ may be interpolated as } 460 - \frac{(460-404)(750-730)}{(750-650)} = 449 \text{ kN·m}$$

$$> 431 \text{ kN·m} \quad \text{OK}$$

$V_h = 564$ kN

$$I_g \text{ may be interpolated as } \left[1\ 020 - \frac{(1\ 020-784)(750-730)}{(750-650)}\right] \times 10^6$$

or $I_g = 973 \times 10^6$ mm^4

Top chord (HSS system)
A diagonal angle of approximately 45 degrees is desirable
11 500/(730×2) = 7.9 – use 8 panel points
Select top chord panel width at 11 500/8 = 1 437.5 mm

– Construction Stage 1 – Deck placement

$A = 11.5 \times 3 = 34.5$ m^2
27 m^2 < A < 54 m^2
$q_L' = 0.7 - A/135$ (Table 3.2 requires q_L' to be linearly
$= 0.44$ kPa varied from 0.5 to 0.3 kPa, when
54 > A > 27)

Dead load (deck + truss steel) = 0.3 + 0.6 = 0.9 kN/m
Factored total load at deck placement,
$w_{fl} = 1.25\ (0.9) + 1.5\ (0.44)(3) = 3.11$ kN/m

Using linear interpolation from Table 5.5, for trial top chord section **HSS76.2×76.2×6.35**, of truss span 11.5 m, lateral support of top chord at 1/3 span, panel width (p) = 1 438 mm, top to bottom chord centroidal distance (h) = 730−0.5(127+76) = 628.5 mm, p/h = 2.29, the values of w_{rl} and w_{r2} can be obtained as 5.15 and 12.9 kN/m respectively. See computation below:

The following calculation is intended to show how to obtain w_{r2} value from Table 5.5 by interpolation, when span = 11.5 m, h = 628.5 mm, p/h = 2.29

Span = 11 , h = 550 , p/h = 1 , $w_{r2} = 18$ (from table)
Span = 13 , h = 550 , p/h = 1 , $w_{r2} = 13$ (from table)

Interpolating, when span = 11.5 , h = 550 , p/h = 1 ,
then, $w_{r2} = 18 - (18-13)(11.5-11)/(13-11) = 16.75$ kN/m
Similarly, when span = 11.5 , h = 700 , p/h = 1 ,
$w_{r2} = 21.6 - (21.6-15.8)(11.5-11)/(13-11) = 20.15$ kN/m
and, when span = 11.5 , h = 550 , p/h = 2.5 ,
$w_{r2} = 13 - (13-9.8)(11.5-11)/(13-11) = 12.2$ kN/m

and, when span = 11.5 , h = 700 , p/h = 2.5 ,
$w_{r2} = 12.5 - (12.5-9.8)(11.5-11)/(13-11) = 11.83$ kN/m

Interpolating using values obtained above,
when span = 11.5 , h = 550 , p/h = 2.29
$w_{r2} = 16.75 - (16.75-12.2)(2.29-1)/(2.5-1) = 12.8$ kN/m
and when span = 11.5 , h = 700 , p/h = 2.29
$w_{r2} = 20.15 - (20.15-11.83)(2.29-1)/(2.5-1) = 13$ kN/m

again, interpolating for value of w_{r2}, when
span = 11.5 , h = 628.5 , p/h 2.29

$w_{r2} = 12.8 + (13-12.8)(628.5-550)/(700-550) = 12.9$ kN/m

Since maximum top chord force is calculated at location 'x', Fig. 5.E1, instead of at the mid-span, the values of w_{rl} and w_{r2} may be multiplied by the ratio (moment at mid-span / moment at location 'x'). Thus,

Factored u.d.l. for deck placement = w_{rl} = 5.22 kN/m, and
Factored u.d.l. for slab placement = w_{r2} = 13.1 kN/m.

Since w_{rl} (= 5.22 kN/m) is greater than w_{fl} (= 3.11 kN/m) (calculated above), the top chord is satisfactory for use under the deck placement loading.

– Construction Stage 2 – concrete placement

$q_L = q_L' = 0.88$ kPa
Dead load (truss steel + deck + concrete) = 0.6 + 7.42 = 8.02 kN/m

Factored total load at concrete placement,
$w_{f2} = 1.25(8.02) + 1.5(0.88)(3) = 14.0$ kN/m > (w_{r2} = 13.1)

Since top chord panel width may be modified by web framing changes, or connection details, HSS76.2×76.2×6.35 is kept for more detailed analysis, as shown later.

Web members (Double angle struts without spacers or battens)
Using tables 5.8 and 5.10, the web members (shown in Fig. 5.E2) are selected for the factored axial loads and are tabulated in the table below:

Web Member Reference	Factored Axial Force (kN) (−ve = tension)	Factored axial Resistance (kN) (C_r or T_r)	Section Selected using Tables 5.8 and 5.10 (Theoretical length 955 mm)
A	− 199	T_r = 224	2L45×30×6
B	+ 199	C_r = 201	2L55×55×10
C	− 143	T_r = 175	2L35×35×5
D	+ 143	C_r = 167	2L55×55×8
E	− 85.6	T_r = 121	2L25×25×5
F	+ 85.6	C_r = 112	2L55×55×5
G	− 28.6*	T_r = 121,	
		C_r = 18.9	2L25×25×5
H	+ 28.6	C_r = 44	2L35×35×5

*Note: Possibility of force reversal during erection (6 kN), from results of unbalanced load analysis.

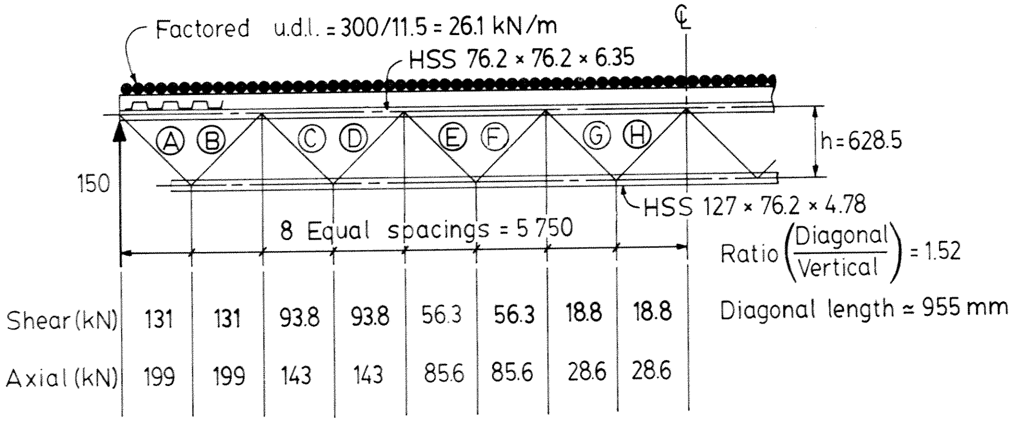


Figure 5.E2
Computation of Factored Web Forces for Preliminary Design (HSS chords)

(b) Truss Framing Layout and Truss Modelling:

Following the trial selection, a scaled layout of truss members is constructed (as shown in Fig. 5.E3) to ensure simplicity in connection details and to evaluate the amount of connection eccentricity at each web to chord joint.

With the scaled truss layout, structural analysis models may be constructed and analysed using a computer. Two basic truss models are constructed and analysed, as shown in Fig. 5.E4(a) and Fig. 5.E4(b):

- Non-composite steel truss model (see Fig. 5.E4a)
  (to be analysed under construction factored loads caused by deck placement and concrete placement)
- Composite truss model (see Fig. 5.E4b)
  (to be analysed under occupancy factored load and specified floor live load with superimposed dead loads) (Also see Fig. 5.E5 for exaggerated deflected truss shape)

(c) Detailed Member Design (bottom chord)

$$\text{Out to out composite truss depth} = t_o + D = 141 + 730 = 871 \text{ mm}$$

Using bottom chord section HSS127×76.2×4.78 (A_s = 1 790 mm^2), and checking moment resistance of composite section (see formulas 5.1 and 5.2, Fig. 5.1),

$$a = \frac{\phi A_s F_y}{0.85 \phi_c f'_c b_1} = \frac{0.9(1\,790)(350)}{0.85\,(0.60)(20)(2\,330)} = 23.7 \text{ mm} < t_c$$

$$\begin{aligned} M_{rc} &= e' \phi A_s F_y \\ &= [871 - (127+23.7)/2](0.9)(1\,790)(350)\,10^{-6} \\ &= 449 \text{ kN·m} > (M_f = 431 \text{ kN·m}) \\ V_h &= \phi A_s F_y = 564 \text{ kN} \quad \text{(same as before)} \end{aligned}$$

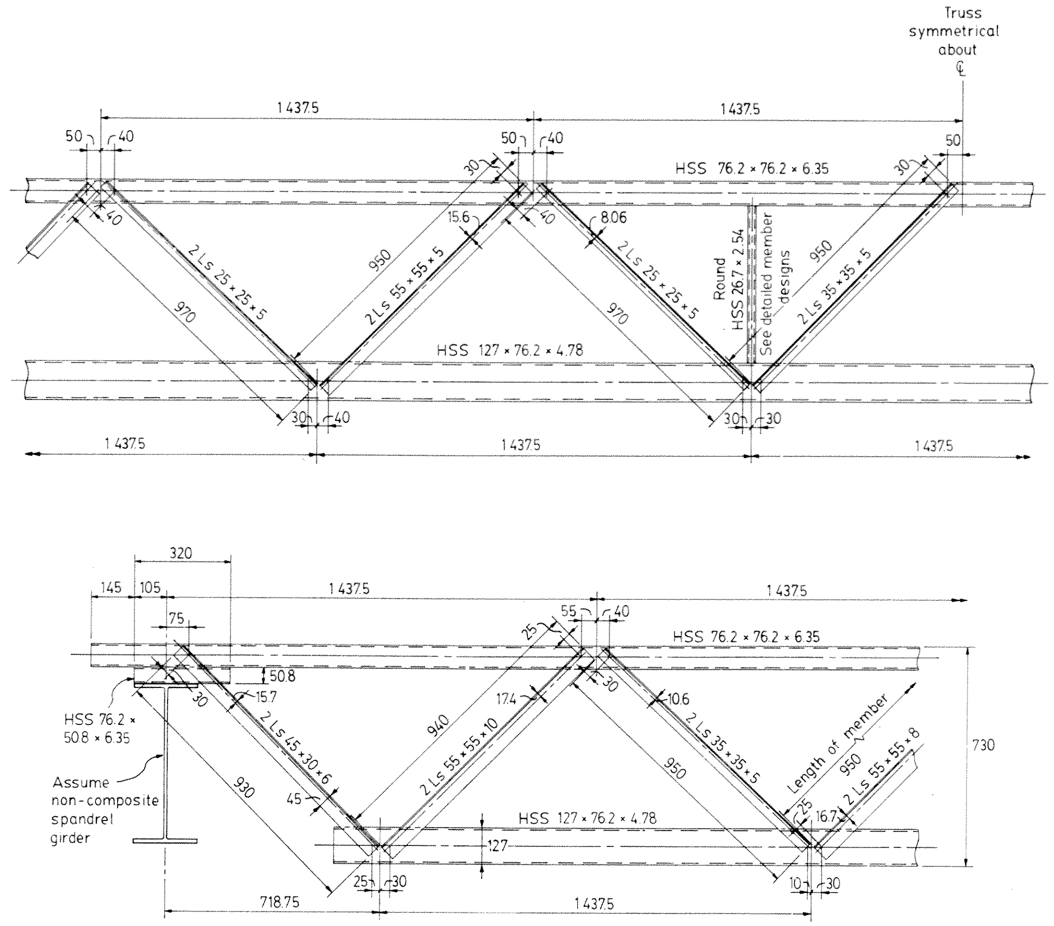


Figure 5.E3
Truss Framing Layout (Truss T1)
(HSS Chords)

(d) Detailed Member Design (top chord)

Five load cases are included in the computer analysis of the non-composite truss model:

Case 1 :	factored total u.d.l.	= 3.11 kN/m
Case 2 :	factored dead u.d.l. factored live load	= 1.13 kN/m, plus = 6 kN, acting at fourth panel of top chord from the left support and distributed for a length of 0.3 metre.
Case 3 :	factored total u.d.l.	= 14.0 kN/m
Case 4 :	factored dead u.d.l. factored live load	= 10.0 kN/m = 6 kN, acting at fourth panel of top chord from the left support and distributed for a length of 0.3 metre.
Case 5 :	specified dead load (at concrete placement)	= 8.02 kN/m (results will be discussed in (f))

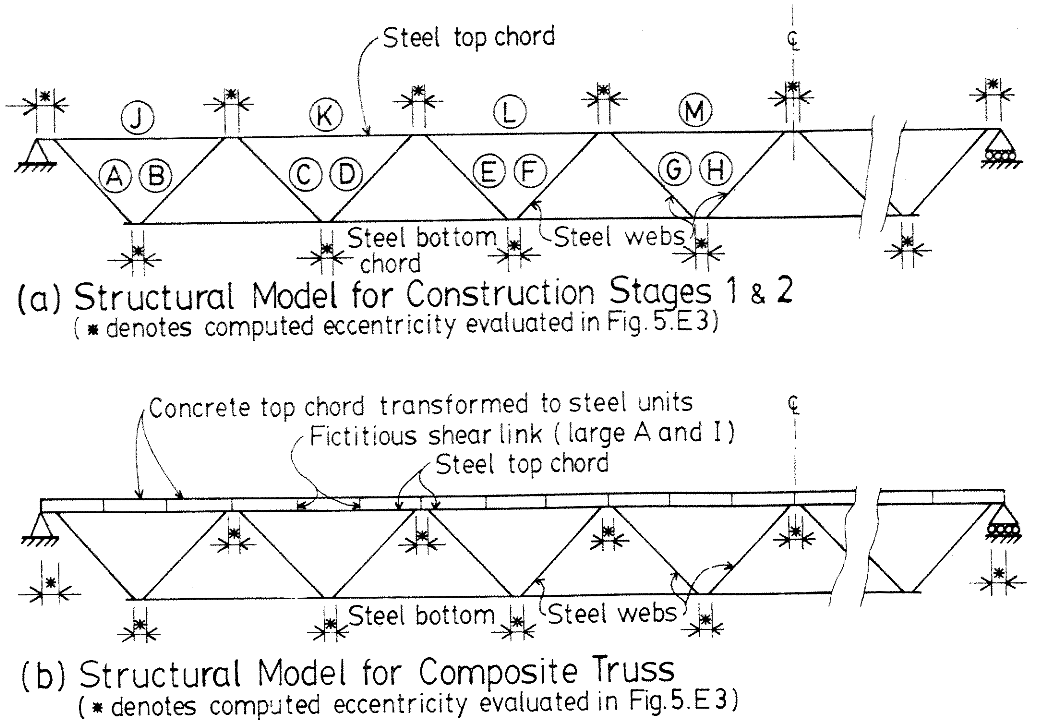


Figure 5.E4
Structural Modelling for Truss T1

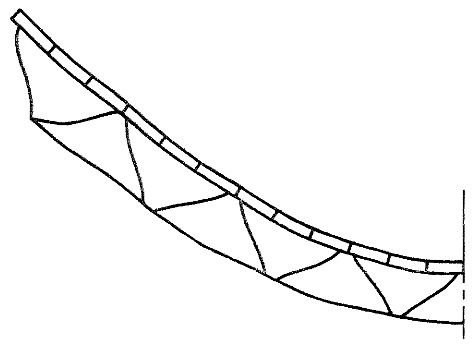


Figure 5.E5
Deflected Shape of Composite Truss
(Exaggerated to Show Member Curvature)

Member forces at critical top chord panels under load cases 1 to 4 are tabulated as follows:

Load Case	Load Description	Critical top chord member	Member Design Forces: Factored Axial (kN) C_f	Factored Moment (kN·m) at mid panel M_{fm}	Factored Moment (kN·m) (max) at support M_{fs}
1	deck placement	M J	78.8 17.8	0.494 1.06	0.29 1.15
2	deck placement	M	52.5	1.49	0.58
3	concrete placement	M J	365 82.4	2.29 4.89	1.34 5.34
4	concrete placement	M	287	2.96	1.45

Design checks for top chord panel M (HSS76.2×76.2×6.35)

For load Cases 1 and 2:

– panel width (conservatively assumed) $= \dfrac{11\,500}{8} = 1\,438$ mm

– lateral support (provided by bridging or by direct deck-to-top chord connection) = 11 500/3 = 3 833 mm (governs)

effective length $= Kl_y = (1.0)(3\,833) = 3\,833$ mm

$$\frac{Kl_y}{r_y} = \frac{3\,833}{28} = 137$$

For 350W steel, $\dfrac{C_r}{A} = 83.1$ MPa, (Handbook, Table 4-3).

$C_{r(kl)} = 83.1\,A = (83.1)\,(1\,670)\,10^{-3} = 139$ kN
$M_r = 13.5$ kN·m (Handbook, Page 4-61)
$C_{ro} = 526$ kN

effective length $Kl_x = (0.9)(1\,438) = 1\,294$ mm (K = 0.9, interior panel)

$$\frac{Kl_x}{r_x} = \frac{1\,294}{28} = 46.2 \;, \quad \frac{C_e}{A} = 923 \;, \quad C_e = 923\,A = 1\,541 \text{ kN}$$

$$\frac{C_f}{C_e} = \frac{78.8}{1\,541} = 0.05 \;, \quad U = 1.05$$ (Handbook, Table 4-9) (for Case 1 loading)

Utilization of combined axial and bending resistance (for Case 1 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{78.8}{139} + \frac{0.494\,(1.0)(1.05)}{13.5} = 0.61 < 1.0$$

$$\frac{C_f}{C_e} = \frac{52.5}{1\,541} = 0.034 \quad , \quad U = 1.04 \qquad \text{(for Case 2 loading)}$$

Utilization of combined axial and bending resistance (for Case 2 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{52.5}{139} + \frac{1.49\,(1.0)(1.04)}{13.5} = 0.49 < 1.0$$

For load Cases 3 and 4: (top chord is laterally supported by deck to chord connections.)

For 350W steel, $\frac{C_r}{A} = 260$ MPa (for $Kl_x/r_x = 46.2$)

$C_{r(kl)} = 260\,A = (260)(1\,670)\,10^{-3} = 434$ kN

$C_{ro} = 526$ kN , $M_r = 13.5$ kN·m

$$\frac{C_f}{C_e} = \frac{365}{1\,541} = 0.24 \quad , \quad U = 1.32 \qquad \text{(for Case 3 loading)}$$

Utilization of combined axial and bending resistance (for Case 3 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{365}{434} + \frac{2.29(1.0)(1.32)}{13.5} = 1.06 > 1.0 \text{ fail}$$

$$\frac{C_f}{C_e} = \frac{287}{1\,541} = 0.19 \quad , \quad U = 1.23 \qquad \text{(for Case 4 loading)}$$

Utilization of combined axial and bending resistance (for Case 4 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{287}{434} + \frac{2.96\,(1.0)(1.23)}{13.5} = 0.93 < 1.0$$

Top chord failed code check under load Case 3 (slab placing under the total u.d.l. loading). The designer may chose one of several options to continue the truss design:

- increase the size of top chord to HSS76.2×76.2×7.95
- introduce vertical post to support top chord panel M at location 'x' (Fig. 5.E1)
- rearrange web framing to reduce width of panel M

Let us assume the use of vertical post to modify the Warren framing under the panel M.

Tributary floor area for the computation of maximum axial force in the vertical post may be approximated as $(11.5)(3)/16 = 2.2\ m^2$.

Total factored axial force

$$= (2.2)\,\frac{W_c + W_p + W_{OD}}{(11.5)(3)}\,(1.25) + (9)(1.5)$$

$$= 12.6 + 13.5 = 26.1 \text{ kN}$$

Note that NBC minimum specified concentrated load of 9.0 kN, applied over any area of 0.75 m × 0.75 m has been used as the second term in the above equation. (NBCC Table 4.1.6.B)

Using Table 5.12, $Kl = (730 - 76.2 - 127) \simeq 530$ mm

Round Hollow Section of 26.7 OD is OK for use as a support post. See Fig. 5.E3 for final detail.

It can be shown by further analysis and design check that the utilization of combined axial and bending resistance of top chord member M is reduced to less than 1.0, when the vertical post is introduced.

Similar design checks may be made to top chord panel L (Fig. 5.E4a), without the use of a vertical post. The utilization of combined axial and bending resistance of the top chord member, under the critical load Case 3, can be shown as follows:

$$\frac{C_f}{C_e} = \frac{318}{1\,541} = 0.21 \quad , \quad U = 1.27$$

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{318}{434} + \frac{2.3\,(1.0)(1.27)}{13.5} = 0.95 < 1.0$$

Design checks for top chord panel J

For load Case 1:
- panel width (conservatively assumed) = 1 438 − 75 = 1 363 mm
- lateral support (same as for top chord panel M) = 3 833 mm

Therefore, $C_{r(kl)} = 139$ kN (same as for top chord panel M)
$M_r = 13.5$ kN $\quad C_{ro} = 526$

$$\frac{Kl_x}{r_x} = \frac{(1.0)(1\,363)}{28} = 48.7 \qquad K = 1.0 \text{ (end panel)}$$

$$\frac{C_e}{A} = 830 \quad , \quad C_e = 830(1\,670)10^{-3} = 1\,386 \text{ kN}$$

$$C_f = \frac{17.8}{1\,386} = 0.013 \quad , \quad U = 1.01$$

Utilization of combined axial and bending resistance (for Case 1 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{17.8}{139} + \frac{1.15\,(1.0)(1.01)}{13.5} = 0.21 < 1.0$$

For load Case 3:

$$\frac{C_r}{A} = 255 \quad , \quad C_{r(kl)} = 255(1\,670)\,10^{-3} = 426 \text{ kN}$$

$$\frac{C_f}{C_e} = \frac{82.4}{1\,386} = 0.06 \quad , \quad U = 1.06$$

Utilization of combined axial and bending resistance (for Case 3 loading)

$$\frac{C_f}{C_{r(kl)}} + \frac{M_{fm}\,\omega\,U}{M_r} = \frac{82.4}{426} + \frac{5.34\,(1.0)(1.06)}{13.5} = 0.61 < 1.0$$

Check maximum end shear at truss supports

$V_f = 150$ kN , as shown before
$V_r = \phi\,(0.66)\,F_y\,C_{rt}$
where C_{rt} = Shear constant (mm^2) See Handbook
(or effective shear area)
$V_r = 0.9\,(0.66)(350)(645)(10^{-3})$
$= 134$ kN ($< V_f$) (assuming top chord section only)
See shoe detail provided in Fig. 5.E3 (Therefore $V_r > V_f$).

Check maximum shear at location 'y' (See Fig. 5.E1)

$$V_f \simeq 150\left(\frac{3}{4}\right) = 113 \text{ kN}$$

$V_r = 134$ kN OK

The design checks provided up to this stage show that the top chord is satisfactory for construction load conditions (Cases 1 to 4) as well as for occupancy load condition (with respect to ultimate shear resistance).

With the selected top chord member of HSS76.2×76.2×6.35 the maximum diameter of shear studs permitted by Clause 17.3.5.5 of S16.1 can be calculated as 2.5t, or **stud diameter** = 15.9 mm

The number of studs (based on single stud per flute) per truss = $2V_h/q_r$ = (2)(564/51.6) = **22 studs** (q_r value obtained from Table 2.1) (See Fig. 5.E1).

(e) Detailed Member Designs (Web members)

S16.1 Clause 17.3.1.2 states that web steel members of composite joist (or truss) shall be proportioned to carry the total vertical shear, V_f. In addition, section 5.4d of this publication has identified the types of local moments to be included in the design of web members in compression. However, during the design of web members in tension, such local moments are neglected, as noted in the text.

In the following example calculations, two typical web members are design checked. Diagonal A consists of a pair of single angles in tension, and diagonal B consists of a pair of single angles in compression. In either case, the angles are end-connected on one leg by welding, but are not battened nor connected by spacers between supports. The design of fillet weld web-to-chord connections is also included in order to provide a complete picture of connected web members. All design procedures, assumptions and details are illustrated and are assumed to be appropriate only to this type of steel web members.

– **Diagonal 'A'** (see Fig. 5.E6)

$T_f = 199$ kN (as shown previously)
$M_{fx} = 0.66$ kN·m (from stiffness analysis, composite model)
(under factored occupancy loading)
M_{fy} due to out of plane eccentricity (ignored as explained)

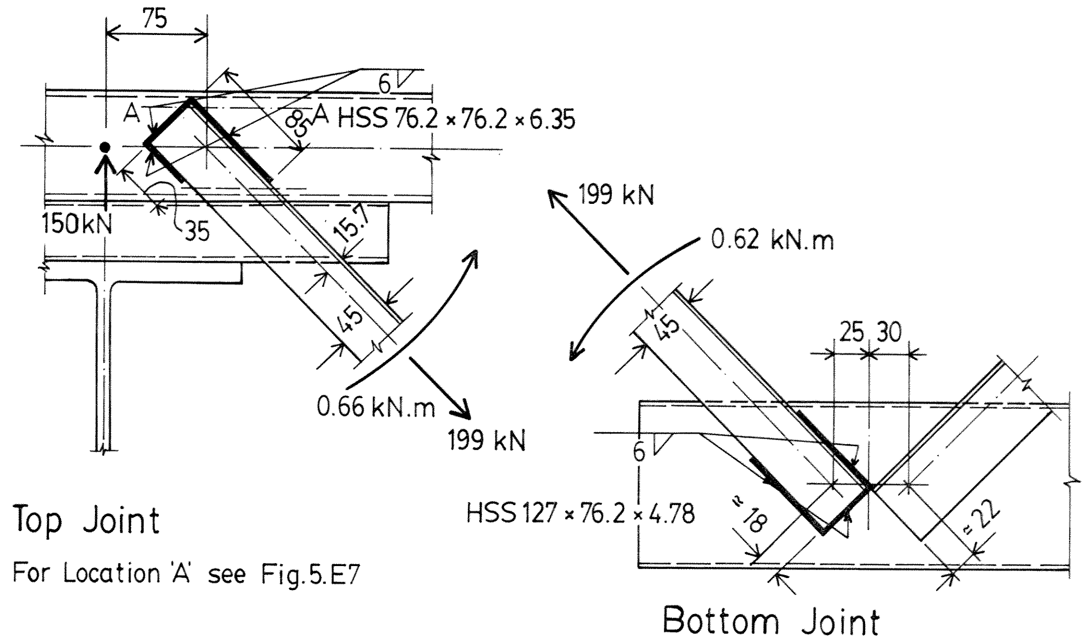


Figure 5.E6
Detail at Diagonal 'A'

Section previously selected consists of 2 angles L45×30×6
$A_s = 828$ mm^2 $S_x = 5.58 \times 10^3$ mm^3
$T_r = 0.9\,A_s\,F_y = 0.9(828)(0.300) = 224$ kN
$M_{rx} = 0.9\,S_x\,F_y = 0.9(5.58)(0.300) = 1.51$ kN·m

Utilization of factored member resistance

$$\frac{T_f}{T_r} = \frac{199}{224} = 0.89 < 1.0$$

Since stiffness analysis of the composite model has been performed based on rigid-joint basis whilst the axial tension of 199 kN was obtained using a pin-jointed model, the utilization calculation may be computed based on tensile values only provided that the top chord alone is able to resist the total bending moment caused by joint eccentricity ($M_f = 150 \times 0.075 = 11.25$ kN·m, M_r for HSS76.2×76.2×6.35 = 13.5 kN·m, see Fig. 5.E6).

– Welding at ends of diagonal 'A' (see Fig. 5.E6)
The following calculation is provided to illustrate the design-check-methodology of weldment using a simple and conservative approach.

Upper joint of diagonal 'A'

Figure 5.E6 provides assumed detail of fillet weld for the joint along with the applied member forces. For the assessment of weld resistance, portion of weld material above the assumed line AA is neglected for effects of corner radius of the HSS top chord; see Fig.5.E7.

C.G. of weld group about 'x-x' may be computed as,

$$\frac{(40)^2/2+(70)(45)}{40+70+35} = 27.2 \text{ mm}$$

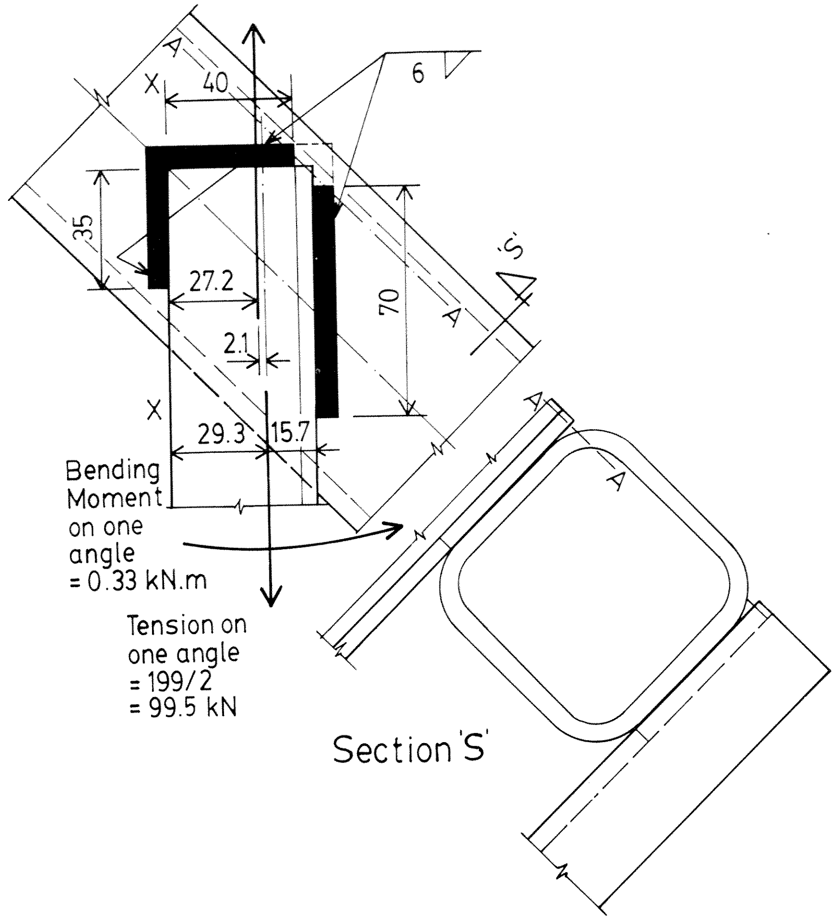


Figure 5.E7
Design of Fillet Welds for
Diagonal 'A' (Upper Joint)

To estimate maximum factored moment on weld group, let us consider the following two assumptions:

Case A Bending moment acting on the weld group (without the consideration of local moment) caused by the eccentricity of the weldment,

$$M_{f1} \text{ (on one angle)} = \frac{199}{2}(2.1)\ 10^{-3} = 0.209 \text{ kN·m}$$

Case B Bending moment acting on the weld group caused by both the eccentricity of the weldment and the effect of local moment (from stiffness analysis of composite model assuming rigid-end joints),

$$M_{f2} \text{ (on one angle)} = \frac{0.66}{2} - \frac{199}{2}(2.1)\ 10^{-3} = 0.12 \text{ kN·m}$$

Let us conservatively assume that the maximum factored moment acting on the weld group is equal to the larger of the values computed above; $M_f = 0.209$ kN·m.

$$\text{Factored tensile force on one angle} = \frac{199}{2} = 99.5 \text{ kN}$$

Using 6 mm fillet weld, the factored shear resistance per millimetre of weld length can be found from the Handbook of Steel Construction, Table 3-24, as 0.918 kN.

Hence total factored shear resistance of weld group is computed as,
$(40+70+35)(0.918) = 133$ kN

Considering only 35 mm of fillet weld on each side of steel angle (angle leg width = 45 mm), total factored moment resistance of the assumed weld group may be approximated as,

$$0.918(35)(45)\ 10^{-3} = 1.45 \text{ kN·m}$$

Utilization of weld resistance under the combined shear and moment may be computed as,

$$\frac{99.5}{133} + \frac{0.209}{1.45} = 0.89 < 1.0 \quad \text{OK}$$

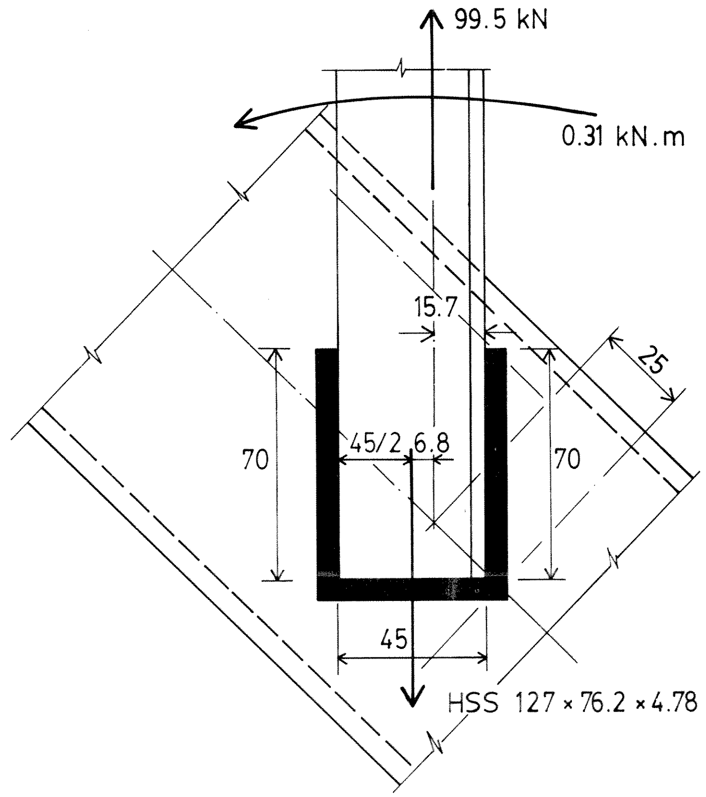


Figure 5.E8
Design of Fillet Welds for
Diagonal 'A' (Lower Joint)

Lower joint of diagonal 'A'

Figure 5.E8 provides assumed detail of fillet weld along with applied member forces for one angle diagonal. To estimate maximum factored moment on the assumed weld group, let us consider the following two assumptions:

Case A Bending moment acting on the weld group caused by the eccentricity of weldment (without the consideration of local moment)

M_{f1} (on one angle) $= 99.5\ (6.8)\ 10^{-3} = 0.677$ kN·m

Case B Bending moment acting on the weld group caused by both the eccentricity of weldment and the effect of local moment (from stiffness analysis of composite model assuming rigid-end joints)

M_{f2} (on one angle) $= 0.677 + 0.31 = 0.987$ kN·m

Let us conservatively assume that the maximum factored moment acting on the weld group is equal to the larger of the values computed above; $M_f = 0.987$ kN·m

Factored tensile force on one angle = 99.5 kN

Using 6 mm fillet weld, total factored shear resistance of weld group can be computed as,

$$\left[(70)(2) + 45\right](0.918) = 170 \text{ kN}$$

Estimated factored moment resistance of two lines of 6 mm weld of length 70 mm each, spaced 45 mm apart may be computed as

$$(70)(0.918)(45)\ 10^{-3} = 2.89 \text{ kN·m}$$

Utilization of weld resistance under the combined shear and moment may be computed as,

$$\frac{99.5}{170} + \frac{0.987}{2.89} = 0.93 < 1.0 \qquad \text{OK}$$

Note that for the design of weld groups, the effect of out-of-plane bending (due to end connection to one leg of angle) may be neglected for long members. See section 5.9 of Reference (5.16)

– **Diagonal 'B'** (see Fig. 5.E9)

Section previously selected consists of two angles L55×55×10 (batten or spacer not used). Factored forces, acting at ends of one angle of diagonal 'B', are shown in Fig. 5.E10; and they are as summarized below:

$C_f = 99.5$ kN
$M_{fs(top)} = 1.84$ kN·m $\qquad M_{fs(bottom)} = 1.13$ kN·m
$M_{fz(top)} = 0.257$ kN·m $\qquad M_{fz(bottom)} = -0.449$ kN·m
Member curvature about s-s axis = single
Member curvature about z-z axis = double

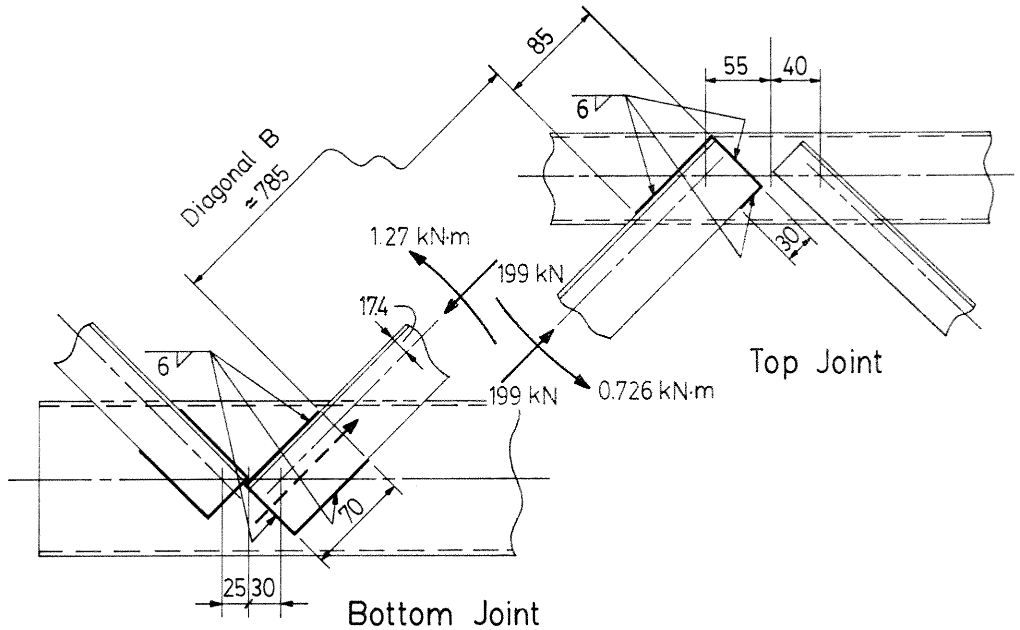


Figure 5.E9
Detail at Diagonal 'B'

Member properties: L55×55×10 (not a class 4 section, see 5.7)

Kl = clear length = 785 (say)
A_s = 1 000 mm² $\qquad r_x = r_y = 16.4$ mm $\qquad r_z = 10.7$ mm
y = 17.4 mm
$r_s^2 = 2\,r_x^2 - r_z^2 = 423$ (for equal leg angles)
$I_s = r_2^2\,A_s = 0.423\times10^6$ mm⁴
$S_s = I_s/38.9 = 10.9\times10^3$ mm³ (see Fig. 5.E10)
$I_z = r_z^2\,A_s = 10.7^2\,(A_s) = 0.114\times10^6$ mm⁴
$S_z = I_z/14.3 = 7.97\times10^3$ mm³ (with respect to point 'A')

Factored resistances calculations:

$$\frac{Kl}{r_z} = \frac{785}{10.7} = 73 \quad , \quad C_{ez} = 370A = 370 \text{ kN}$$
(Table 4-8 of Handbook)

$C_{ro} = 0.9\ (0.3)(1\ 000) = 270$ kN
$C_{rz} = 182$ kN (Use Table 4-3 of Handbook)
$M_{rs} = 0.9(0.3)(S_s) = 2.94$ kN·m with respect to point 'A'
$M_{rz} = 0.9(0.3)(S_z) = 2.15$ kN·m

$$\frac{Kl}{r_s} = \frac{785}{20.6} = 38 \quad , \quad C_{es} = 1\ 364 \text{ kN}$$

$$\frac{C_f}{C_{ez}} = \frac{99.5}{370} = 0.27 \quad , \quad U_z = (1-0.27)^{-1} = 1.37$$

$$\frac{C_f}{C_{es}} = \frac{99.5}{1\ 364} = 0.07 \quad , \quad U_s = (1-0.07)^{-1} = 1.08$$

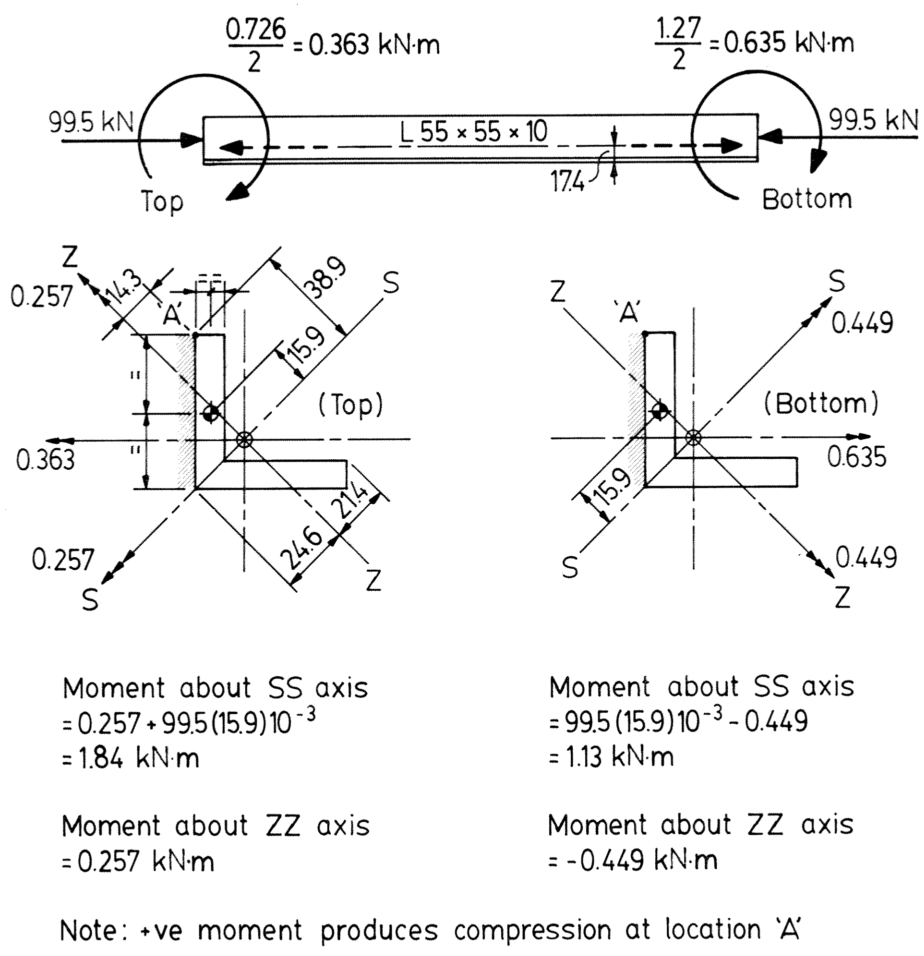


Figure 5.E10
Factored Forces on One Angle of Diagonal 'B'

$$\omega_s = 0.6 + 0.4\frac{1.13}{1.84} = 0.85 \qquad \text{(Single curvature)}$$

$$\omega_z = 0.6 - 0.4\frac{0.257}{0.449} = 0.37 \text{ (i.e 0.4)} \qquad \text{(Double curvature)}$$

Check Strength*:

$$\frac{99.5}{270} + \left(\frac{1.84}{2.94} + \frac{0.257}{2.15}\right)/2 = 0.74 < 1.0 \qquad \text{OK (top joint)}$$

$$\frac{99.5}{270} + \left(\frac{1.13}{2.94} - \frac{0.449}{2.15}\right)/2 = 0.46 < 1.0 \qquad \text{OK (bottom joint)}$$

Check Stability*:

$$\frac{C_f}{C_{rz}} + \left(\frac{\omega_s\, M_{fs}\, U_s}{M_{rs}} + \frac{\omega_z\, M_{fz}\, U_z}{M_{rz}}\right)/2$$

$$= \frac{99.5}{182} + \left[\frac{0.85(1.84)(1.08)}{2.94} + \frac{0.4(0.449)(1.37)}{2.15}\right]/2$$

$$= 0.89 < 1.0 \qquad \text{OK}$$

*The sum of utilization of moment resistances as shown above has been arbitrarily halved due to the following reasons:

- Although the axial compression is calculated based on pin-jointed members, the local bending at ends of diagonal member are obtained assuming that all joints are rigid. This is too conversative.
- Test results(5.17) on single angle struts of similar slenderness have shown that it is too conservative to assume full end moment (caused by connecting one leg of the angle strut).

Upper and Lower joints of diagonal 'B'

Same procedure shall be used for the sizing of weldments for diagonal 'B' as that given for diagonal 'A'. For finished detail, see Fig. 5.E9.

(f) Truss Deflection Estimate

Top chord	HSS76.2×76.2×6.35	$I_x = 1.31\times10^6$ mm^4
Bottom chord	HSS127×76.2×4.78	$I_x = 3.78\times10^6$ mm^4

Web member	A	2L45×30×6
	B	2L55×55×10
	C	2L35×35×5
	D	2L55×55×8
	E	2L25×25×5
	F	2L55×55×5
	G	2L25×25×5
	H	2L35×35×5

– Camber requirement

Using approximate calculation method and Table 5.14, steel truss moment of inertia I_s† may be calculated as,

$$I_s = 743\,(h^2) = 734\,[D-(76.2+127)/2]^2$$
$$= 289.8 \times 10^6 \text{ mm}^4$$

Total effective truss moment of inertia,

$$I = (289.8+1.31+3.78)\,10^6 = 295 \times 10^6 \text{ mm}^4$$

†Note, I_s values in Table 5.14 include a 15% reduction to allow for web deflection due to strain.

Specified dead load at concrete placement is estimated as 8.02 kN/m (see item (d))

$$\Delta_c = \frac{5}{384}\,\frac{8.02(11.5)^4}{200\,(295)}\,10^3 = 31 \text{ mm}$$

Using stiffness analysis computer run, based on rigid jointed steel truss members, Δ_c can be shown as 30.7 mm.

Therefore truss T1 is to be **cambered** at mid span for about **30 mm.**

– Deflection due to all specified superimposed loads including long term effects.

Using approximate calculation method

Step 1. Compute composite truss moment of inertia

I_g, from item (a), is given as 973×10^6 mm^4

Assume I loss for web deflection due to strain,

$I_{wr} \simeq$ 15% of I of truss chords
$= (0.15/0.85)\ 289.8\ (10^6)$
$= 51 \times 10^6$ mm^4

Therefore moment of inertia of composite truss,

$I_t \simeq (973-51) \times 10^6 = 922 \times 10^6$ mm^4

Step 2. Composite deflection due to all superimposed loads

$$= \frac{5}{384}\ \frac{(W_L+W_p+W_{OD})\ L^3}{E\ I_t}\ (1.0 + 15\% \text{ for slip} + 15\% \text{ for creep})$$

$$= \frac{5}{384}\ \frac{(68.7+41.4+24.2)\ 11.5^3}{200\ (922)}\ (10^3)\ 1.30$$

$$= 18.7 \text{ mm} \quad < \quad \frac{L}{300} = 38 \qquad \text{OK}$$

Computer stiffness analysis of rigid joint truss model gives 18.5 mm

Composite Truss with WT Chords and Angle Webs

(a) Trial Selection

Bottom Chord (WT system)

Compute effective slab width (assuming width of top chord = 150 mm)

b_l is governed by $16t_o + b = 16(141)+150 = 2\ 406$ mm in this truss design, (see previous design).

Using Table 5.4 – Composite Truss Bottom Chord
– trial selection table (WT Bottom Chord)

for D = 730 mm, t_d = 76 mm, t_c = 65 mm.
b_l = 2 410 mm and using **WT125×16.5**

By interpolation, M_{rc} = 468 kN·m > (M_f = 431) OK
V_h = 562 kN V_r = 140 kN > V_f = $(W_f/2)-W_f/16$
= 131 kN OK

y = 27.3 mm

By interpolation, $I_g = 1\ 224 \times 10^6$ mm^4

Top Chord (WT system)

As computed before:
for deck placement, w_{f1} = 3.11 kN/m
for concrete placement, w_{f2} = 14.0 kN/m

Using Table 5.7 and **WT125×19.5** as top chord,
Truss span = 11 500 mm lateral support spacing = span/2
panel width = 1 438 mm

Centroidal distance, h = 730 – (27.3 + 26.7)
= 676 mm

p/h = 2.13

w_{r1} may be interpolated as 5.3 kN/m (> 3.11) OK
w_{r2} may be interpolated as 15.4 kN/m (> 14.0) OK
V_r = 154 kN (> W_f/2 OK)

Web members (Double angle struts, interconnected at mid length)
(Double angle tension members)

Factored axial forces (under occupancy loading) in web members are presented in Fig. 5.E11 and are tabulated (with the selected web members) in the table below:

Web Member Reference	Factored Axial Force (kN) (–ve = tension)	Factored Axial Resistance (kN) (C_r or T_r)	Section Selected using Tables 5.8 and 5.11 (Theoretical length 990 mm)
A	– 191	T_r = 207	2L35×35×6
B	+ 191	C_r = 214	2L55×55×5
C	– 137	T_r = 175	2L35×35×5
D	+ 137	C_r = 169	2L45×45×5
E	– 82.2	T_r = 121	2L25×25×5
F	+ 82.2	C_r = 102	2L35×35×5
G	– 27.4*	T_r = 60.8	
		C_r = 9	L25×25×5
H	+ 27.4	C_r = 44	2L25×25×5

*Note: Possibility of force reversal during erection (6 kN)

(b) Truss Framing Layout and Truss Modelling

A scaled layout of truss members is provided as shown in Fig. 5.E12, to ensure simplicity in connection detailing and to evaluate amount of eccentricity at each web to chord joint.

(c) Detailed Member Design Checks (all chord and web members)

Bottom Chord: Use method as shown for HSS truss T1.

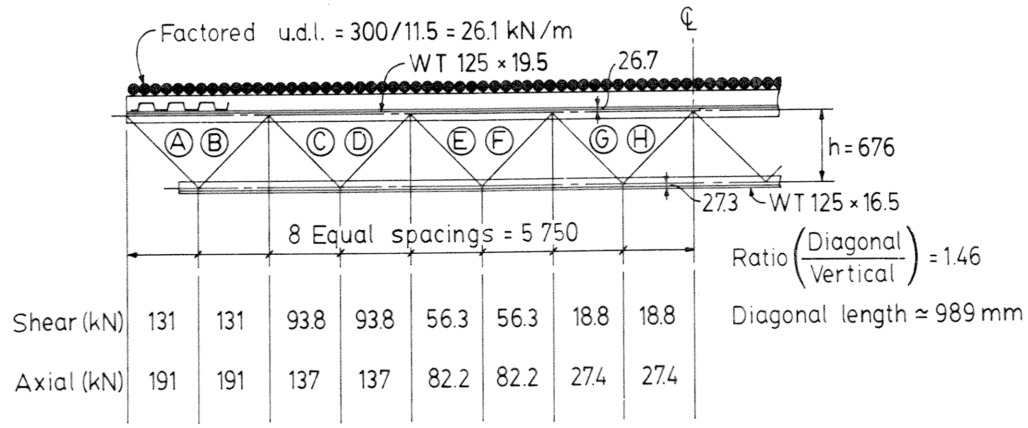


Figure 5.E11
Computation of Factored Web Forces for Preliminary Design (WT Chords)

Top Chord: Five load cases as shown for HSS truss T1 shall be investigated. Note that WT chord selected is a "Class 4" section. Values of sectional resistance are computed based on properties of a reduced 'tee' section whose d/w ratio satisfies "Class 3" limit.

Web members (in compression): Double angle struts interconnected at mid length are assumed. Torsional-flexural instability of each web member in compression is to be investigated using an equivalent radius of gyration method, see Reference (5.18).

Web members (in tension): Use method as shown for HSS truss T1.

(d) Compute Total Number of Shear Studs

With the selected top chord member of WT125×19.5, **stud diameter** of 19 mm is found to be satisfactory.

2.5 times flange thickness of WT $= 2.5(11.2) = 28 > 19$ OK

q_r for 19 mm studs in 20 MPa, 2 300 kg/m^3 concrete may be obtained from Table 2.1 as 74.3 kN.
V_h is shown as 562 kN; see item (a) of composite truss with WT chords.

Total number of 19 mm diameter studs (based on single stud per flute)
$= 2\,V_h/q_r$
$= (2)(562)/74.3$
$=$ **16 studs** per truss.

(e) Truss Deflection Estimate

Top Chord WT 125×19.5 $I_x = 3.26\times10^6$ mm^4
Bottom Chord WT 125×16.5 $I_x = 2.85\times10^6$ mm^4

– Camber requirement (use Table 5.15)
$I_s = 958\,(h^2) = 958(676)^2 = 438 \times 10^6$ mm^4

Total effective truss moment of inertia
$I = (438 + 3.26 + 2.85)\,10^6 = 444 \times 10^6$ mm^4

$$\Delta_c = \frac{5}{384}\,\frac{8.02\,(11.5)^4}{200\,(444)}\,10^3 = 21 \text{ mm}$$

20 mm camber required

– Deflection due to all specified superimposed loads including long term effects.

$I_g = 1\,224 \times 10^6$ mm^4 $\quad L = 11.5$ m
$I_{wr} = (0.15/0.85)\,438 \times 10^6$
$\quad = 77 \times 10^6$ mm^4
$I_t = (1\,224 - 77) \times 10^6$
$\quad = 1\,147 \times 10^6$ mm^4

$$\Delta = \frac{5}{384}\,\frac{(68.7+41.4+24.2)L^3}{200\,(1\,147)}\,10^3 \times 1.3$$

$$= 15.1 \text{ mm} < \frac{L}{300} = 38 \qquad \text{OK}$$

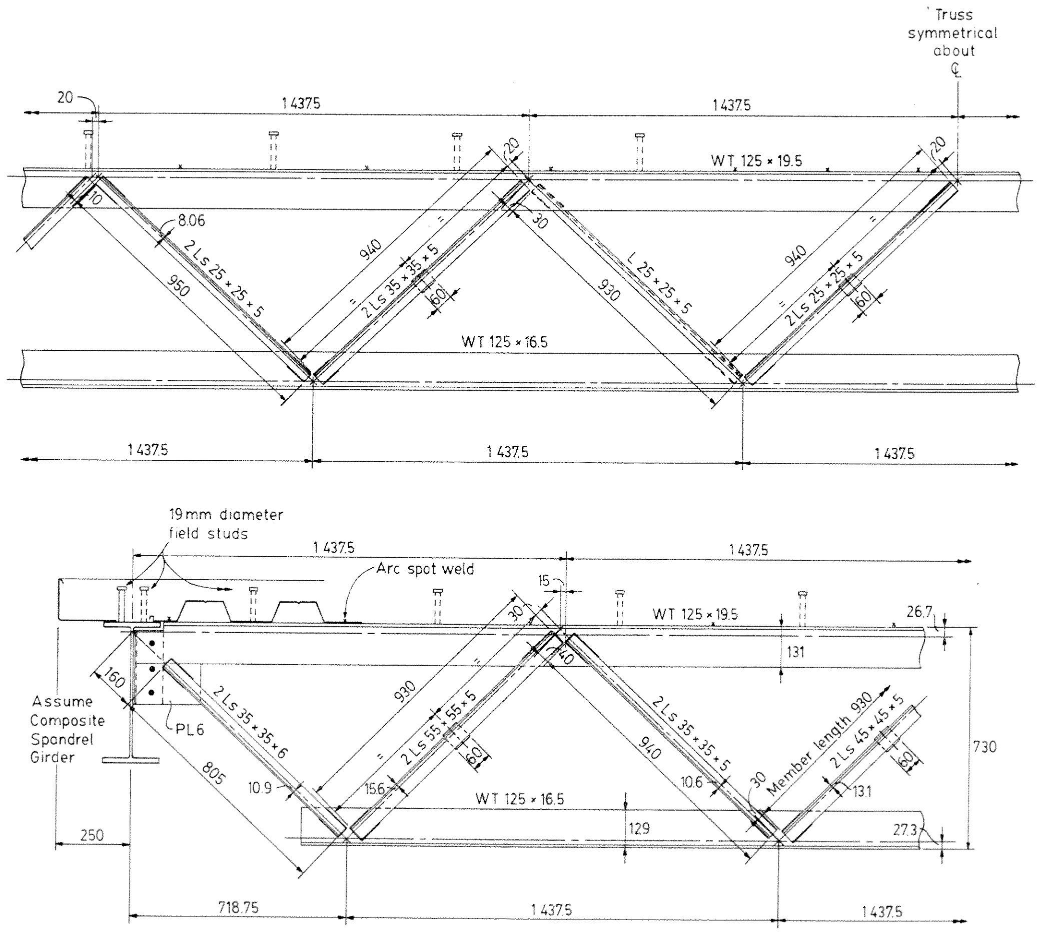


Figure 5.E12
Truss Framing Layout (Truss T1)
(WT Chords)

Summary of Truss Mass Takeoff

Truss Chord Type	Mass per Truss (kg)	
HSS Chords	427	(or 37.1 kg/m)
WT Chords	487	(or 42.4 kg/m)

From reference (15): cost factor for trusses with HSS chords is 1.8, and cost factor for trusses with WT chords is 1.5.

Therefore factored truss mass ratio for costing purposes (HSS versus WT truss) $= 1.8\ (427)/(1.5(487)) \simeq 1.0$. Say about the same cost.

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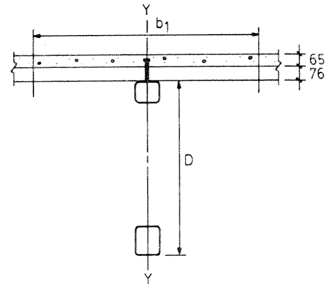
Table 5.3
COMPOSITE TRUSS BOTTOM CHORD
Trial Selection Table

65 mm Concrete Cover on 76 mm Deck
20 MPa Normal Density+ Concrete



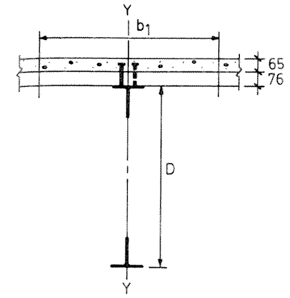
HSS BOTTOM CHORDS (G40.21-M 350W)

Bottom Chord Section: Designation and Design Data	Mass kg/m	Composite Truss: D(mm) / b_1(mm)	Factored Moment Resistances, M_{rc} (kN•m): 650	750	850	950	Gross Moment of Inertia, I_g (10^6 mm^4): 650	750	850	950
HSS152.4×101.6×11.13	38.0	2360	1040	1190	1340	1490	1740	2290	2910	3610
V_h = 1520 kN		2330	1040	1190	1340	1490	1740	2280	2900	3590
r_y = 38.4 mm		2290	1040	1190	1340	1490	1730	2270	2890	3580
HSS152.4×101.6×9.53	33.3	2360	921	1050	1190	1320	1570	2060	2620	3250
V_h = 1340 kN		2190	918	1050	1190	1320	1550	2030	2580	3200
r_y = 39.1 mm		2020	914	1050	1180	1320	1520	1990	2540	3140
HSS152.4×101.6×7.95	28.4	2360	788	902	1020	1130	1380	1820	2310	2860
V_h = 1140 kN		2040	784	898	1010	1130	1350	1770	2250	2780
r_y = 39.9 mm		1720	778	892	1010	1120	1300	1700	2160	2680
HSS152.4×101.6×6.35	23.2	2360	648	741	835	928	1170	1540	1960	2420
V_h = 932 kN		1880	644	737	830	923	1130	1480	1880	2330
r_y = 40.6 mm		1410	636	729	822	916	1060	1390	1770	2190
HSS101.6×101.6×9.53	25.7	2360	740	843	946	1050	1370	1790	2250	2780
V_h = 1030 kN		1960	736	839	942	1040	1330	1730	2180	2680
r_y = 36.8 mm		1550	729	832	935	1040	1260	1640	2070	2550
HSS101.6×101.6×7.95	22.1	2360	641	730	819	907	1210	1570	1990	2450
V_h = 888 kN		1850	636	725	814	903	1160	1510	1910	2350
r_y = 37.6 mm		1340	628	717	806	895	1090	1410	1780	2200
HSS127.0×76.2×7.95	22.1	2330	629	718	807	896	1160	1520	1930	2380
V_h = 888 kN		1840	625	714	803	891	1120	1460	1850	2290
r_y = 29.4 mm		1340	617	706	795	884	1050	1370	1730	2140
HSS127.0×76.2×6.35	18.2	2330	521	594	667	740	985	1290	1630	2010
V_h = 731 kN		1720	517	590	663	736	941	1230	1560	1930
r_y = 30.1 mm		1100	508	581	654	727	861	1130	1430	1760
HSS127.0×76.2×4.78	14.1	2330	404	460	516	573	784	1020	1300	1600
V_h = 564 kN		1670	401	457	514	570	752	983	1240	1540
r_y = 30.8 mm		1000	395	451	508	564	689	901	1140	1410
HSS101.6×76.2×9.53	21.9	2330	634	722	810	898	1200	1560	1960	2420
V_h = 879 kN		1830	630	718	806	894	1150	1490	1890	2320
r_y = 27.7 mm		1330	622	710	798	886	1070	1400	1770	2180
HSS101.6×76.2×7.95	18.9	2330	550	626	701	777	1060	1370	1730	2130
V_h = 759 kN		1740	546	621	697	773	1010	1310	1660	2040
r_y = 28.5 mm		1140	537	613	689	765	926	1210	1520	1880
HSS101.6×76.2×6.35	15.6	2330	456	519	581	644	892	1160	1460	1800
V_h = 627 kN		1670	453	515	578	641	854	1110	1400	1730
r_y = 29.3 mm		1000	445	508	570	633	776	1010	1270	1570

No. of studs per truss = $2(V_h/q_r)$

+ Concrete density = 2300 kg/m^3

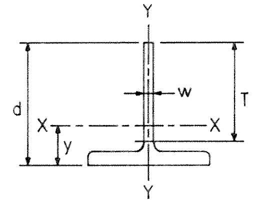
Table 5.4
COMPOSITE TRUSS BOTTOM CHORD
Trial Selection Table



65 mm Concrete Cover on 76 mm Deck
20 MPa Normal Density+ Concrete

WT BOTTOM CHORDS (G40.21-M 300W)

Bottom Chord Section and Design Data



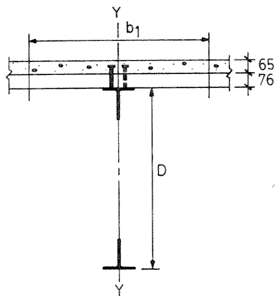
Designation			Composite Truss: D(mm) / b_1(mm)	Factored Moment Resistances M_{rc} (kN•m): 600	750	900	1050	Gross Moment of Inertia, I_g (10^6 mm^4): 600	750	900	1050
WT180×39.5	r_y=48.9 mm	d=177 mm	2460	923	1130	1330	1540	1770	2640	3690	4920
V_h=1360 kN	y =35.0 mm	T=141 mm	2250	920	1120	1330	1530	1730	2590	3620	4810
V_r= 296 kN	w= 9.4 mm		2050	916	1120	1320	1530	1690	2530	3530	4700
WT180×36	r_y=48.5 mm	d=175 mm	2460	839	1020	1210	1390	1640	2440	3410	4540
V_h=1230 kN	y =34.2 mm	T=141 mm	2160	835	1020	1200	1390	1590	2370	3310	4410
V_r= 268 kN	w= 8.6 mm		1860	829	1010	1200	1380	1530	2290	3190	4250
WT180×32	r_y=48.1 mm	d=174 mm	2460	754	919	1080	1250	1500	2240	3130	4160
V_h=1100 kN	y =33.2 mm	T=141 mm	2060	750	915	1080	1240	1450	2160	3010	4010
V_r= 239 kN	w= 7.7 mm		1660	743	908	1070	1240	1370	2050	2860	3810
WT180×28.5	r_y=39.2 mm	d=179 mm	2430	665	811	958	1100	1340	2000	2800	3730
V_h=975 kN	y =39.2 mm	T=149 mm	1950	660	807	953	1100	1280	1910	2680	3570
V_r=252 kN	w= 7.9 mm		1470	653	799	945	1090	1200	1790	2500	3340
WT180×25.5	r_y=38.8 mm	d=178 mm	2430	595	725	856	986	1220	1820	2550	3390
V_h=869 kN	y =38.8 mm	T=149 mm	1870	590	721	851	981	1160	1740	2430	3230
V_r=228 kN	w= 7.2 mm		1310	582	712	843	973	1070	1600	2240	2980
WT180×22.5	r_y=37.8 mm	d=176 mm	2430	531	647	764	880	1100	1650	2300	3070
V_h=775 kN	y =40.2 mm	T=149 mm	1800	527	643	759	876	1050	1570	2190	2920
V_r=216 kN	w= 6.9 mm		1170	518	634	750	867	948	1420	1990	2650
WT155×43	r_y=63.7 mm	d=155 mm	2460	1020	1240	1470	1690	1940	2880	4010	5330
V_h=1490 kN	y =26.1 mm	T=122 mm	2350	1020	1240	1470	1690	1920	2850	3970	5270
V_r= 251 kN	w= 9.1 mm		2250	1020	1240	1460	1690	1900	2820	3920	5210
WT155×39.5	r_y=63.1 mm	d=153 mm	2460	935	1140	1340	1550	1810	2690	3750	4980
V_h=1360 kN	y =26.1 mm	T=121 mm	2250	932	1140	1340	1540	1770	2640	3670	4880
V_r= 240 kN	w= 8.8 mm		2050	928	1130	1340	1540	1730	2580	3590	4760
WT155×37	r_y=49.7 mm	d=155 mm	2460	878	1070	1260	1450	1710	2550	3560	4730
V_h=1280 kN	y =29.7 mm	T=122 mm	2190	874	1070	1260	1450	1670	2480	3460	4600
V_r= 260 kN	w= 9.4 mm		1930	869	1060	1250	1440	1610	2410	3350	4460
WT155×33.5	r_y=49.2 mm	d=153 mm	2460	793	965	1140	1310	1580	2350	3270	4340
V_h=1150 kN	y =28.7 mm	T=121 mm	2100	788	961	1130	1310	1520	2270	3160	4200
V_r= 232 kN	w= 8.5 mm		1730	782	954	1130	1300	1450	2160	3010	4000
WT155×30	r_y=49.0 mm	d=152 mm	2460	714	868	1020	1180	1450	2150	2990	3980
V_h=1030 kN	y =27.5 mm	T=122 mm	2000	709	863	1020	1170	1390	2060	2870	3820
V_r= 203 kN	w= 7.5 mm		1550	701	856	1010	1160	1300	1940	2700	3590

No. of studs per truss = 2 (V_h/q_r)

+ Concrete density = 2 300 kg/m^3

Table 5.4 (continued)
COMPOSITE TRUSS BOTTOM CHORD
Trial Selection Table

65 mm Concrete Cover on 76 mm Deck
20 MPa Normal Density+ Concrete



WT BOTTOM CHORDS (G40.21-M 300W)

Bottom Chord Section and Design Data			D(mm) / b_1(mm)	Composite Truss: Factored Moment Resistances M_{rc} (kN•m) 600	750	900	1050	Gross Moment of Inertia, I_g (10^6 mm^4) 600	750	900	1050
WT155×26	r_y=39.2 mm	d=158 mm	2430	620	755	890	1020	1270	1900	2650	3530
V_h=899 kN	y =32.9 mm	T=128 mm	1890	616	750	885	1020	1220	1810	2530	3360
V_r =214 kN	w= 7.6 mm		1360	607	742	877	1010	1120	1680	2340	3120
WT155×22.5	r_y=38.8 mm	d=156 mm	2430	532	647	762	877	1120	1660	2320	3090
V_h=767 kN	y =32.1 mm	T=128 mm	1790	528	643	758	873	1060	1580	2210	2930
V_r =183 kN	w= 6.6 mm		1160	519	634	749	864	962	1430	2000	2660
WT155×19.5	r_y=38.3 mm	d=155 mm	2410	464	564	664	764	992	1480	2060	2740
V_h=677 kN	y =31.4 mm	T=128 mm	1710	461	561	661	761	940	1400	1950	2600
V_r=160 kN	w= 5.8 mm		1010	452	552	652	752	838	1250	1740	2320
WT125×29	r_y=50.4 mm	d=126 mm	2430	699	849	999	1150	1440	2130	2960	3930
V_h=1000 kN	y =22.2 mm	T= 95 mm	1970	694	844	994	1140	1380	2040	2840	3770
V_r= 180 kN	w= 8.0 mm		1510	686	836	986	1140	1290	1920	2660	3540
WT125×24.5	r_y=49.2 mm	d=124 mm	2430	591	717	844	970	1250	1850	2560	3400
V_h=842 kN	y =22.1 mm	T= 96 mm	1850	587	713	839	965	1190	1760	2440	3240
V_r =164 kN	w= 7.4 mm		1270	578	704	831	957	1090	1610	2240	2970
WT125×22.5	r_y=35.1 mm	d=133 mm	2410	538	654	770	886	1140	1690	2350	3130
V_h=772 kN	y =27.8 mm	T=110 mm	1790	534	650	766	882	1080	1610	2240	2970
V_r =180 kN	w= 7.6 mm		1160	525	641	757	873	979	1460	2030	2700
WT125×19.5	r_y=34.7 mm	d=131 mm	2410	465	565	665	764	1000	1490	2070	2750
V_h=664 kN	y =26.7 mm	T=110 mm	1700	462	561	661	760	949	1410	1960	2610
V_r =154 kN	w= 6.6 mm		1000	453	552	652	751	845	1260	1750	2320
WT125×16.5	r_y=33.7 mm	d=129 mm	2410	395	479	563	648	864	1280	1790	2370
V_h=562 kN	y =27.3 mm	T=110 mm	1700	392	476	561	645	824	1220	1700	2260
V_r =140 kN	w= 6.1 mm		1000	386	470	554	639	744	1110	1540	2050
WT100×18	r_y=40.8 mm	d=100 mm	2410	439	532	625	717	967	1430	1980	2630
V_h=618 kN	y =17.5 mm	T= 77 mm	1700	436	529	622	714	919	1360	1890	2500
V_r =110 kN	w= 6.2 mm		1000	428	521	614	707	823	1220	1690	2240
WT100×15.5	r_y=32.0 mm	d=105 mm	2390	383	464	545	626	849	1260	1740	2310
V_h=540 kN	y =21.1 mm	T= 85 mm	1690	380	461	542	623	811	1200	1670	2210
V_r =120 kN	w= 6.4 mm		1000	374	455	536	617	735	1090	1510	2010
WT100×13.5	r_y=31.2 mm	d=104 mm	2390	324	392	461	529	730	1080	1500	1990
V_h=456 kN	y =21.1 mm	T= 86 mm	1690	322	391	459	527	701	1040	1440	1910
V_r =107 kN	w= 5.8 mm		1000	318	386	455	523	644	954	1330	1760

No. of studs per truss = $2(V_h/q_r)$

+ Concrete density = 2 300 kg/m^3

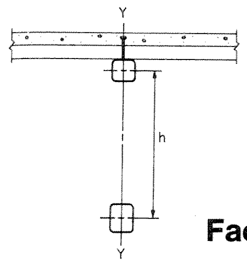
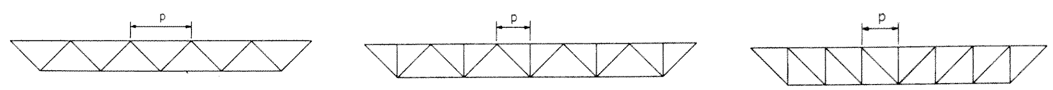


Table 5.5
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table

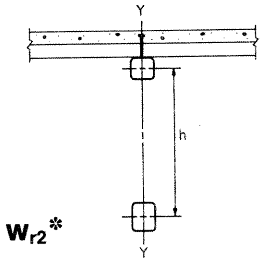
Class C HSS Top Chords (G40.21-M 350W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

Top chord section and design data	Truss Span L(mm)	h(mm) / L′ / p/h	W_{r1}+ (kN/m) 550 L/2	550 L/3	700 L/2	700 L/3	850 L/2	850 L/3	W_{r2} (kN/m) 550	700	850
HSS101.6×101.6×9.53	15 000	1.0		5.4		6.8		8.2	19.7	24.4	28.5
Mass = 25.7 kg/m		2.5		5.3		6.5		7.6	16.6	18.1	18.2
r_y = 36.8 mm	17 000	1.0		3.4		4.3		5.3	15.4	19.1	22.4
2.5t = 23.8 mm		2.5		3.4		4.2		5.0	13.2	14.6	14.9
	19 000	1.0		2.2		2.8		3.4	12.4	15.4	18.1
		2.5		2.2		2.8		3.3	10.7	12.0	12.5
HSS101.6×101.6×7.95	14 000	1.0	3.0	6.1	3.8	7.7	4.6	9.3	19.5	24.0	28.0
Mass = 22.1 kg/m		2.5	3.0	5.9	3.7	7.3	4.4	8.3	16.3	17.7	17.6
r_y = 37.6 mm	16 000	1.0		3.8		4.8		5.8	15.0	18.5	21.7
2.5t = 19.9 mm		2.5		3.7		4.7		5.5	12.7	14.1	14.3
	18 000	1.0		2.5		3.1		3.8	11.9	14.7	17.3
		2.5		2.4		3.0		3.6	10.2	11.5	11.9
HSS101.6×101.6×6.35	13 000	1.0	3.5	6.6	4.4	8.4	5.3	10.1	18.6	22.8	26.5
Mass = 18.2 kg/m		2.5	3.4	6.4	4.2	7.8	4.9	8.8	15.4	16.6	16.4
r_y = 38.4 mm	15 000	1.0	2.0	4.1	2.5	5.1	3.0	6.2	14.0	17.3	20.3
2.5t = 15.9 mm		2.5	2.0	4.0	2.5	4.9	2.9	5.7	11.9	13.1	13.2
	17 000	1.0		2.6		3.3		4.0	10.9	13.6	15.9
		2.5		2.6		3.2		3.8	9.4	10.6	10.9
HSS101.6×76.2×6.35	10 000	1.0	4.9	9.5	6.2	11.9	7.5	14.2	26.4	32.0	36.4
Mass = 15.6 kg/m		2.5	4.8	8.9	5.8	10.5	6.7	11.4	20.4	20.6	19.1
r_y = 29.3 mm	12 000	1.0		5.1		6.4		7.7	18.6	22.7	26.2
2.5t = 15.9 mm		2.5		4.9		6.0		6.8	15.0	15.7	15.1
	14 000	1.0		2.9		3.6		4.4	13.7	16.9	19.6
		2.5		2.8		3.5		4.1	11.4	12.3	12.2
HSS76.2×76.2×7.95	10 000	1.0	4.3	8.6	5.4	10.7	6.5	12.7	25.9	30.8	34.1
Mass = 15.8 kg/m		2.5	4.1	7.9	5.0	9.1	5.7	9.6	17.9	16.8	14.6
r_y = 27.2 mm	12 000	1.0		4.5		5.7		6.9	18.2	22.0	24.8
2.5t = 19.9 mm		2.5		4.4		5.3		5.9	13.3	13.0	11.6
	14 000	1.0		2.5		3.2		3.8	13.5	16.4	18.7
		2.5		2.5		3.0		3.5	10.2	10.3	9.4
HSS76.2×76.2×6.35	9 000	1.0	5.7	10.6	7.1	13.2	8.5	15.5	26.4	31.2	34.2
Mass = 13.1 kg/m		2.5	5.4	9.5	6.3	10.5	6.9	10.6	17.8	16.4	14.2
r_y = 28.0 mm	11 000	1.0	2.6	5.4	3.3	6.7	4.0	8.1	18.0	21.6	24.2
2.5t = 15.9 mm		2.5	2.5	5.1	3.1	6.0	3.6	6.5	13.0	12.5	11.2
	13 000	1.0		2.9		3.7		4.5	13.0	15.8	18.0
		2.5		2.9		3.5		4.0	9.8	9.8	9.0



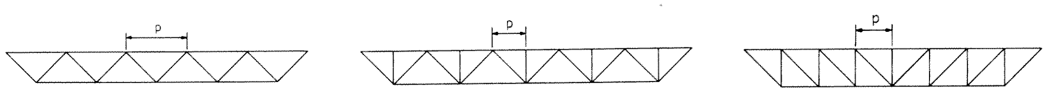
* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.
+ Steel truss should have an axis of symmetry in its plane.

Table 5.6
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table



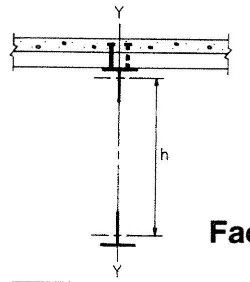
Class H HSS Top Chords (G40.21-M 350W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

Top chord section and design data	Truss Span L(mm)	h(mm) → / L′ → / p/h ↓	W_{r1}+ (kN/m) 550 L/2	550 L/3	700 L/2	700 L/3	850 L/2	850 L/3	W_{r2} (kN/m) 550	700	850
HSS101.6×101.6×9.53	15 000	1.0		6.0		7.6		9.1	19.9	24.9	29.4
Mass = 25.7 kg/m		2.5		5.8		7.2		8.3	17.7	19.7	19.8
r_y = 36.8 mm	17 000	1.0		3.6		4.6		5.6	15.5	19.5	23.2
2.5t = 23.8 mm		2.5		3.6		4.5		5.3	14.1	15.9	16.4
	19 000	1.0		2.4		3.0		3.6	12.5	15.7	18.7
		2.5		2.3		2.9		3.5	11.5	13.2	13.7
HSS101.6×101.6×7.95	14 000	1.0	3.2	6.8	4.1	8.7	4.9	10.4	19.6	24.5	28.8
Mass = 22.1 kg/m		2.5	3.2	6.6	4.0	8.1	4.7	9.2	17.4	19.1	19.1
r_y = 37.6 mm	16 000	1.0		4.1		5.3		6.4	15.1	18.9	22.4
2.5t = 19.9 mm		2.5		4.1		5.1		5.9	13.6	15.3	15.6
	18 000	1.0		2.6		3.3		4.0	11.9	15.0	17.9
		2.5		2.6		3.2		3.8	10.9	12.5	13.0
HSS101.6×101.6×6.35	13 000	1.0	3.7	7.7	4.7	9.7	5.6	11.6	18.7	23.3	27.3
Mass = 18.2 kg/m		2.5	3.6	7.4	4.5	8.9	5.2	9.9	16.4	17.9	17.7
r_y = 38.4 mm	15 000	1.0	2.1	4.5	2.7	5.7	3.2	6.9	14.1	17.6	20.9
2.5t = 15.9 mm		2.5	2.1	4.4	2.6	5.4	3.1	6.3	12.7	14.2	14.4
	17 000	1.0		2.8		3.6		4.3	11.0	13.8	16.4
		2.5		2.8		3.4		4.1	10.1	11.5	11.9
HSS101.6×76.2×6.35	10 000	1.0	5.2	10.9	6.6	13.7	7.9	16.3	26.7	32.7	37.5
Mass = 15.6 kg/m		2.5	5.1	10.1	6.2	11.8	7.0	12.6	21.6	22.0	20.4
r_y = 29.3 mm	12 000	1.0		5.6		7.1		8.5	18.7	23.2	27.0
2.5t = 15.9 mm		2.5		5.4		6.5		7.4	15.9	16.9	16.3
	14 000	1.0		3.1		3.9		4.7	13.8	17.3	20.3
		2.5		3.0		3.7		4.3	12.2	13.3	13.2
HSS76.2×76.2×7.95	10 000	1.0	4.6	9.7	5.8	12.1	6.9	14.3	26.6	32.0	35.8
Mass = 15.8 kg/m		2.5	4.4	8.8	5.3	10.1	6.0	10.5	19.3	18.2	15.8
r_y = 27.2 mm	12 000	1.0		4.9		6.2		7.4	18.7	22.9	26.1
2.5t = 19.9 mm		2.5		4.7		5.6		6.2	14.5	14.2	12.7
	14 000	1.0		2.7		3.4		4.1	13.9	17.1	19.8
		2.5		2.6		3.2		3.7	11.2	11.3	10.4
HSS76.2×76.2×6.35	9 000	1.0	6.1	12.4	7.6	15.4	9.1	18.0	27.0	32.3	35.8
Mass = 13.1 kg/m		2.5	5.7	10.9	6.7	11.8	7.3	11.7	19.1	17.7	15.1
r_y = 28.0 mm	11 000	1.0	2.8	6.0	3.5	7.5	4.2	8.9	18.4	22.4	25.4
2.5t = 15.9 mm		2.5	2.7	5.6	3.3	6.6	3.8	7.1	14.1	13.6	12.1
	13 000	1.0		3.1		4.0		4.8	13.3	16.4	18.9
		2.5		3.0		3.7		4.2	10.7	10.7	9.8

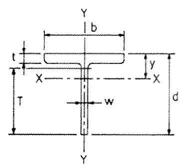


* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.
\+ Steel truss should have an axis of symmetry in its plane.

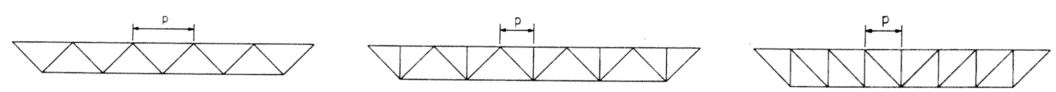
Table 5.7
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table



WT Top Chords (G40.21-M 300W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

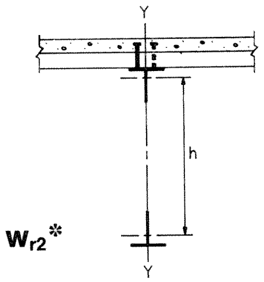


Top chord section and design data		Truss Span L(mm)	h(mm) → / L′ → / p/h ↓	W_{r1}+ (kN/m) 650 L/2	650 L/3	800 L/2	800 L/3	950 L/2	950 L/3	W_{r2} (kN/m) 650	800	950
WT180×39.5		20 000	1.0		5.2		6.3		7.5	17.2	20.5	23.3
V_r = 296 kN	w = 9.4 mm		2.5		5.0		5.8		6.5	13.9	14.5	14.2
b = 205 mm	y = 35.0 mm	21 500	1.0		4.0		4.9		5.8	15.0	17.9	20.4
t = 16.8 mm	T = 141 mm		2.5		3.9		4.6		5.2	12.3	13.0	12.9
d = 177 mm		23 000	1.0		3.2		3.9		4.6	13.1	15.7	18.0
			2.5		3.1		3.7		4.2	10.9	11.7	11.7
WT180×36‡		18 000	1.0	3.4	6.5	4.2	7.9	4.9	9.3	18.7	22.1	24.9
V_r = 268 kN	w = 8.6 mm		2.5	3.3	6.1	3.9	7.0	4.4	7.6	14.5	14.7	14.0
b = 204 mm	y = 34.2 mm	19 500	1.0		5.0		6.1		7.1	16.0	19.0	21.5
t = 15.1 mm	T = 141 mm		2.5		4.7		5.5		6.1	12.7	13.1	12.6
d = 175 mm		21 000	1.0		3.9		4.7		5.5	13.9	16.5	18.8
			2.5		3.7		4.4		4.9	11.2	11.7	11.5
WT180×28.5‡		15 000	1.0	3.5	6.8	4.3	8.2	5.0	9.6	20.2	23.5	26.0
V_r = 252 kN	w = 7.9 mm		2.5	3.3	6.2	3.9	7.0	4.4	7.4	14.5	14.2	12.9
b = 172 mm	y = 39.2 mm	16 500	1.0		4.9		6.0		7.0	16.8	19.7	22.0
t = 13.1 mm	T = 149 mm		2.5		4.6		5.3		5.8	12.6	12.5	11.6
d = 179 mm		18 000	1.0		3.6		4.4		5.2	14.2	16.8	18.9
			2.5		3.5		4.1		4.5	10.9	11.1	10.5
WT155×33.5		17 000	1.0	4.1	7.6	5.0	9.2	5.9	10.7	19.5	22.8	25.3
V_r = 232 kN	w = 8.5 mm		2.5	3.9	6.9	4.6	7.7	5.0	8.1	14.2	13.8	12.7
b = 204 mm	y = 28.7 mm	18 500	1.0	3.0	5.7	3.6	7.0	4.3	8.2	16.6	19.5	21.8
t = 14.6 mm	T = 121 mm		2.5	2.9	5.3	3.4	6.1	3.8	6.6	12.4	12.4	11.5
d = 153 mm		20 000	1.0		4.4		5.4		6.3	14.3	16.9	19.0
			2.5		4.2		4.8		5.3	10.9	11.1	10.5
WT155×30‡		15 000	1.0	5.7	10.1	7.0	12.1	8.1	13.9	21.7	25.0	27.2
V_r = 203 kN	w = 7.5 mm		2.5	5.3	8.6	5.9	9.2	6.3	9.1	14.6	13.7	12.1
b = 203 mm	y = 27.5 mm	16 500	1.0	4.0	7.4	4.9	8.9	5.8	10.3	18.2	21.1	23.2
t = 13.1 mm	T = 122 mm		2.5	3.8	6.6	4.4	7.3	4.8	7.5	12.7	12.2	11.0
d = 152 mm		18 000	1.0	2.9	5.5	3.5	6.7	4.2	7.8	15.4	18.0	19.9
			2.5	2.8	5.1	3.3	5.7	3.6	6.1	11.1	10.9	10.0
WT155×26‡		14 000	1.0	4.3	8.1	5.3	9.8	6.2	11.3	21.8	25.2	27.5
V_r = 214 kN	w = 7.6 mm		2.5	4.1	7.2	4.7	8.0	5.1	8.2	15.0	14.2	12.7
b = 167 mm	y = 32.9 mm	15 500	1.0	2.9	5.8	3.6	7.0	4.2	8.2	18.0	20.9	23.1
t = 13.2 mm	T = 128 mm		2.5	2.8	5.3	3.3	6.0	3.7	6.4	12.9	12.5	11.4
d = 158 mm		17 000	1.0		4.2		5.2		6.0	15.1	17.7	19.7
			2.5		4.0		4.6		5.0	11.2	11.1	10.3



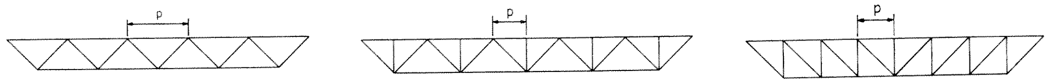
* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.
‡ Class 4 section due to stem slenderness. w_{r1} and w_{r2} values have been computed based on properties of a reduced tee section whose d/w ratio satisfies class 3 limit.
\+ Steel truss should have an axis of symmetry in its plane.

Table 5.7 (continued)
COMPOSITE TRUSS STEEL TOP CHORD
Trial Selection Table



WT Top Chords (G40.21-M 300W)
Factored Resistances to U.D.L. While Placing DECK, w_{r1}*
Factored Resistances to U.D.L. While Placing CONCRETE, w_{r2}*

Top chord section and design data	Truss Span L(mm)	h(mm) / L′ / p/h	w_{r1}^{+} (kN/m) 650 L/2	650 L/3	800 L/2	800 L/3	950 L/2	950 L/3	w_{r2} (kN/m) 650	800	950
WT155×22.5‡ V_r = 183 kN w = 6.6 mm b = 166 mm y = 32.1 mm t = 11.2 mm T = 128 mm d = 156 mm	12 500	1.0	9.4	9.5	6.5	11.2	7.5	12.7	21.4	24.1	25.5
		2.5	4.8	7.7	5.3	8.0	5.4	7.6	12.9	11.5	9.8
	13 500	1.0	4.0	7.4	4.8	8.8	5.6	10.1	18.6	21.1	22.6
		2.5	3.7	6.3	4.1	6.7	4.4	6.6	11.7	10.6	9.2
	14 500	1.0	3.0	5.8	3.7	7.0	4.3	8.1	16.3	18.7	20.1
		2.5	2.8	5.1	3.3	5.6	3.6	5.7	10.6	9.8	8.6
WT125×22.5 V_r = 180 kN w = 7.6 mm b = 148 mm y = 27.8 mm t = 13.0 mm T = 110 mm d = 133 mm	12 500	1.0	4.8	8.9	5.8	10.7	6.7	12.2	23.2	26.2	27.9
		2.5	4.4	7.6	4.9	8.0	5.2	7.9	14.4	13.0	11.2
	13 500	1.0	3.6	6.9	4.3	8.3	5.1	9.6	20.1	23.0	24.7
		2.5	3.3	6.1	3.8	6.6	4.2	6.7	13.0	12.0	10.4
	14 500	1.0		5.4		6.6		7.6	17.6	20.3	22.0
		2.5		4.9		5.5		5.7	11.8	11.0	9.7
WT125×19.5‡ V_r = 154 kN w = 6.6 mm b = 147 mm y = 26.7 mm t = 11.2 mm T = 110 mm d = 131 mm	11 500	1.0	5.5	9.8	6.6	11.7	7.7	13.2	23.0	25.7	26.9
		2.5	4.9	7.9	5.3	8.1	5.5	7.7	13.5	11.9	10.0
	12 500	1.0	4.0	7.5	4.9	9.0	5.6	10.2	19.8	22.3	23.7
		2.5	3.7	6.3	4.1	6.7	4.4	6.6	12.2	10.9	9.4
	13 500	1.0	3.0	5.8	3.6	7.0	4.2	8.1	17.2	19.6	21.0
		2.5	2.8	5.1	3.2	5.5	3.5	5.6	11.0	10.0	8.7
WT125×16.5‡ V_r = 140 kN w = 6.1 mm b = 146 mm y = 27.3 mm t = 9.1 mm T = 110 mm d = 129 mm	10 000	1.0	7.1	12.0	8.4	13.9	9.5	15.3	24.1	26.2	26.6
		2.5	5.8	8.7	6.0	8.4	5.8	7.5	12.7	10.7	8.8
	11 000	1.0	5.1	8.9	6.1	10.5	7.0	11.8	20.4	22.5	23.3
		2.5	4.4	7.0	4.7	7.0	4.8	6.5	11.4	9.8	8.2
	12 000	1.0	3.6	6.7	4.4	8.0	5.1	9.1	17.5	19.5	20.5
		2.5	3.3	5.6	3.7	5.8	3.8	5.6	10.2	9.0	7.6
WT100×15.5 V_r = 120 kN w = 6.4 mm b = 134 mm y = 21.1 mm t = 10.2 mm T = 85 mm d = 105 mm	9 000	1.0	9.2	15.3	10.7	17.3	11.8	18.2	27.2	28.2	27.3
		2.5	6.8	9.5	6.6	8.4	5.9	7.1	12.0	9.6	7.6
	10 000	1.0	6.5	11.1	7.7	12.9	8.6	14.0	22.8	24.2	24.0
		2.5	5.2	7.7	5.3	7.2	5.0	6.3	10.8	8.9	7.1
	11 000	1.0	4.6	8.3	5.5	9.7	6.2	10.8	19.4	21.0	21.2
		2.5	3.9	6.2	4.1	6.1	4.1	5.5	9.8	8.2	6.7
WT100×13.5 V_r = 107 kN w = 5.8 mm b = 133 mm y = 21.1 mm t = 8.4 mm T = 86 mm d = 104 mm	8 000	1.0	10.9	17.5	12.5	19.2	13.5	19.7	28.0	28.5	27.0
		2.5	7.4	9.8	6.8	8.4	6.0	6.9	11.6	9.1	7.1
	9 000	1.0	7.5	12.5	8.7	14.2	9.7	15.1	23.2	24.2	23.6
		2.5	5.6	8.0	5.5	7.2	5.1	6.1	10.5	8.4	6.7
	10 000	1.0	5.2	9.1	6.2	10.5	7.1	11.5	19.5	20.8	20.7
		2.5	4.3	6.5	4.4	6.1	4.2	5.4	9.4	7.7	6.2



* If truss spacing is less than 2 500 mm and in the case of spandrel trusses, check as to whether concentrated construction load case is critical.

‡ Class 4 section due to stem slenderness. w_{r1} and w_{r2} values have been computed based on properties of a reduced tee section whose d/w ratio satisfies class 3 limit.

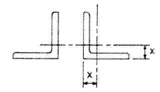
\+ Steel truss should have an axis of symmetry in its plane.

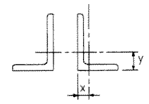
WEB TENSION MEMBER
Trial Selection Tables

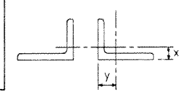
Factored Axial Tensile Resistances, T_r in kN

Table 5.8 Double Angle Tension Members

	Double Angles*	Mass kg/m	x mm	y mm	T_r kN
Equal Leg Angles	2L75×75×10	22.0	22.4		756
	2L65×65×10	18.8	19.9		648
	2L55×55×10	15.7	17.4		540
	2L55×55×8	12.8	16.7		440
	2L45×45×8	10.3	14.2		354
	2L35×35×6	6.03	10.9		207
	2L35×35×5	5.10	10.6		175
	2L25×25×5	3.53	8.06		121
Unequal Leg Angles	2L90×65×10	22.8	17.3	29.8	783
	2L80×60×10	20.4	16.5	26.5	702
	2L80×60×8	16.6	15.8	25.8	570
	2L75×50×8	14.7	13.0	25.5	505
	2L65×50×8	13.4	13.8	21.3	462
	2L65×50×5	8.63	12.7	20.2	258
	2L55×35×6	7.93	9.04	19.0	273
	2L45×30×6	6.50	8.22	15.7	224



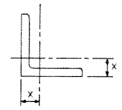


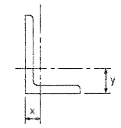


* G40.21-M 300W Steel

Table 5.9 Single Angle Tension Members

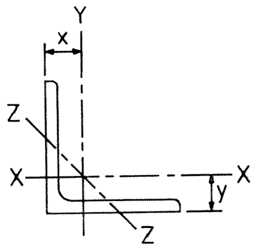
	Single Angles*	Mass kg/m	x mm	y mm	T_r kN
Equal Leg Angles	L100×100×13	19.1	29.8	29.8	656
	L90×90×13	17.0	27.2	27.2	586
	L75×75×13	14.0	23.5	23.5	481
	L65×65×10	9.42	19.9	19.9	324
	L55×55×10	7.85	17.4	17.4	270
	L55×55×8	6.41	16.7	16.7	220
	L45×45×8	5.15	14.2	14.2	177
	L45×45×6	3.96	13.4	13.4	136
	L35×35×6	3.01	10.9	10.9	104
	L35×35×5	2.55	10.6	10.6	87.8
	L25×25×5	1.77	8.06	8.06	60.8
Unequal Leg Angles	L125×90×13	20.6	23.7	41.2	710
	L100×90×13	18.1	26.1	31.1	621
	L100×75×13	16.5	20.9	33.4	570
	L90×75×13	15.5	21.8	29.3	535
	L90×75×10	12.2	20.7	28.2	419
	L90×65×10	11.4	17.3	29.8	392
	L80×60×10	10.2	16.5	26.5	351
	L80×60×8	8.29	15.8	25.8	286





* G40.21-M 300W Steel

Table 5.10
SINGLE ANGLE WEB STRUTS*
(with one leg welded to chords)



Factored Compressive Resistances, C_{re}#, in kN

G40.21-M 300W

Single Angle Struts	Mass kg/m	x or y mm	Strut lengths between intersections of axes of strut and chords, L, (mm)									
			600	700	800	900	1000	1100	1200	1300	1400	1500
L100×100×16	23.1	30.8	393	388	381	375	368	360	352	344	335	325
L100×100×13	19.1	29.8	331	327	322	316	310	304	297	290	282	274
L100×100×10	14.9	28.7	264	260	256	252	247	242	237	231	225	218
L90×90×13	17.0	27.2	291	286	280	275	268	262	254	247	238	230
L90×90×10	13.3	26.2	233	229	224	220	215	209	203	197	191	183
L90×90×8	10.8	25.5	191	188	184	180	176	172	167	162	157	151
L75×75×13	14.0	23.5	229	224	218	212	205	197	189	180	170	159
L75×75×10	11.0	22.4	185	181	176	171	165	159	153	145	137	128
L75×75×8	8.92	21.7	153	149	146	141	137	132	126	120	113	106
L65×65×10	9.42	19.9	153	149	144	138	132	125	117	109	102	95
L65×65×8	7.66	19.2	127	123	119	114	109	103	97	90	84	78
L65×65×6	5.84	18.5	99	96	93	89	85	81	75	70	65	61
L55×55×10	7.85	17.4	121	116	111	105	97	89	82	76	71	66
L55×55×8	6.41	16.7	101	97	92	87	81	74	68	63	58	54
L55×55×6	4.90	16.0	79	76	72	68	63	58	54	49	46	42
L55×55×5	4.12	15.6	68	65	61	58	54	49	45	42	39	36
L45×45×8	5.15	14.2	75	70	65	59	53	48	44	40	37	34
L45×45×6	3.96	13.4	60	56	52	46	42	38	35	32	29	27
L45×45×5	3.34	13.1	51	48	44	40	36	32	30	27	25	23
L35×35×6	3.01	10.9	39	35	30	27	24	22	19	17	15	14
L35×35×5	2.55	10.6	34	30	26	23	21	18	17	15	13	12
L25×25×5	1.77	8.1	17	14	12	10	9					

Force resultant assumed to act through centroid of attached leg and half the moment caused by such eccentricity distributed to the strut. K = 0.9
* Multiply C_{re} values by 2 **for angle struts in pairs.**

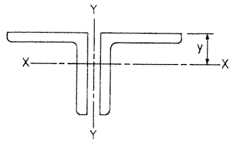


Table 5.11
DOUBLE ANGLE STRUTS
(interconnected at mid length)

Factored Axial Compressive Resistances, C_r+, in kN
Legs 6 mm Back to Back*

G40.21-M 300W

Double angle Struts	Mass kg/m	x or y mm	M_{rx} kN•m	Strut lengths between intersections of axes of strut and chords, L‡ (mm)								
				700	800	900	1000	1100	1200	1300	1400	1500
2L90×90×10	26.7	26.2	10.9	788	785	782	778	773	767	759	750	739
2L90×90×8	21.6	25.5	8.91	589	588	586	584	581	577	573	568	562
2L75×75×10	22.0	22.4	7.45	673	669	662	654	643	631	618	601	582
2L75×75×8	17.8	21.7	6.10	518	515	512	507	501	494	486	476	464
2L65×65×10	18.8	19.9	5.48	585	576	566	551	534	516	497	477	457
2L65×65×8	15.3	19.2	4.51	458	454	448	440	430	419	406	390	374
2L65×65×6	11.7	18.5	3.48	322	320	317	313	309	303	296	288	279
2L55×55×10	15.7	17.4	3.83	481	465	448	431	412	392	371	349	325
2L55×55×8	12.8	16.7	3.16	388	380	367	353	338	322	305	287	269
2L55×55×6	9.80	16.0	2.45	282	278	273	266	258	249	237	224	210
2L55×55×5	8.24	15.6	2.08	224	221	218	214	208	202	194	186	177
2L45×45×8	10.3	14.2	2.07	299	285	270	253	236	218	197	178	162
2L45×45×6	7.91	13.4	1.61	231	221	210	198	185	172	156	142	129
2L45×45×5	6.67	13.1	1.37	189	183	177	168	157	146	133	120	110
2L35×35×6	6.03	10.9	0.94	159	147	133	118	104	92	83	75	68
2L35×35×5	5.10	10.6	0.81	135	125	114	101	89	79	71	64	58
2L25×25×5	3.53	8.10	0.39	70	58	50	43	37	33	29	25	22

* If angles are connected to WT-chords with stems thicker than 6 mm, C_r values err no more than 1% on the conservative side.
‡ C_r values computed based on $K_x = 0.9$; $K_y = 1$.
+ Concentrically loaded. Resistance to torsional-flexural instability computed by means of an equivalent radius of gyration method.

Table 5.12
HSS WARREN POSTS (CLASS C)

Factored Compressive Resistances, Cr, in kN
G40.21-M 350W

CLASS C HSS		Mass kg/m	Post lengths between intersections of axes of post and chords, L† in millimetres					
			600	700	800	900	1000	1100
SQUARE HSS	38.1×38.1×3.81	3.81	129	121	114	105	96	86
	38.1×38.1×3.18	3.28	112	106	99	92	85	76
	38.1×38.1×2.54	2.71	93	88	83	77	71	65
	31.8×31.8×2.54	2.20	70	65	59	53	46	40
	25.4×25.4×3.18	2.01	54	46	38	33	28	24
	25.4×25.4×2.54	1.69	47	41	34	29	25	22
ROUND HSS	48.3×3.18	3.54	125	120	114	108	101	94
	42.2×3.18	3.06	104	98	92	85	78	70
	42.2×2.54	2.48	85	80	75	70	64	58
	33.4×2.54	1.93	60	54	49	42	36	32
	26.7×3.18	1.84	48	39	33	28	24	21
	26.7×2.54	1.51	40	33	28	24	20	18

†K = 1.0

Table 5.13
HSS WARREN POSTS (CLASS H)

Factored Compressive Resistances, Cr, in kN
G40.21-M 350W

CLASS H HSS		Mass kg/m	Post lengths between intersections of axes of post and chords, L† in millimetres					
			600	700	800	900	1000	1100
SQUARE HSS	38.1×38.1×3.81	3.81	143	138	132	125	116	107
	38.1×38.1×3.18	3.28	124	120	115	109	102	94
	38.1×38.1×2.54	2.71	103	99	95	91	85	79
	31.8×31.8×2.54	2.20	80	76	71	65	58	50
	25.4×25.4×3.18	2.01	65	57	48	39	32	28
	25.4×25.4×2.54	1.69	56	50	43	35	29	25
ROUND HSS	48.3×3.18	3.54	136	133	129	124	119	113
	42.2×3.18	3.06	115	111	106	101	94	87
	42.2×2.54	2.48	94	91	87	83	77	72
	33.4×2.54	1.93	68	64	59	53	46	38
	26.7×3.18	1.84	57	50	41	33	27	23
	26.7×2.54	1.51	48	42	35	28	23	20

†K = 1.0

Table 5.14
I_s/h^2 Values in mm^2

HSS CHORDS

Top Chord / Bottom Chord	Size (mm)	101.6×101.6			101.6×76.2	76.2×76.2	
	Thickness (mm)	9.53	7.95	6.35	6.35	7.95	6.35
	Mass (kg/m)	25.7	22.1	18.2	15.6	15.8	13.1
152.4×101.6×11.13	38.0	1660	1510				
152.4×101.6×9.53	33.3	1570	1440	1270			
152.4×101.6×7.95	28.4	1460	1350	1200			
152.4×101.6×6.35	23.2	1320	1230	1110			
101.6×101.6×9.53	25.7	1390	1290	1160			
101.6×101.6×7.95	22.1		1200	1080			
127.0×76.2×7.95	22.1				992	998	
127.0×76.2×6.35	18.2				911	915	825
127.0×76.2×4.78	14.1				801	805	734
101.6×76.2×9.53	21.9				987	993	888
101.6×76.2×7.95	18.9				926	932	838
101.6×76.2×6.35	15.6				846	850	772

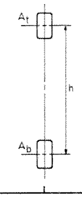
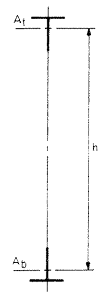


Table 5.15
I_s/h^2 Values in mm^2

WT CHORDS

Bottom Chord	Top Chord	WT180			WT155				WT125			WT100	
	Mass kg/m	39.5	36	28.5	33.5	30	26	22.5	22.5	19.5	16.5	15.5	13.5
WT180×39.5		2150	2030	1790	1960	1840							
WT180×36			1930	1710	1870	1760	1630						
WT180×32				1630	1770	1670	1560	1420					
WT180×28.5				1530	1660	1570	1470	1350	1360				
WT180×25.5						1480	1390	1280	1290	1190			
WT180×22.5							1310	1210	1220	1130	1030		
WT155×43		2240	2120		2040	1910							
WT155×39.5		2140	2030	1790	1960	1840							
WT155×37			1970	1740	1900	1790	1660						
WT155×33.5				1660	1810	1710	1590	1450					
WT155×30				1570	1710	1620	1510	1380	1390				
WT155×26						1510	1420	1300	1310	1200			
WT155×22.5							1300	1210	1210	1120	1020		
WT155×19.5								1120	1130	1050	960	939	
WT125×29							1490	1370	1370				
WT125×24.5							1370	1260	1270	1170	1060		
WT125×22.5								1210	1220	1120	1020	1000	
WT125×19.5									1120	1050	958	938	852
WT125×16.5										958	884	867	793
WT100×18											926	907	827
WT100×15.5											867	850	779
WT100×13.5												779	718



Steel truss moment of inertia* (mm^4), $I_s = (I_s/h^2)$ values tabulated multiplied by h^2

$$I_s = 0.85\left[\left(\frac{A_b}{A_t + A_b}\right)^2 A_t + \left(1 - \frac{A_b}{A_t + A_b}\right)^2 A_b\right] h^2$$

h = vertical distance between centroids of chords in mm and 15% reduction due to open web effect included.