10.1 OBJECTIVE

The objective of the lesson is to explain the various LRFD provisions related to some of the component-specific or structure-specific provisions.

10.2 SPECIFIC PROVISIONS FOR VARIOUS TYPE OF STRUCTURES

Article S5.14 specifies provisions for:

- beams and girders,
- segmental construction,
- arches,
- slab superstructures, and
- culverts.

10.2.1 Beams and Girders

Beams and girders are defined as linear elements, either partial or full span, and either longitudinal or transverse.

The provisions of Article S5.14.1 apply to the design of cast-in-place and precast beams and girders with rectangular, I, Tee, bulb-Tee, double-Tee, open and closed box sections. These provisions supplement the appropriate provisions of other articles of the Specifications.

Precast concrete beams are discussed in Article S5.14.1.2.

In the design of precast concrete components, all loading, restraint and instability conditions from initial fabrication to completion of the structure, including, but not limited to, form removal, storage, transportation and erection must be considered. For transportation and erection, the component should be designed for not less than 1.5 times its self-weight.

Slender precast components may need temporary braces or harnesses during transportation and erection to provide stability. The stiffness of such devices could, therefore, be as important as their strength.
If critical to the safety of the component during transportation or erection, the locations of temporary braces and support points and their minimum resistance and stiffness should be indicated in the contract documents.

Division II places the responsibility for providing adequate devices and methods for the safe storage, handling, erection and temporary bracing of precast members on the Contractor. If the design is such that limits need be placed on the locations or magnitude of temporary supports or lateral restraints, these limits should be shown in the contract documents.

Precast beams may resist transient loads with or without a superimposed deck. Where a structurally separate concrete deck is applied, it must be made composite with the precast beams.

Minimum and maximum dimensions for precast members are discussed in Article S5.14.1.2.2.

The maximum dimensions and weight of precast members manufactured at an off-site casting yard shall conform to local hauling restrictions. For highway transportation, the permissible load size and weight limits are constantly being revised. For large members, an investigation should be made prior to design to ensure transportability.

The thickness of any part of precast concrete beams shall not be less than:

- top flange: 50 mm
- web, non post-tensioned: 125 mm
- web, post-tensioned: 165 mm
- bottom flange: 125 mm

The 50 mm minimum dimension relates to bulb-Tee and double-Tee types of girders on which cast-in-place decks are used. The 125 and 165 mm web thicknesses have been successfully used by contractors experienced in working to close tolerances. The 125 mm limit for bottom flange thickness normally relates to box-type sections.

Field splices may be used where precast members exceed transportable lengths. Such splices shall conform to the provisions, as specified in either Article S5.14.1.2.6 or in Article S5.14.2.4.2.

Division II allows the Contractor to select the type of lifting device for precast members and assign responsibility to him for their performance. Anchorages for lifting devices generally consist of loops of prestressing strand or mild steel bars, with their tails
embedded in the concrete or threaded anchorage devices cast into the concrete.

If it is anticipated that anchorages for lifting devices will be cast into the face of a member that will be exposed to view or to corrosive materials in the completed structure, any restriction on locations of embedded lifting devices, the depth of removal and the method of filling the cavities after removal shall be shown in the contract documents. The depth of removal shall be not less than the depth of cover required for the reinforcing steel.

The design of all details of precast beams must be shown in the contract documents. These include details of reinforcement, connections, bearing seats, inserts or anchors for diaphragms, concrete cover, openings, and fabrication and erection tolerances. For any details left to the Contractor's choice, such as prestressing materials or methods, the submittal and review of working drawings shall be required.

Article S5.14.1.2.5 allows the designer to take advantage of the typical age of precast components (greater than 90 days) when a bridge is opened to traffic.

For slow curing concretes, such as those containing fly-ash, the 90-day compressive strength may be used for all stress combinations which take place after 90 days.

For normal density concrete, the 90-day strength of slow curing concretes may be estimated as 115% of their 28 days strength.

Article S5.14.1.2.6 codifies the current best practice with regard to transverse construction joints, allowing the designer considerable latitude to formulate new structural systems.

In-span construction joints must be either match-cast or closure type. Joints at internal piers in continuous construction shall be of the closure type. Match-cast joints shall satisfy the requirements of Article S5.14.2.4.2. For prestressed beams, in-span construction joints must be post-tensioned.

If the closure joint exceeds 150 mm, its compressive chord section must be reinforced for confinement. The sequence of placing concrete for the closure and the slab shall be specified in the contract documents.

Precast concrete beam segments, with or without a cast-in-place slab, may be made longitudinally continuous for both permanent and transient loads with combinations of post-tensioning and reinforcement.
The width of a closure joint between precast concrete segments shall allow for the splicing of steel whose continuity is required by design considerations, and to accommodate the splicing of post-tensioning ducts, but it shall not be less than 300 mm when the joint is located in the span, and 100 mm at an internal pier. If the joint is located in the span, its web reinforcement, $A_{S}/s$, shall be the larger of that in the adjacent girders.

The intent of the joint width requirement is to allow proper compaction of concrete in the joint. At internal piers, the diaphragms may incorporate the joint, thus facilitating compaction of concrete in a narrower space.

In the case of multi-stage post-tensioning, lengths of draped ducts for tendons which are to be tensioned before the compressive strength of the slab concrete attains $f_{c}^{'}$, shall not be located in the slab. This provision is to ensure that ducts, which are not yet secured by concrete, will not be used for active post-tensioning.

Article S5.14.1.2.7 discusses simple span precast girders made continuous for live load and includes applicable provisions.

Bridges consisting of precast concrete girders and cast-in-place concrete slabs may be made continuous for transient loads by using a cast-in-place closure placement at piers, with tensile reinforcement located in the slab, or by using closure pours at other locations. At interior piers where the diaphragms contain the closure placement, the design may be based on the strength of the concrete in the precast elements.

Longitudinal reinforcement which makes, or contributes to making, the precast girder continuous over an internal pier shall be anchored in regions of the slab which can be shown to be crack-free at strength limit states and shall satisfy the requirements as specified in Article S5.11.1.2.3. This reinforcement anchorage shall be staggered. Regular longitudinal slab reinforcement may be utilized as part of the total longitudinal reinforcement required.

Recent limited tests on continuous model and full-size structural components indicate that, unless the reinforcement is anchored in a compressive zone, its effectiveness becomes questionable at strength limit state. The intent of staggering bar ends is to distribute local force effects.

If the calculated stress at the bottom of the joint for the combination of superimposed permanent loads, settlement, creep, shrinkage, 50% live load and temperature gradient, if applicable, is compressive, the joint may be considered fully effective.
It has been observed that in isolated cases, especially where the beams were green at the time of placing of the slab concrete, upward bowing of the superstructure due to creep resulted in separation of the closure joint. Such separation may cause the structure to act as simply supported at service and fatigue limit states, with continuity regained only at, or close to, strength limit state.

In addition to providing for a pouring sequence of cast-in-place concrete, the designer may impose age requirement on the use of precast concrete components.

Structures with fully-effective construction joints at the internal piers shall be designed as fully-continuous structures at all limit states for load applied after closure.

Structures with partially-effective construction joints at internal piers shall be designed as continuous structures for loads applied after closure for strength and extreme event limit states only.

If the negative moment resistance of the joint at an internal pier is less than the total amount required, positive moment resistances in the adjacent spans shall be increased appropriately for each limit state investigated.

Longitudinal construction joints are used in deck systems composed entirely of precast beams of box, tee and double-tee sections, laid side-by-side, and preferably joined together by transverse post-tensioning.

Longitudinal construction joints between precast concrete flexural components shall consist of a key filled with a non-shrinkage mortar attaining a compressive strength of 35 MPa within 24 hours. The depth of the key should not be less than 165 mm. The preferred joint is a simple v-joint.

If the components are post-tensioned together transversely, the top flanges may be assumed to act as a monolithic slab, except that the empirical slab design, as specified in Article S9.7.2, is not applicable.

The amount of transverse prestress may be determined by either the strip method or two-dimensional analysis. The transverse prestress, after all losses, shall not be less than 1.7 MPa through the key. In the last 900 mm at a free end, the required transverse prestress shall be doubled. The transverse post-tensioning tendons should be located at the centerline of the key.

The thickness of top flanges serving as deck slabs shall be:

- as specified in Section 9,
- as required for anchorage and cover for transverse prestressing, if used, and
- not less than 1/20th of the clear span between fillets, haunches or webs, unless either transverse ribs at a spacing equal to the clear span are used, or transverse prestressing is provided.

The bottom flange thickness shall be not less than:

- 140 mm,
- 1/16 of the distance between fillets or webs of non-prestressed girders and beams, or
- 1/30th of the clear span between fillets, haunches or webs for prestressed girders, unless transverse ribs at a spacing equal to the clear span are used.

The thickness of webs shall be determined by requirements for shear, torsion, concrete cover and placement of concrete.

Changes in girder web thickness shall be tapered for a minimum distance of 12.0 times the difference in web thickness.

For adequate field placement and consolidation of concrete, a minimum web thickness of 200 mm is needed for webs without prestressing ducts, 300 mm is needed for webs with only longitudinal or vertical ducts, and 380 mm is needed for webs with both longitudinal and vertical ducts. For girders over about 2400 mm in depth, these dimensions should be increased to compensate for the increased difficulty of concrete placement.

The reinforcement in the deck slab of cast-in-place T-beams and box girders may be determined by either the traditional or the empirical design methods as specified in Section 9.

Where the deck slab does not extend beyond the exterior web, at least one-third of the bottom layer of the transverse reinforcement in the deck slab shall be extended into the exterior face of the outside web and anchored by a standard 90E hook. If the slab extends beyond the exterior web, at least one-third of the bottom layer of the transverse reinforcement shall be extended into...
the slab overhang and shall have an anchorage beyond the exterior face of the web not less in resistance than that provided by a standard hook.

A uniformly distributed reinforcement of 0.4% of the flange area shall be placed in the bottom slab parallel to the girder span, either in single or double layers. The spacing of such reinforcement shall not exceed 450 mm. This provision is intended to apply to both reinforced and prestressed boxes.

A uniformly distributed reinforcement of 0.5% of the cross-sectional area of the slab, based on the least slab thickness, shall be placed in the bottom slab transverse to the girder span. Such reinforcement shall be distributed over both surfaces with a maximum spacing of 450 mm. All transverse reinforcement in the bottom slab shall be extended to the exterior face of the outside web in each group and be anchored by a standard 90E hook.

**10.2.2 Segmental Construction**

For segmental construction, generally superstructures of single or multiple box sections are used, but beam-type sections may also be utilized. Segmental construction includes construction by the free cantilever, span-by-span or incremental launching methods using either precast or cast-in-place concrete segments which are joined to produce either continuous or simple-spans.

The requirements specified in Article S5.14.2 supplement the requirements of other sections of these Specifications for concrete structures which are designed to be constructed by the segmental method.

The span length of bridges considered by these Specifications ranges to 240,000 mm. Bridges supported by stay cables are not specifically covered in this article, although many of the specification provisions are also applicable to them.

The provisions of the LRFD Specification apply only to segmental construction using normal density concrete.

Low-density concrete has been infrequently used for segmental bridge construction. Provision for the use of low-density aggregates represents a significant complication of both design and construction specifications. For these reasons, as well as questions concerning the economic benefit of use of low-density aggregates for segmental bridges, their use is not explicitly covered in the LRFD Specification.

The method of construction, assumed for the design, shall be shown in the contract documents. Temporary supports required
prior to the time the structure or component thereof capable of supporting itself and subsequently applied loads, shall also be shown in the contract documents.

Article S5.14.2.1 specifies that the method of construction and any required temporary support is of paramount importance in the design of segmental concrete bridges. Such considerations often govern over final conditions in the selection of section dimensions and reinforcing and/or prestressing.

The contract documents must state whether alternative methods of construction are permitted, and the Contractor's responsibilities if alternative methods are chosen.

For segmentally constructed bridges, designs should, and generally do, allow the Contractor some latitude in choice of construction methods. To ensure that the design features and details to be used are compatible with the proposed construction method, it is essential that the Contractor be required to prepare working drawings and calculations, based on the method of choice, for review and approval by the Engineer before the work begins.

In the absence of other criteria, the dimensional parameters, specified in Article S5.14.1.3, should be taken as applying to segmental construction.

The analysis of segmentally constructed concrete bridges is discussed in Article S5.14.2.2.

The analysis of segmentally constructed bridges must conform to the requirements of Section 4 and those specified in Article S5.14.2.2.

The analysis of the structure involves two stages:

1. the construction stage, and
2. the final structural system.

For the analysis of the structure during the construction stage, the construction load combinations, stresses and stability considerations are specified in Article S5.14.2.3.

The final structural system must be analyzed for redistribution of construction stage force effects due to internal deformations and changes in support and restraint conditions.

The LRFD Specifications require that joints in segmental girders made continuous by unbonded post-tensioning steel be investigated for the simultaneous effect of axial force, moment and
shear that may occur at a joint. These force effects, the opening of the joint and the remaining contact surface between the components must be determined by global consideration for strain and deformation. Shear is assumed to be transmitted through the contact area only.

In addition to the loads specified in Section S3, the construction loads for segmental concrete bridges are specified in Articles S5.14.2.3.2 through S5.14.2.3.4.

Construction loads and conditions frequently determine section dimensions and reinforcing and/or prestressing requirements in segmentally constructed bridges. It is important that the designer show these assumed conditions in the contract documents, therefore, the LRFD Specification requires that construction loads and conditions assumed in the design which determine section dimensions and reinforcing and/or prestressing requirements be shown as maxima allowed in the contract documents. In addition to erection loads, any required temporary supports or restraints must be either defined as to magnitude or included as part of the design. The acceptable closure forces due to misalignment corrections must be stated. Due allowance must be made for all effects of any changes of the statical structural scheme during construction and the application, changes or removal of the assumed temporary supports of special equipment taking into account residual force effects, deformations and any strain-induced effects.

These provisions are not meant to be limitations on the Contractor as to the means which may be used for construction. Controls are essential to prevent damage to the structure during construction and to ensure adequacy of the completed structure. It is also essential for the bidders to be able to determine if their equipment and proposed construction methods can be used without modifying the design or the equipment.

The following construction loads are to be considered:

\[
dc = \text{weight of the supported structure (N)} \\
\text{diff} = \text{differential load: applicable only to balanced cantilever construction, taken as 2\% of the dead load applied to one cantilever (N)} \\
\text{dw} = \text{superimposed dead load (N) or (N/mm)}
\]
CLL = distributed construction live load: an allowance for miscellaneous items of plant, machinery and other equipment, apart from the major specialized erection equipment, taken as $4.8 \times 10^{-4}$ MPa of deck area. In cantilever construction, this load is taken as $4.8 \times 10^{-4}$ MPa on one cantilever and $2.4 \times 10^{-4}$ MPa on the other. For bridges built by incremental launching, this load may be neglected (MPa).

CE = specialized construction equipment: the load from any special equipment, including a form traveler, launching gantry, beam and winch, truss or similar major auxiliary structures, segment delivery trucks and the maximum loads applied to the structure by the equipment during the lifting of segments (N).

IE = dynamic load from equipment: determined according to the type of machinery anticipated (N).

For very gradual lifting of segments, where the load involves small dynamic effects, the dynamic load IE may be taken as 10% of the lifted weight.

CLE = longitudinal construction equipment load: the longitudinal load from the construction equipment (N).

U = segment unbalance: the effect of any out of balance segments or other unusual condition as applicable. It applies primarily to balanced cantilever construction, but may be extended to include any unusual lifting sequence which may not be a primary feature of the generic construction system (N).

WS = horizontal wind load on structures in accordance with the provisions of Section 3 (MPa).

WE = horizontal wind load on equipment taken as $4.8 \times 10^{-3}$ MPa of exposed surface (MPa).

WUP = wind uplift on cantilever: $2.4 \times 10^{-4}$ MPa of deck area for balanced cantilever construction applied to one side only, unless an analysis of site conditions or structure configuration indicates otherwise (MPa).

A = static weight of precast segment being handled (N).

AI = dynamic response due to accidental release or application of a precast segment load or other sudden application of an otherwise static load to be added to the dead load, taken as 100% of load A (N).
CR = creep effects in accordance with Article S5.14.2.3.6

SH = shrinkage in accordance with Article S5.14.2.3.6

T = thermal: the sum of the effects due to uniform temperature variation (TU) and temperature gradients (TG) (DEG)

Construction load stresses are calculated at service limit states for the load combinations specified in Table 10.2.2-1.
Table 10.2.2-1 - Load Factors and Allowable Tensile Stresses for Construction Load Combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Dead Load</th>
<th>Live Load</th>
<th>Wind Load</th>
<th>Other Loads</th>
<th>Excluding &quot;Other Loads&quot;</th>
<th>Including &quot;Other Loads&quot;</th>
<th>See Note</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>DC</td>
<td>DIFF</td>
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<td>CLL</td>
<td>CE</td>
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<td>0.3</td>
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</tbody>
</table>

Note 1: equipment not working
Note 2: normal erection
Note 3: moving equipment

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The stresses in Table 1 limit construction load stresses to less than the modulus of rupture of the concrete for structures with internal tendons and Type A joints. The construction load stresses should not, therefore, generate any cracking.

The distribution and application of the individual erection loads, appropriate to a construction phase, must be selected to produce the most unfavorable effects. The construction load compressive stress in concrete is limited to 0.50 f_{1c} where f_{1c} is the compressive strength at the time of load application.

Tensile stresses in concrete due to construction loads are limited to the values specified in Table 1, except for structures with less than 60% of the tendon capacity provided by internal tendons and having Type A joints, the tensile stresses is limited to 0.25/ f_{1c}, and for structures with Type B joints, no tensile stresses are permitted.

The factored resistance of a component, determined by using resistance factors as specified in Article S5.5.4.2.2, must be greater than that required by the following factored construction load combinations at strength limit states:

- for maximum force effects:
  \[ \Sigma \phi F_u = 1.1(DL + DIFF) + 1.3CE + A + AI \]
- for minimum force effects:
  \[ \Sigma \phi F_u = DL + CE + A + AI \]

Thermal effects which may occur during the construction of the bridge must be considered.

The temperature setting variations for bearings and expansion joints must be stated in the contract documents.

The provisions of Article S3.12 relate to annual temperature variations, and should be adjusted for the actual duration of superstructure construction, as well as for local conditions.

Transverse analysis for the effects of differential temperature outside and inside box girder sections is not generally considered necessary. However, such an analysis may be necessary for relatively shallow bridges with thick webs. In that case, a ±6.0EC temperature differential is recommended.

Creep and shrinkage strains, and prestress losses which occur after closure of the structure cause a redistribution of the force effects.
Stresses must be determined for redistribution of restraint stresses developed by creep and shrinkage which are based on the assumed construction schedule as stated in the contract documents.

Creep coefficient $\Psi(t, t_i)$ shall be determined either in accordance with Article S5.4.2.3 or by comprehensive tests.

For permanent loads, the behavior of segmental bridges after closure may be approximated by use of an effective modulus of elasticity, $E_{eff}$, which may be calculated as:

$$E_{eff} = \frac{E_c}{\Psi(t, t_i)}$$

where:

$\Psi(t, t_i) = $ the creep coefficient

For determining the final post-tensioning forces, prestress losses must be calculated for the construction schedule stated in the contract documents.

Lump sum losses may be used only for preliminary design purposes.

For all considerations other than preliminary design, prestress losses must be determined as specified in Article S5.9.5, including consideration of the time-related construction method and schedule shown in the contract documents.

Experience with segmental concrete bridges to-date has often indicated higher friction and wobble losses than that calculated using the coefficients of Article S5.9.5.2.2, which were developed for conventional cast-in-place bridges, due to movement of ducts during concrete placement, and misalignment at segment joints. For this reason, in-place friction tests are recommended at an early stage in major projects as a basis for modifying friction and wobble loss values. No reasonable values for friction and wobble coefficients can be recommended to account for gross duct misalignment problems. As a means of compensating for high friction and wobble losses or for provisional post-tensioning tendons, as well as other contingencies, additional ducts are required in accordance with Article S5.14.2.3.8.

Provisions for adjustments of prestressing force to compensate for unexpected losses, either during construction or at a later time, and future dead loads, and for control of cracking and deflections, must be considered.
For bridges with internal ducts, provisional anchorage and duct capacity for negative and positive moment tendons located symmetrically about the bridge centerline must provide for an increase in the post-tensioning force during original construction. The total provisional force potential of both positive and negative moment anchorages and ducts should not be less than 5% of the total positive and negative moment post-tensioning forces, respectively. Anchorages for the provisional prestressing force should be distributed uniformly at three segment intervals along the length of the bridge.

Excess capacity may be provided by use of oversize ducts and oversize anchorage hardware at selected anchorage locations.

Any provisional ducts not used for adjustment of the post-tensioning force shall be grouted at the same time as other ducts in the span.

Provision should be made for access and for anchorage attachments, pass-through openings and deviation block attachments to permit future addition of corrosion protected unbonded external tendons located inside the box section symmetrically about the bridge centerline for a post-tensioning force of not less than 10% of the positive moment and negative moment post-tensioning force.

This provides for future addition of internal unbonded post-tensioning tendons draped from the top of the diaphragm at piers to the intersection of the web and flange at mid-span. Tendons from adjacent spans should be lapped at opposite faces of the diaphragm to provide negative moment capacity. The requirement of a force of 10% of the positive moment and negative moment post-tensioning force is an arbitrary, but reasonable value. Provision for larger amounts of post-tensioning might be developed, as necessary, to carry specific amounts of additional dead load as considered appropriate for the structure.

Bridges designed for segmentally placed superstructures must conform to the requirements specified in Article S5.14.2.4.1, based on the concrete placement method and the erection methods which are to be used.

Precast segmental bridges are normally erected by balanced cantilever, by use of erection trusses, or by progressive placement.

Bridges erected by balanced cantilever or progressive placement normally utilize internal tendons. Bridges built with erection trusses may utilize internal tendons, external tendons or combinations. Due to considerations of segment weight, span
lengths for precast segmental box girder bridges, except for cable-stayed bridges, rarely exceed 125 000 mm.

The compressive strength of precast concrete segments must not be less than 17 MPa, prior to removal from the forms, and must have a maturity equivalent to 14 days at 21°C prior to assembly into the structure.

This intends to limit the magnitude of construction deflections and to prevent erratic construction deflections and creep.

Multiple, small amplitude shear keys at match-cast joints in webs of precast segmental bridges must extend over as much of the web as is compatible with other details. Details of shear keys in webs should be similar to those shown in Figure 10.2.2-1. Shear keys must also be provided in top and bottom slabs. Keys in the top and bottom slabs may be larger single element keys.

Small amplitude shear keys in the webs are less susceptible to construction damage, which will result in loss of geometry control, than larger single element keys. Shear keys in the top and bottom flanges are less susceptible to such damage.

Joints in precast segmental bridges must be either cast-in-place closures or match cast.
Match casting is necessary to ensure control of the geometry upon reassembly of the segments.

Precast segmental bridges using internal post-tensioning tendons and bridges located in areas subject to freezing temperatures or deicing chemicals must employ Type A joints. Other precast segmental bridges may employ Type B joints, as specified in Article S5.5.4.2.2.

Epoxy serves as a lubricant during placement of the segments, prevents water intrusion, provides a seal to prevent cross-over during grouting and provides some tensile strength across the joint.

Dry joints are susceptible to freeze-thaw damage and cannot preclude the intrusion of water which may lead to corrosion of internal tendons.

For Type A joints, the prestressing system must provide a minimum compressive stress of 0.21 MPa and an average stress of 0.28 MPa across the joint until the epoxy has cured. Provisions are provided to determine the direct shear resistance of dry joints.

Division II requires this temporary stress to ensure full bond and to prevent uneven epoxy thickness. Such variations could lead to a systematic accumulation of geometric error. Large stress changes on epoxy joints should be avoided during the initial curing period.

Joints between cast-in-place segments must be specified as either intentionally roughened or keyed.

Division II requires vertical joints to be keyed. However, proper attention to roughened joint preparation is expected to assure bond between the segments, providing shear strength better than shear keys.

The width of closure joints must permit the coupling of the tendon ducts.

The provisions specified in Article S5.14.2.4.4 apply to both precast and cast-in-place cantilever construction.

Longitudinal tendons may be anchored in the webs, in the slab, or in blisters built out from the web or slab. A minimum of two longitudinal tendons must be anchored in each segment.

The cantilevered portion of the structure must be investigated for overturning during erection. The factor of safety against overturning shall not be less than 1.5 under any
combination of loads, as specified in Article S5.14.2.3.3. Minimum wind velocity for erection stability analyses shall be 90 km/hr, unless a better estimate of probable wind velocity is obtained by analysis or meteorological records.

Stability during erection may be provided by moment resisting column/superstructure connections, falsework bents or a launching girder. Loads to be considered include construction equipment, forms, stored material and wind.

The 90 km/hr corresponds to the load factor 0.30 in Table S3.4.1-1.

Continuity tendons must be anchored at least one segment beyond the point where they are theoretically required for stresses.

Tendon force requires an "induction length" due to shear lag before it may be assumed to be effective over the whole section.

The segment lengths assumed in the design must be shown on the plans. Any changes proposed by the Contractor must be supported by reanalysis of the construction and computation of the final stresses.

Lengths of segments for free cantilever construction usually range between 3000 and 5500 mm. Lengths may vary with the construction method, the span length and with the location within the span.

The form traveler weight assumed in stress and camber calculations must be stated on the plans.

Form travelers for a typical 12 000 mm wide two-lane bridge with 4500 to 4850 mm segments may be estimated to weigh 700 000 to 800 000 N. Weight of form travelers for wider two-cell box sections may range up to 1 250 000 N. Segment length is adjusted for deeper and heavier segments to control segment weight. Consultation with contractors experienced in free cantilever construction is recommended to obtain a design value for form traveler weight for a specific bridge cross-section.

Provisions must be made in design of span-by-span construction for accumulated construction stresses due to the change in the structural system as the construction progresses.

Span-by-span construction is defined as construction where the segments, either precast or cast-in-place, are assembled or cast on falsework supporting one entire span between permanent piers. The falsework is removed after application of post-tensioning to make the span capable of supporting its own weight and any.
construction loads. Additional stressing may be utilized after adjacent spans are in place to develop continuity over piers.

Stresses due to the changes in the structural system, in particular the effects of the application of a load to one system and its removal from a different system, must be accounted for. Redistribution of such stresses by creep must be taken into account and allowance made for possible variations in the creep rate and magnitude.

Incrementally launched girders are subject to reversal of moments during launching. Temporary piers and/or a launching nose may be used to reduce launching stresses.

Stresses under all stages of launching must not exceed the limits specified in Article S5.9.4 for members with bonded reinforcement through the joint and internal tendons.

Provision must be made to resist the frictional forces on the substructure during launching and to restrain the superstructure if the structure is launched down a gradient. For determining the critical frictional forces, the friction on launching bearings must be assumed to vary between 0 and 4%, whichever is critical. The upper value may be reduced to 3.5% if pier deflections and launching jack forces are monitored during construction.

These friction coefficients are only applicable to bearings employing a combination of virgin Teflon and stainless steel with a roughness of less than 2.5x10^{-3} mm.

Force effects due to the following permissible construction tolerances must be superimposed upon those resulting from gravity loads:

• in the longitudinal direction between two adjacent bearings ................................. 5 mm

• in the transverse direction between two adjacent bearings ................................. 2.5 mm

• between the fabrication area and the launching equipment in the longitudinal and transverse direction ................................. 2.5 mm

• lateral deviation at the outside of the webs ................................. 2.5 mm

The horizontal force acting on the lateral guides of the launching bearings must not be taken less than 1% of the vertical support reaction.
For stresses during construction, one-half of the force effects due to construction tolerances and one-half of the force effects due to temperature in accordance with Article 5.14.2.3 must be superimposed upon those from gravity loads. Concrete tensile stresses due to the combined moments must not exceed $0.58/ f_c$.

Piers and superstructure diaphragms at piers must be designed to permit jacking of the superstructure during all launching stages and for the installation of permanent bearings. Frictional forces during launching must be considered in the design of the substructure.

The dimensional restrictions on placement of launching bearings are shown in Figure 10.2.2-2. Eccentricity between the intersection of the centerlines of the web and the bottom slab and the centerline of the bearing is illustrated in Figure 10.2.2-3.

![Figure 10.2.2-2 - Location of Launching Pads](image)

![Figure 10.2.2-3 - Eccentric Reaction at Launching Pads](image)
Local stresses which may develop at the underside of the web during launching must be investigated. The following requirements must be satisfied:

- launching pads shall be placed not closer than 75 mm to the outside of the web,

- concrete cover between the soffit and post-tensioning ducts shall not be less than 150 mm, and

- bearing pressures at the web/soffit corner shall be investigated and the effects of ungrouted ducts and any eccentricity between the intersection of the centerlines of the web and the bottom slab and the centerline of the bearing shall be considered.

The straight tendons required for launching must be placed in the top and bottom slabs for box girders and in the lower third of the web for T-sections. Not more than 50% of the tendons must be coupled at one construction joint. Anchorages and locations for the straight tendons shall be designed for the concrete strength at the time of tensioning.

The faces of construction joints shall be provided with shear keys or a roughened surface with a minimum roughness amplitude of 6 mm. Bonded non-prestressed reinforcement must be provided longitudinally and transversely at all concrete surfaces crossing the joint and over a distance of 2100 mm on either side of the joint. Minimum reinforcing must be equivalent to No. 15 bars spaced at 125 mm.

The stresses in each cross-section change from tension to compression during launching. These tensile stresses during launching are counteracted by the straight tendons. The straight tendons are stressed at an early concrete age (e.g., 3 days).

### 10.2.3 Arches

Article S5.14.3.1 specifies that the shape of an arch shall be selected so as to minimize flexure under the effect of combined permanent and transient loads.

The requirements for concrete arch ribs are specified in Articles S5.14.3.2.

The in-plane stability of the arch rib(s) shall be investigated using a modulus of elasticity and moment of inertia appropriate for the combination of loads and moment in the rib(s).
Stability under long-term loads with a reduced modulus of elasticity may govern the stability. In this condition, there is typically little flexural moment in the rib and the appropriate modulus of elasticity is the long-term tangent modulus, and the appropriate moment of inertia is the transformed section inertia. Under transient load conditions, the appropriate modulus of elasticity is the short-term tangent modulus, and the appropriate moment of inertia is the cracked section inertia, including the effects of the factored axial load.

The value indicated may be used in stability calculations, since the scatter in predicted versus actual modulus of elasticity is greater than the difference between the tangent modulus and the secant modulus at stress ranges normally encountered.

The long-term modulus may be found by dividing the short-term modulus by the creep coefficient.

Under certain conditions the moment of inertia may be taken as the sum of the moment of inertia of the deck and the arch ribs at the quarter point. A large deflection analysis may be used to predict the in-plane buckling load. A preliminary estimate of second order moments may be made by adding to the first order moments the product of the thrust and the vertical deflection of the arch rib at the point under consideration.

In lieu of a rigorous analysis, the effective length for buckling may be estimated as the product of the arch half span length and the factor as specified in Table 10.2.3-1.

Table 10.2.3-1 - Effective Length Factors for Arch Ribs

<table>
<thead>
<tr>
<th>Rise to Span Ratio</th>
<th>3-Hinged Arch</th>
<th>2-Hinged Arch</th>
<th>Fixed Arch</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1 - 0.2</td>
<td>1.16</td>
<td>1.04</td>
<td>0.70</td>
</tr>
<tr>
<td>0.2 - 0.3</td>
<td>1.13</td>
<td>1.10</td>
<td>0.70</td>
</tr>
<tr>
<td>0.3 - 0.4</td>
<td>1.16</td>
<td>1.16</td>
<td>0.72</td>
</tr>
</tbody>
</table>

For the analysis of arch ribs, the provisions of Article S4.5.3.2.2 may be applied. When using the approximate second order correction for moment, specified in Article S4.5.3.2.2c, an estimate of the short-term secant modulus of elasticity may be calculated, as specified in Article S5.4.2.4, based on a strength of 0.40 f_c.
Arch ribs shall be reinforced as compression members. The minimum reinforcing of 1.0% of the gross concrete area shall be evenly distributed about the section of the rib. Confinement reinforcement shall be provided as required for columns.

Unfilled spandrel walls greater than 7500 mm in height shall be braced by counterforts or diaphragms.

Spandrel walls shall be provided with expansion joints, and temperature reinforcing shall be provided corresponding to the joint spacing.

The spandrel wall shall be jointed at the springline.

The spandrel fill shall be provided with effective drainage. Filters shall be provided to prevent clogging of drains with fine material.

The ACI 207.2R73 Manual of Concrete Practice contains a discussion of joint spacing and temperature reinforcement of restrained walls.

Drainage of the spandrel fill is important for durability of the concrete in the rib and in the spandrel walls and to control the unit weight of the spandrel fill. Drainage details should keep the drainage water from running down the ribs.

### 10.2.4 Slab Superstructures

Slab superstructures can be subdivided into:

- cast-in-place solid slab superstructures,
- cast-in-place voided slab superstructures, and
- precast deck bridges.

In this simple bridge superstructure, the deck slab also serves as the principal load carrying component. The concrete slab, which may be solid, voided, or ribbed, is supported directly on the substructures.

The provisions are based on the performance of relatively small span structures constructed to-date. Any significant deviation from successful past practice for larger units which may become both structurally and economically feasible under these Specifications should be reviewed carefully.
10.2.4.1 CAST-IN-PLACE SOLID SLAB SUPERSTRUCTURES

Cast-in-Place solid slab superstructures are covered in Article S5.14.4.1.

Cast-in-place longitudinally reinforced slabs may be either conventionally reinforced or prestressed and may be used as slab-type bridges and culvert tops.

The distribution of live load may be determined by a two-dimensional analysis or as specified in Article S4.6.2.3. Slabs and slab bridges designed for moment in conformance with Article S4.6.2.3, may be considered satisfactory for shear.

Edge beams must be provided, as specified in Article S9.7.1.4.

Transverse distribution reinforcement shall be placed in the bottoms of all slabs, except culvert tops or bridge slabs, where the depth of fill over the slab exceeds 600 mm. The amount of the bottom transverse reinforcement may be determined either by two-dimensional analysis, or the amount of distribution reinforcement may be taken as the percentage of the main reinforcement required for positive moment taken as:

- for longitudinal reinforced concrete construction:
  \[
  \frac{1750}{\sqrt{L}} \#50\% \tag{10.2.4.1-1}
  \]

- for longitudinal prestressed construction:
  \[
  \frac{1750}{\sqrt{L}} \frac{f_{pe}}{410} \#50\% \tag{10.2.4.1-2}
  \]

where:

\( L \) = span length (mm)

\( f_{pe} \) = effective stress in the prestressing steel after losses (MPa)

Transverse shrinkage and temperature reinforcement in the tops of slabs must conform to the requirements of Article S5.10.8.

10.2.4.2 CAST-IN-PLACE VOIDED SLAB SUPERSTRUCTURE

Article S5.14.4.2 details provisions for cast-in-place voided slab superstructures.
Cross-sections of alternative typical round-voided concrete deck system, taken between piers, are shown in Figure 10.2.4.2-1.

Figure 10.2.4.2-1 - Cross-section of Typical Voided Concrete Deck System

Cast-in-place voided slab superstructures may be post-tensioned both longitudinally and transversely.

For circular voids, the center-to-center spacing of the voids should not be less than the total depth of the slab, and the minimum thickness of concrete, taken at the centerline of the void perpendicular to the outside surface, shall not be less than 140 mm.

For rectangular voids, the transverse width of the void should not exceed 1.5 times the depth of the void, the thickness of the web between voids should not be less than 20% of the total depth of the deck, and the minimum thickness of concrete above the voids shall not be less than 175 mm.

The dimensions provided for spacing and size of voids in this article are based on past experience, and are expected to provide safe results. They may be taken as preliminary design values.

The bottom flange depth shall satisfy the requirements as specified in Article S5.14.1.3.1b.

Where the voids conform to the dimensional requirements herein, and where the void ratio does not exceed 40%, the superstructure may be analyzed as a slab, using either the provisions of Article S4.6.2.3 or a two-dimensional analysis for isotropic plates.

If the void ratio exceeds 40%, the superstructure shall be treated as cellular construction and analyzed as:

- a monolithic multi-cell box, as specified in Article S4.6.2.2.1-1, Type d, or
• an orthotropic plate, or

• a three-dimensional continuum.

Columns may be framed into the superstructure, or a single bearing may be used for the internal supports of continuous structures. A minimum of two bearings must be employed at end supports.

The transverse rotation of the superstructure must not exceed 0.5% at service limit states.

The high torsional stiffness of voided concrete decks, and the inherent stability of horizontally curved continuous structures, permits the use of a single support at internal piers. A minimum of two bearings are required at the abutments to ensure torsional stability in the end zones. If the torsional rotation requirement is not satisfied, pairs of bearings may be used at some internal piers.

A solid section at least 900 mm long, but not less than 5% of the length of the span must be provided at either end of a span. Post-tensioned anchorage zones must satisfy the requirements specified in Article 5.10.9. In the absence of more refined analysis, the solid sections of the deck may be analyzed as a transverse beam distributing forces to bridge bearings and to post-tensioning anchorages.

The intent is to provide for the distribution of concentrated post-tensioning and bearing forces to the voided sections. For relatively wide decks, the analysis of the solid sections as beams is an acceptable approximation. For deep and narrow decks, a three-dimensional analysis, or use of a strut-and-tie model is advisable.

For voided slabs, conforming to the provisions of Article S5.14.4.2.1, global and local force effects due to wheel loads need not be combined. The top flange of deck with rectangular voids may either be analyzed and designed as a framed slab, or designed with the provisions of the empirical process, as specified in Article S9.7.2.

The top part of the slab over circular voids made with steel void-formers shall be post-tensioned transversely. At the minimum thickness of concrete, the average precompression after all losses, as specified in Article S5.9.5, must not be less than 3.5 MPa. When transversely post-tensioned, no additional reinforcing steel need be applied to the concrete above the circular voids.

Transverse shrinkage and temperature steel at the bottom of the voided slab must satisfy the requirements as specified in Article S5.10.8.
Continuous voided decks should be longitudinally post-tensioned. Unless specified otherwise in this article, or required for construction purposes, additional global longitudinal reinforcement may be deemed to be unnecessary if longitudinal post-tensioning is used. The preference for longitudinal post-tensioning of continuous decks reflects the limited experience with this system in North America.

Experience indicates that due to a combination of transverse bending moment, shrinkage of concrete around the steel void former and Poisson’s effect, where steel void-formers are used, high transverse tensile stresses tend to develop at the top of the deck, resulting in excessive cracking at the centerline of the void. The minimum transverse prestress specified to counteract this tension is a conservative value. The intent of transverse temperature steel at the bottom of voided deck is also for control of cracks resulting from transverse positive moments due to post-tensioning.

The hidden solid transverse beam over an internal pier may be post-tensioned.

At internal piers, the part of the cross-section under compressive stresses may be considered as a horizontal column and reinforced accordingly.

Recent tests on two-span continuous, post-tensioned structures indicate that first failure occurs in the bottom compressive zones adjacent to the bearing at the internal pier. The failure is thought to be caused by a combination of shear and compression at those points in the bottom flange. The phenomenon is not yet clearly understood, and no specific design provisions have been developed so far. At this time, the best that can be done is to treat the bottom chord as a column with a reinforcement ratio of 1.0% and column-ties as specified in Article S5.10.6.

Adequate drainage of the voids shall be provided in accordance with the provisions of Article S2.6.6.5.

Occasional cracks, large enough to permit entry of water into the voids, may develop in these deck systems. The accumulating water adds to gravitational loads, and may cause structural damage when it freezes.

10.2.4.3 PRECAST DECK BRIDGES

Precast units may have solid, voided, box, tee- and double-tee cross-sections.
Precast concrete units placed adjacent to each other in the longitudinal direction may be joined together transversely to form a deck system. Precast concrete units may be continuous either for transient loads only or for both permanent and transient loads. Span-to-span continuity, where provided, shall be in accordance with the provisions of Article 5.14.1.2.6.

Differential creep and shrinkage due to differences in age, concrete mix, environmental and support conditions have been observed to cause internal force effects that are difficult to predict at the design phase. These force effects are often relieved by separation of the joints causing maintenance problems, as well as negatively affecting structural performance.

Where structural concrete overlay is not provided, the minimum thickness of concrete is 140 mm at the top of round voided components and 175 mm for all other components.

Precast longitudinal components may be joined together transversely by a shear key, not less than 175 mm in depth. In this case, the joint is assumed to transfer shear only. For the purpose of analysis, the longitudinal shear transfer joints is to be modeled as hinges.

The joint must be filled with non-shrinking grout with a minimum compressive strength of 35 MPa at 24 hours.

Many bridges have indications of joint distress where load transfer among the components relies entirely on shear keys, the grout being subject to extensive cracking. Long-term performance of the key joint should be investigated for cracking and separation.

Precast longitudinal components may be joined together by transverse post-tensioning, cast-in-place closure joints, a structural overlay, or a combination thereof.

These shear-flexure transfer joints are intended to provide full continuity and monolithic behavior of the deck.

Decks with shear-flexure transfer joints should be modeled as continuous plates, except that the empirical design procedure of Article S9.7.2 shall not be used. The joints must be designed as flexural components, satisfying the provisions of Article S5.14.1.2.8.

From the modeling point of view, these precast concrete deck systems are not different from cast-in-place ones of the same geometry.

Transverse post-tensioning must be uniformly distributed in the longitudinal direction. Block-outs may be used to facilitate
splicing of the post-tensioning ducts. The compressed depth of the joint must not be less than 175 mm, and the prestress after all losses shall not be less than 1.7 MPa therein.

When tensioning narrow decks, losses due to anchorage setting should be kept to a minimum. Ducts should preferably be straight and grouted.

The post-tensioning force is known to spread at an angle of 45° or larger and to attain a uniform distribution within a short distance from the cable anchorage. The economy of prestressing is also known to increase with the spacing of ducts. For these reasons, the spacing of the ducts need not be smaller than about 1200 mm or the width of the component housing the anchorages, whichever is larger.

The provisions of Article S5.14.1.2.8 shall apply to longitudinal construction joints of precast deck bridges.

Concrete in the cast-in-place closure joint should have strength comparable to that of the precast components. The width of the longitudinal joint shall be large enough to accommodate development of reinforcement in the joint, but in no case shall the width of the joint be less than 300 mm.

The thickness of structural concrete overlay must not be less than 115 mm. An isotropic layer of reinforcement shall be provided in accordance with the requirements of Article 5.10.8. The top surface of the precast components must be roughened.

The composite overlay should be regarded as a structural component and be designed and detailed accordingly.

10.2.5 Culverts

The soil-structure aspects of culvert design are specified in Section 12 of the LRFD Specification.

The provisions of Article S5.7 shall apply for the design of culverts for flexure.

The provisions of Article S5.8 apply for the design for shear unless modified herein. For slabs of box culverts under 600 mm or more fill, shear strength $V_c$ may be computed by:

$$V_c' = 0.178 \sqrt{f_{c'}^2} \% 32 \frac{A_s}{b d_e} \frac{V_u}{M_u} b d_e$$

(10.2.5-1)

but $V_c$ shall not exceed $0.332/ f_{N_e} b d_e$.
where:

\[ A_s = \text{area of reinforcing steel (mm}^2) \]

\[ d_e = \text{effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (mm)} \]

\[ V_u = \text{shear from factored loads (N)} \]

\[ M_u = \text{moment from factored loads (N@m)} \]

\[ b = \text{design width usually taken as 1 (mm)} \]

For single cell box culverts only, \( V_c \) for slabs monolithic with walls need not be taken less than \( 0.25/ f_N b d_e \), and \( V_c \) for slabs simply supported need not be taken less than \( 0.207/ f_N b d \). The quantity \( V_c d_e/M_u \) must not be taken greater than 1.0 where \( M_u \) is the factored moment occurring simultaneously with \( V_u \) at the section considered. For slabs of box culverts under less than 600 mm of fill and for sidewalls, the provisions of Articles S5.8 and S5.13.3.6 shall apply.

### 10.3 SPECIFIC MEMBERS

Provisions relating to specific members are given in Article S5.13. The members included in this article are deck slabs, deep members (diaphragms, deep beams, brackets, corbels and beam ledges), footings and piles.

#### 10.3.1 Deep Members

**General**

Deep members are those members whose depth is large relative to their span length, and, therefore, primarily subject to shear and torsion. They should be analyzed and designed by either the strut-and-tie model, as specified in Article S5.6.3, or by other recognized deep beam theory, but not using the sectional models of the LRFD Specification. Figure 10.3.1-1 illustrates the application of the strut-and-tie model to deep beams. Deep members include, but are not limited to, diaphragms, brackets, corbels and beam ledges.

**Diaphragms**

Diaphragms resist lateral forces and transmit loads to points of support. They should be essentially solid, except for access and utility holes where required. Diaphragms may be omitted where experiments or analysis demonstrate them to be unnecessary.
End diaphragms should be provided at abutments, piers and hinge joints. In certain types of construction, such as shallow I-beams and double-T beams, end diaphragms may be replaced by an edge beam made to act as a vertical frame with the ends of the beams. These frames should be designed to resist wheel loads.

Intermediate diaphragms can be provided between curved beams or where deemed necessary to provide torsional resistance. The need for such diaphragms depends upon the radius of curvature and the proportions of the flanges and webs. Intermediate diaphragms can also be provided to support the deck at points of discontinuity or at angle points in the girders.

Brackets and Corbels

Brackets or corbels are those cantilevered components in which the distance from the face of the support to the load point, $a_v$, is less than the depth of the member at the support, $d$, as illustrated in Figure 10.3.1-2. Otherwise, the component is treated as a cantilevered beam.

The section at the face of the support should be designed simultaneously to resist a factored shear, factored moment and factored horizontal tensile force, as specified in Article 10.3.1. Provisions on the proportioning and detailing of the steel reinforcement are also given in this article.

Beam Ledges

Beam ledges may be distinguished from brackets and corbels in that their width along the face of the supporting member is greater than $(W + 5a_i)$ as shown in Figure 10.3.1-6. In addition, beam ledges are supported primarily by tension ties to the supporting member, while corbels utilize a compression strut penetrating directly into the supporting member. Beam ledges are generally continuous between points of application of bearing forces. Daps should be considered to be inverted beam ledges.

Examples of beam ledges include hinges within spans and inverted T-beam caps as illustrated in Figure 10.3.1-3.
Beam ledges should be designed to resist the following force effects at the locations indicated in Figure 10.3.1-4:
Figure 10.3.1-4 - Notation and Potential Crack Locations for Ledge Beams

- flexure, shear and horizontal forces at Crack 1,
- tension in the supporting element at Crack 2,
- punching shear at Crack 3, and
- bearing at Crack 4.

Beam ledges can be designed by either the strut-and-tie model or the provisions given for shear, flexure and horizontal force, punching shear, and hanger reinforcement given respectively in Articles S5.13.2.5.2 through S5.13.2.5.5 and summarized below.

Design of beam ledges for shear is to be in accordance with the requirements for shear friction as specified in Article S5.8.4.

The width of the concrete face assumed to participate in resistance to shear is taken not to exceed $S$ or $(W + 4a_v)$ or $2c$, as illustrated in Figure 10.3.1-5.

Figure 10.3.1-5 - Design of Beam Ledges for Shear
The primary tension reinforcement shall be spaced uniformly within the region \((W + 5a_f)\) or \(2c\), as illustrated in Figure 10.3.1-6, except that the widths of these regions shall not overlap.

![Figure 10.3.1-6 - Design of Beam Ledges for Flexure and Horizontal Force](image)

The truncated pyramids, assumed as failure surfaces for punching shear, as illustrated in Figure 10.3.1-7, shall not overlap.

The area of concrete resisting the punching shear for each concentrated load is shown in Figure 10.3.1-7. The area of the truncated pyramid is approximated as the average of the perimeter of the bearing plate or pad and the perimeter at depth \(d_e\), assuming 45° slopes. If the pyramids overlap, an investigation for the combined surface areas will be necessary.

Nominal punching shear resistance, \(V_n\), shall be taken as:

- at interior pads:
  \[
  V_n = 0.328 \sqrt{f_c (\frac{W}{2} + \frac{L}{2} + d_e)} d_e
  \]  \hspace{1cm} (10.3.1-1)

- at exterior pads:
  \[
  V_n = 0.328 \sqrt{f_c (W + L + d_e)} d_e
  \]  \hspace{1cm} (10.3.1-2)

where:

- \(f_c\) = specified strength of concrete at 28 days (MPa)
- \(W\) = width of bearing plate or pad as shown in Figure 10.3.1-7 (mm)
- \(L\) = length of bearing pad as shown in Figure 10.3.1-7 (mm)
- \(d_e\) = effective depth from extreme compression fiber to centroid of tensile force (mm)
The hanger reinforcement specified herein is provided in addition to the lesser shear reinforcement required on either side of the beam reaction being supported.

The arrangement for hanger reinforcement, $A_{hr}$, in single-beam ledges is shown in Figure 10.3.1-7.

Using the notation in Figure 10.3.1-7, the nominal shear resistance, $V_n$, in N, for single-beam ledges shall be taken as:

- for the service limit state:
  \[
  V_n = \frac{A_{hr} (0.5 f_y)}{s} (W \%3 a_v) \tag{10.3.1-3}
  \]

- for the strength limit state:
  \[
  V_n = \frac{A_{hr} f_y}{s} S \tag{10.3.1-4}
  \]

where:

$A_{hr}$ = area of one leg of hanger reinforcement as illustrated in Figure 10.3.1-8 (mm$^2$)

$S$ = spacing of bearing places (mm)

$s$ = spacing of hangers (mm)

$f_y$ = yield strength of reinforcing steel (MPa)
$a_v = \text{distance from face of wall to the load as illustrated in Figure 10.3.1-8 (mm)}$

**Figure 10.3.1-8 - Single Ledge Hanger Reinforcement**

Using the notation in Figure 10.3.1-9, the nominal shear resistance of the ledges of inverted T-beams shall be the lesser of that specified by either Equations 10.3.1-4 or 10.3.1-5.

$$V_n' = (0.165 \sqrt{f_c b_f} d_f) \% A_{hr} \frac{f_y}{s} (W/2d_f) \quad (10.3.1-5)$$

where:

$d_f = \text{distance from top of ledge to compression reinforcement as illustrated in Figure 10.3.1-9 (mm)}$

The edge distance between the exterior bearing pad and the end of the inverted T-beam shall not be less than $d_f$.

**Figure 10.3.1-9 - Inverted T-beam Hanger Reinforcement**

Inverted T-beams must also satisfy the torsional moment provisions as specified in Articles S5.8.3.6 and S5.8.2.1.
For the design for bearings supported by beam ledges, the provisions of Article S5.7.5 apply.

10.3.2 Footings

While the provisions of Article S5.13.3 related to footings apply to isolated footings supporting a single column or wall, most of the provisions are generally applicable to combined footings and mats supporting several columns or walls or a combination thereof.

In sloped or stepped footings, the angle of slope or depth and location of steps are to be such that design requirements are satisfied at every section.

Circular or regular polygon-shaped concrete columns or piers may be treated as square members with the same area, for location of critical sections for moment, shear and development of reinforcement in footings.

The resistance of foundation material for piles shall be as specified in Section 10.

Where an isolated footing supports a column, pier or wall, the footing shall be assumed to act as a cantilever. Where a footing supports more than one column, pier or wall, the footing shall be designed for the actual conditions of continuity and restraint.

For the design of footings, unless the use of special equipment is specified to assure precision driving of piles, it shall be assumed that individual driven piles may be out of planned position in a footing by either 150 mm or one-quarter of the pile diameter, and that the center of a group of piles may be 75 mm from its planned position. For pile bents, the contract documents may require a 50 mm tolerance for pile position, in which case that value should be accounted for in the design.

The assumption that the as-built location of piles may differ from the planned location recognizes the construction variations sometimes encountered and is consistent with the tolerances allowed by Division II. Lesser variations may be assumed if the contract documents require the use of special equipment, such as templates, needed for more precise driving.

For non-circular piles, the larger cross-sectional dimension should be used as the “diameter”.

The critical section for flexure is taken at the face of the column, pier or wall. For footings under masonry walls, the critical section shall be taken as halfway between the center and edge of the wall. For footings under metallic column bases, the critical
section shall be taken as halfway between the column face and the edge of the metallic base.

Moment at any section of a footing may be determined by passing a vertical plane through the footing and computing the moment of the forces acting on one side of that vertical plane. Note that for structural design of footing on rock or soil, the triangular or trapezoidal pressure distribution is used, as appropriate. This is the case even though a rectangular distribution of soil pressure is used to determine bearing resistance.

In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across the entire width of the footing.

The following guidelines apply to the distribution of reinforcement in two-way rectangular footings:

- in the long direction, reinforcement shall be distributed uniformly across the entire width of footing, and
- in the short direction, a portion of the total reinforcement as specified by Equation S5.13.3.5-1, shall be distributed uniformly over a band width equal to the length of the short side of footing and centered on centerline of column or pier. The remainder of reinforcement required in the short direction shall be distributed uniformly outside of the center band width of footing. The area of steel in the band width shall satisfy Equation 1.

\[
\frac{A_{s-BW}}{A_{s-SD}} = \frac{2}{\beta \%1}
\]  

(S5.13.3.5-1)

where:

\[ \beta \] = the ratio of long side to short side of footing

\[ A_{s-BW} \] = area of steel in the band width (mm²)

\[ A_{s-SD} \] = total area of steel in short direction (mm²)

In determining the shear resistance of slabs and footings in the vicinity of concentrated loads or reaction forces, the more critical of the following conditions shall govern:

- one-way action, with a critical section extending in a plane across the entire width and located at a distance, "d", from the face of the concentrated load or reaction area, or from any abrupt change in slab thickness,
two-way action, with a critical section perpendicular to the plane of the slab and located so that its perimeter, $b_o$, is a minimum, but not closer than $0.5d$ to the perimeter of the concentrated load or reaction area, or

where the slab thickness is not constant, critical sections located at a distance not closer than $0.5d$ from the face of any change in the slab thickness and located such that the perimeter, $b_o$, is a minimum.

If a haunch has a rise-to-span ratio of 1:1 or more where the rise is in the direction of the shear force under investigation, it may be considered an abrupt change in section and the design section may be taken as "$d" into the span with "$d" taken as the depth past the haunch.

Where a portion of a pile lies inside the critical section, the pile load shall be considered to be uniformly distributed across the width or diameter of the pile, and that portion of the load which is outside the critical section shall be included in the calculation of shear on the critical section.

If a large diameter pile is subjected to significant flexural moments, the load on the critical section may be adjusted by considering the pile reaction on the footing to be idealized as the stress distribution resulting from the axial load and moment.

All forces and moments applied at the base of a column or pier shall be transferred to the top of footing by bearing on concrete and by reinforcement. Bearing on concrete at the contact surface between the supporting and supported member shall not exceed the concrete bearing strength, as specified in Article S5.7.5, for either surface.

Lateral forces shall be transferred from the pier to the footing in accordance with shear-transfer provisions specified in Article S5.8.4.

Reinforcement shall be provided across the interface between supporting and supported member, either by extending main longitudinal column or wall reinforcement into footings, or by the use of dowels or anchor bolts.

Reinforcement across the interface shall satisfy the following:

- all force effects that exceed the concrete bearing strength in the supporting or supported member shall be transferred by reinforcement,
• if load combinations result in uplift, the total tensile force shall be resisted by the reinforcement, and

• the area of reinforcement shall not be less than 0.5% of the gross area of the supported member, and the number of bars shall not be less than four.

10.3.3 Piles

All loads resisted by the footing, and the weight of the footing itself, is assumed to be transmitted to the piles. The material directly under a pile supported footing is not assumed to carry any of the applied loads. Piles installed by driving are designed to resist driving and handling forces. For transportation and erection, a precast pile should be designed for not less than 1.5 times its self-weight.

Any portion of a pile, where lateral support adequate to prevent buckling may not exist at all times, must be designed as a column. Locations where such lateral support does not exist include any portion of a pile above the anticipated level of scour or future excavation, as well as portions which extend above ground, as in pile bents.

The points or zones of fixity for resistance to lateral loads and moments must be determined by an analysis of the soil properties, as specified in Article S10.7.4.2.

Concrete piles must be embedded into footings or pile caps, as specified in Article S10.7.1.5. Anchorage reinforcement consists of either an extension of the pile reinforcement or the use of dowels. Uplift forces or stresses induced by flexure must be resisted by the reinforcement. The steel ratio for anchorage reinforcement must not be less than 0.005 and the number of bars must not be less than four. The reinforcement must be developed sufficiently to resist a force of $1.25 f_y A_s$.

In addition to the requirements specified in Articles S5.13.4.1 through S5.13.4.5, piles used in the seismic zones must conform to the requirements specified in Article 5.13.4.6.

Splices in concrete of piles must develop the axial, flexural, shear and torsional resistance of the pile. Details of splices must be shown in the contract documents.

Division II has provisions for short extensions or "build-ups" for the tops of concrete piles. This allows for field corrections due to unanticipated events, such as breakage of heads or driving slightly past the cut-off elevation.
Provisions related to pile dimensions and reinforcing steel details are provided for:

- precast reinforced piles,
- precast prestressed piles, and
- cast-in-place piles.

Seismic details are provided and are summarized below.

Piles for structures in Zone 2 may be used to resist both axial and lateral loads. The minimum depth of embedment and axial and lateral pile resistances required for seismic loads must be determined by means of design criteria established by site-specific geological and geotechnical investigations.

Concrete piles must be anchored to the pile footing or cap by either embedment of reinforcement or anchorages to develop uplift forces. The embedment length must not be less than the development length required for the reinforcement specified in Article S5.11.2.

Concrete-filled pipe piles must be anchored with steel dowels as specified in Article S5.13.4.1, with a minimum steel ratio of 0.01. Dowels must be embedded as required for concrete piles. Timber and steel piles, including unfilled pipe piles, must be provided with anchoring devices to develop any uplift forces. The uplift force must not be taken less than 10% of factored axial compressive resistance of the pile.

For cast-in-place piles, longitudinal steel must be provided in the upper end of the pile for a length not less than either one-third of the pile length or 2400 mm, with a minimum steel ratio of 0.005 provided by at least four bars. Spiral reinforcement or equivalent ties of not less than No. 10 bars must be provided at pitch not exceeding 225 mm, except that the pitch must not exceed 75 mm within a length not less than either 600 mm or 1.5 pile diameters below the pile cap reinforcement.

For precast reinforced piles, the longitudinal steel must not be less than 1.0% of the cross-sectional area, provided by not less than four bars. Spiral reinforcement or equivalent ties of not less than No. 10 bars must be provided at a pitch not exceeding 225 mm, except that a 75 mm pitch must be used within a confinement length not less than either 600 mm or 1.5 pile diameters below the pile cap reinforcement.

For precast prestressed piles, the ties must conform to the requirements of precast piles.
In addition to the requirements as specified for Zone 2, piles in Zones 3 and 4 must conform to the following additional provisions.

The upper end of every pile must be reinforced and confined as a potential plastic hinge region, except where it can be established that there is no possibility of any significant lateral deflection in the pile. The potential plastic hinge region must extend from the underside of the pile cap over a length of not less than either 2.0 pile diameters or 600 mm. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the confinement length with the specified transverse reinforcement and closer pitch, as specified for Zone 2, must extend thereto.

For cast-in-place piles, longitudinal steel must be provided for the full-length of the pile. In the upper two-thirds of the pile, the longitudinal steel ratio, provided by not less than four bars, must not be less than 0.75%. Spiral reinforcement or equivalent ties of not less than No. 10 bars must be provided at 225 mm pitch, except for the top length not less than either 1200 mm or two pile diameters, where the pitch must be 75 mm, and where the volumetric ratio and splice details must conform to Article S5.10.11.4.1d.

For precast piles, spiral ties must not be less than No. 10 bars at a pitch not exceeding 225 mm, except for the top 1200 mm where the pitch must be 75 mm and the volumetric ratio and splice details must conform to Article S5.10.11.4.1d.

10.3.4 Provisions for Structure Types

Article S5.14 is a collection of provisions relating to structure types. The structures included are beam and girder bridges, segmental construction, arches, slab superstructures and culverts. The provisions of this article supplement the general provisions of Section 5 as they relate to these particular structural systems.

Beam and Girder Bridges

Included in this structure type are bridges composed of precast and cast-in-place beams and girders.
LECTURE 11 - DECKS

11.1 OBJECTIVE OF THE LESSON

The objective of this lesson is to introduce the provisions for the analysis and design of bridge decks and deck systems of concrete, metal and wood. Design criteria and applicable limit states for different types of decks are also presented. Design examples for concrete decks are included.

11.2 GENERAL DESIGN REQUIREMENTS

11.2.1 Interface Action

Composite action between the deck and the supporting elements enhances stiffness, economy of structures, and prevents vertical separation between the decks and their supporting components. Some decks without shear connectors have historically demonstrated a degree of composite action due to chemical bond and/or friction. This unintended composite action could not be accounted for in structural design due to the uncertainty of the strength of the connection. Therefore, the Specification requires the decks, other than wood and open grid floors, to be made composite with their supporting components, unless there are compelling reasons to the contrary. Where provided, shear connectors and other connections between decks and their supporting members must be designed for force effects calculated on the basis of full composite action whether or not that composite action is considered in proportioning the primary members. This is required to ensure the integrity of the connection under all possible cases of loading.

Composite action between various components of a bridge can also control out-of-plane deformations which contribute to distortion-induced fatigue. For example, bridges with stringers (or other main longitudinal members) and floorbeams, in which the deck is not tied to the main longitudinal members, are often subject to fatigue cracking in tie plates or floorbeams. This is due to the relative movement of the deck and main members as shown in Figure 11.2.1-1.
If the deck is connected to the main members, usually girders, by a properly designed composite interface, then the relative movement can be significantly reduced.

11.2.2 Deck Drainage

Proper drainage of the deck minimizes the water damage that may not only affect the deck, but also may extend to the supporting components and the substructure. In addition, proper drainage of a bridge deck enhances the safety of the public crossing the bridge. With the exception of unfilled steel grid decks, deck surfaces are required to have cross and longitudinal slopes and an adequate drainage system. Based on past experience, expansion joint regions are most affected by poor deck drainage. Thus, special care should be given to the detailing of joint regions. The structural effects of drainage openings need to be considered in the design of decks.

11.2.3 Concrete Appurtenances

Attachments to the deck, such as concrete curbs, parapets, barriers and dividers, enhance the performance of the structure by contributing to its stiffness and strength. Joints in these attachments reduce this desired effect on the structural behavior. Therefore, the LRFD Specification requires that such appurtenances be made continuous, unless otherwise specified by the Owner. The effect of these attachments may be considered when the optional deflection criteria is invoked. However, since they may be damaged during vehicular collision, their contribution to the strength of the structure should not be considered in the design.

11.2.4 Edge Supports

Edge supports need to be provided along the edges of the deck, unless the deck is designed to support wheel loads in extreme positions with respect to its edges. The edge support may be an edge beam or an integral part of the deck. Along the expansion joints in the deck, the LRFD Specification allows the utilization of the joint
hardware as a structural element of the edge beam if it is integrated with the deck.

Where the primary direction of a concrete deck is transverse and/or the deck is composite with a structurally continuous concrete barrier, no additional longitudinal edge beam need be provided.

11.2.5 Stay-in-Place Formwork for Overhangs

Due to the possibility of the separation between the stay-in-place forms and the cast-in-place layer of a concrete deck, stay-in-place formwork should not be used in the overhang of concrete decks.

11.3 LIMIT STATES

11.3.1 Service Limit State

Stresses at the service limit state are usually within the elastic limit of common construction materials. Therefore, fully elastic behavior is to be assumed in the analysis of decks and deck systems at service limit state. The design and details should satisfy provisions specified for the particular deck material.

The effects of excessive deck deformation, in the form of local dishing at wheel loads, should be considered in the design of decks other than concrete and concrete-filled steel decks. Limiting these deformations will reduce or prevent the break-up and loss of the wearing surface. The Specification did not specify an overall limit for such deformation. The limit should be established by testing, as it is a function of the composition of the wearing surface and the adhesion between the deck and the wearing surface which may vary based on the materials used.

11.3.2 Fatigue and Fracture Limit State

Steel grid and steel orthotropic decks, aluminum decks and concrete decks, other than those in multi-girder applications, are required to be investigated for the fatigue limit state as indicated by the Specification for the respective material.

Fatigue need not be investigated for:

• concrete decks and fully-filled grid decks in multi-girder applications,
• the filled portion of partially-filled grid decks, and
• wood decks.

The decks mentioned above can be excluded from fatigue investigation based on observed performance and laboratory testing. The results from a series of pulsating load fatigue tests conducted on
35 model slabs indicated that the fatigue limit for the slabs designed by the conventional AASHTO moment methods was approximately three times the service level. Decks designed using the isotropic reinforcement method, as described later, had fatigue limits of approximately twice the service level.

### 11.3.3 Strength Limit States

At strength limit states, decks and deck systems may be analyzed as either elastic or inelastic structures. The design and details are required to satisfy the Specification provisions for the deck material.

### 11.3.4 Extreme Event Limit States

During a vehicular collision with bridge railings, forces are transmitted from the railing system to the deck. It is important to ensure that collision damage will be limited to the railing system which can be repaired relatively easily. No damage should extend to the deck slab. Therefore, the regions of the deck adjacent to the railings should be designed to resist the effect of collision forces equal to or greater than the resistance of the railing system itself. This causes the railing to be the weaker element. Section 13 of the Specification provides analytical procedures to be used in the analysis of the railing systems and deck slabs under the effect of collision forces. Acceptance testing, complying with the Specification requirements, may be used to satisfy this provision.

### 11.4 ANALYSIS

#### 11.4.1 Approximate Methods of Analysis

For decks other than fully-filled and partially-filled grids, the Specification allows an approximate method of deck analysis in which the deck is subdivided into strips perpendicular to the supporting components. The strips are treated as continuous beams or simply supported beams, as appropriate. Span length is taken as the center-to-center distance between the supporting components. For the purpose of determining force effects in the strip, the supporting components are assumed to be infinitely rigid.

For decks spanning primarily in a direction perpendicular to the traffic, the rear axle of the design truck is moved laterally on the strip to produce positive and negative moment envelopes for all deck panels. One or more design lanes may be assumed to be loaded simultaneously. Within each design lane, the truck axle can be moved laterally, such that the center of the wheels is no closer than 600 mm to the edge of the lane. The location of the design lanes, relative to the longitudinal axes of the deck, can be shifted laterally to produce maximum force effects. Both the dynamic load allowance and the multiple presence factor for live loads corresponding to the number of loaded lanes are applied to the force effects.
For decks spanning primarily in the direction of the traffic, the design truck should be moved along the strip to produce maximum load effects. The multiple presence factors should be applied based on the number of loaded lanes. If the distance between the supporting components of concrete slabs and slab bridges which span primarily in the direction parallel to traffic is more than 4600 mm, the strip width equations provided for use with slab-type bridges will be applicable.

The location of the design section for negative moments and shear forces varies based on the type of supporting components and may be taken as follows:

- for monolithic construction and concrete box beams - at the face of the supporting component,
- for steel and wood beams - one-quarter the flange width from the centerline of support,
- for precast I-shaped and T-shaped concrete beams - one-third the flange width, but not exceeding 380 mm from the centerline of support.

In case of steel and concrete box beams, each web may be treated as a separate supporting component. In this case, the span of the deck should be taken equal to the largest distance between two adjacent webs either from one box or from two adjacent boxes.

In developing the strip width equations for concrete decks spanning primarily in a direction perpendicular to traffic, several simple and two-span bridge superstructures with different number of girders, different girders stiffness and spacings, different span length and different slab thickness were analyzed as three-dimensional structures using finite element analysis. The strip width equations were developed such that the extreme maximum moment per unit width of the strip is as close as possible to that obtained from the three-dimensional models. The assumed stiffness of the girders were selected to represent steel and concrete girders.

The results obtained from the three-dimensional models indicated that maximum positive moments occur at a point near the center of the span of the supporting components while maximum negative moments occur near their intermediate supports. The maximum negative moments in the deck were smaller when the deck was assumed to be in contact with the diaphragms at the intermediate supports of the supporting girders. Therefore, to be conservative, the strip width equations were developed for the case of no contact between the deck and the diaphragms, except at end supports. The results of the three-dimensional study, which included flexible girders, also indicated that the differences in the maximum positive moment in several bays in a cross-section, is much smaller than the differences obtained assuming rigid supports. Thus, where the strip width method is used, the extreme positive moment in any deck panel (bay) between girders is considered to apply to all positive moment regions.
Similarly, the extreme negative moment over any beam or girder is considered to apply to all negative moment regions. Since a three-dimensional analysis was used to develop the strip width equations, the effects of flexure in the secondary direction, and of torsion, on the distribution of internal force effects are accounted for when using the strip width method.

One drawback of this method is that it is based on rectangular layouts. Currently, about two-thirds of all bridges nationwide are skewed. While skew generally tends to decrease extreme force effects, it produces negative moments at corners, torsional moments in the end zones, substantial redistribution of reaction forces, and a number of other structural phenomena which should be considered in the design. Another drawback is that the effect of the girder flanges on the stiffness of the deck transverse strips was ignored in the development of the strip width equations. Including this effect was not practical due to the wide variation in the width and stiffness of the girder flanges. In addition, due to the wide variation in the distance from the centerline of the supporting girders to the design section for negative moments, the strip width equations were calibrated to the results obtained at the center of the supporting girders of the three-dimensional models.

The range of maximum design moments per unit width obtained from different superstructures with 1800, 2700 and 3600 mm girder spacings that was analyzed in the three-dimensional study is compared to the results of both the Standard Specification and the LRFD Specification in Figures 11.4.1-1 and 11.4.1-2. The listed moments include all applicable load factors, multiple presence factors and dynamic load allowance. As shown in Figure 11.4.1-1, both Specifications tend to give conservative values for the design positive moments when compared to the three dimensional analysis. Both Specifications give almost the same design moments for bridges having 1800 mm girder spacing. When the girder spacing was increased to 3600 mm, the LRFD design positive moment was about 10% less than that of the Standard Specification.

The comparison of negative moments is rather difficult. Both the three-dimensional results and the LRFD results are given at the center of the supporting girders while the moments based on the Standard Specification are those at the design section for negative moments that was assumed to be 100 mm from the center of the girders. Figure 11.4.1-2 shows a comparison of the results for different girder spacings. It appears that if all moments were calculated for the design sections, results obtained using the Standard Specification would appear more conservative compared to other results especially for narrow girder spacings and wider flange widths.

Depending on the type of deck, the modeling and design in the secondary direction may utilize one of the following approximations:

• the secondary strip may be designed in the same manner as the primary strip, with all the limit states applicable,
• the resistance requirements in the secondary direction may be determined as a percentage of that in the primary one, i.e., the traditional approach for reinforced concrete slabs,

• the minimum structural and/or geometry requirements may be specified for the secondary direction independent of actual force effects, as is the case of most wood decks.

Figure 11.4.1-1 - Comparison Between Current AASHTO, LRFD, and the 3-D Design Positive Moments

Figure 11.4.1-2 - Comparison Between Standard Specification, LRFD Specification, and the 3-D Design Negative Moments
The values provided by the Specification for equivalent strip widths and strength requirements in the secondary direction are based on past experience. Practical experience and future research work may lead to refinement.

For decks containing prefabricated elements, the Specification permits the use of design aids in lieu of analysis if the performance of the deck is documented and supported by sufficient technical evidence. The designer should exercise extreme caution in the interpretation of the applicability of such design aids and their accuracy.

Table 11.4.1-1 lists the width of the equivalent strip for some types of decks and, for other types, refers to the applicable articles of the Specification. Where decks span primarily in the direction parallel to traffic, the Specification requires that the strips supporting a line of wheels shall not be taken greater than 500 mm for open grids, and not greater than 1800 mm for all other decks where multi-lane loading is being investigated. There are no width limits specified for equivalent strips for decks which span primarily in the transverse direction. For deck overhangs, the live load effects may be calculated either by considering a wheel load placed at 300 mm from the face of the railing in conjunction with the specified strip width or by considering a line load of 14.6 N/mm at 300 mm from the face of the railing. Some limits to the applicability of the line loads need to be satisfied as required by Article S3.6.1.3.4. The following notation shall apply to Table 11.4.1-1:

\[
\begin{align*}
S & = \text{spacing of supporting components (mm)} \\
h & = \text{depth of deck (mm)} \\
L & = \text{span length of deck (mm)} \\
P & = \text{axle load (N)} \\
S_b & = \text{spacing of grid bars (mm)} \\
+M & = \text{positive moment} \\
-M & = \text{negative moment} \\
X & = \text{distance from load to point of support (mm)}
\end{align*}
\]
Table 11.4.1-1 - Equivalent Strip Width

<table>
<thead>
<tr>
<th>TYPE OF DECK</th>
<th>DIRECTION OF PRIMARY STRIP RELATIVE TO TRAFFIC</th>
<th>WIDTH OF PRIMARY STRIP (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Cast-in-place</td>
<td>Overhang</td>
<td>1140 + 0.833X</td>
</tr>
<tr>
<td></td>
<td>Either Parallel or Perpendicular</td>
<td>+M: 660 + 0.55S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>−M: 1220 + 0.25S</td>
</tr>
<tr>
<td>- Cast-in-place with stay-in-place concrete formwork</td>
<td>Either Parallel or Perpendicular</td>
<td>+M: 660 + 0.55S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>−M: 1220 + 0.25S</td>
</tr>
<tr>
<td>- Precast, post-tensioned</td>
<td>Either Parallel or Perpendicular</td>
<td>+M: 660 + 0.55S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>−M: 1220 + 0.25S</td>
</tr>
<tr>
<td>Steel:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Open grid</td>
<td>Main Bars</td>
<td>0.007P + 4.0Sb</td>
</tr>
<tr>
<td>- Filled or partially filled grid</td>
<td>Main Bars</td>
<td>Article 4.6.2.1.8 applies</td>
</tr>
<tr>
<td>- Unfilled, composite grids</td>
<td>Main Bars</td>
<td>Article 9.8.2.4 applies</td>
</tr>
<tr>
<td>Wood:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Prefabricated glulam</td>
<td>Parallel</td>
<td>2.0h + 760</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>2.0h + 1020</td>
</tr>
<tr>
<td></td>
<td>- Interconnected</td>
<td>2280 + 0.07L</td>
</tr>
<tr>
<td></td>
<td>Parallel</td>
<td>4.0h + 760</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>4.0h + 1020</td>
</tr>
<tr>
<td>- Stress-laminated</td>
<td>Parallel</td>
<td>0.066S + 2740</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>0.84S + 610</td>
</tr>
<tr>
<td>- Spike-laminated</td>
<td>Continuous decks or interconnected panels</td>
<td>Parallel</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>4.0h + 1020</td>
</tr>
<tr>
<td></td>
<td>- Non-interconnected panels</td>
<td>Parallel</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>4.0h + 1020</td>
</tr>
<tr>
<td></td>
<td>- Non-interconnected panels</td>
<td>Parallel</td>
</tr>
<tr>
<td></td>
<td>Perpendicular</td>
<td>4.0h + 1020</td>
</tr>
<tr>
<td>- Planks</td>
<td>Plank Width</td>
<td></td>
</tr>
</tbody>
</table>
In addition to the distribution in the primary direction, the Specification includes provisions for distributing the wheel loads in both the primary and the secondary directions in case of slab panels with length-to-width ratio less than 1.5 as presented in Article S4.6.2.1.5. Provisions for proportioning and designing the edge supports of deck slabs are also provided in Article S4.6.2.1.4.

To assist bridge designers in determining the design moments for different girder arrangements, a series of bridge decks was analyzed using the equivalent strip method. Maximum positive and negative live load moments per unit width for girder spacings ranging from 1300 mm to 4600 mm are listed in Table 11.4.1-2. Multiple presence factors and the dynamic load allowance are included in these moments. To cover most possible design cases, the results are listed for design sections for negative moments located at 75, 150, 225, 300, 450 and 600 mm from the centerline of supporting girders.

The table lists the maximum moments for structures with a clear overhang length measured from the centerline of the exterior girder to the inside face of the parapet equal to or less than:

\[(\text{the smaller of } 0.625 \times \text{girder spacing and } 1800 \text{ mm}) - 530 \text{ mm}\]

The listed moments are applicable for decks supported on at least three girders and having a width measured between the centerlines of the exterior girders of not less than 4200 mm. For location of the design sections for negative moments see Article S4.6.2.1.6. The above moments do not apply to the deck overhangs and the adjacent regions of the deck which need to be designed according to Article SA13.4.1.

A railing system width of 530 mm was used to determine the clear overhang width. For other widths of railing systems, the difference in the moments in the interior regions of the deck is expected to be within the acceptable limits for practical design.

Another method of design of concrete decks, called the Empirical Method, is presented in Section S9.7.2. In this method, subject to certain specified conditions, the deck is considered to satisfy all design provisions if the specified percentage of reinforcement is provided. No further analysis is required. A design example using the Empirical Method is presented later in this lecture.

In addition to the above methods, the Specification allows the use of yield line analysis if permitted by the bridge owner.
<table>
<thead>
<tr>
<th>S mm</th>
<th>Positive Distance from CL of Girder to Design Section for Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0 mm</td>
</tr>
<tr>
<td>1300</td>
<td>21130</td>
</tr>
<tr>
<td>1400</td>
<td>21010</td>
</tr>
<tr>
<td>1500</td>
<td>21050</td>
</tr>
<tr>
<td>1600</td>
<td>21190</td>
</tr>
<tr>
<td>1700</td>
<td>21440</td>
</tr>
<tr>
<td>1800</td>
<td>21790</td>
</tr>
<tr>
<td>1900</td>
<td>22240</td>
</tr>
<tr>
<td>2000</td>
<td>22780</td>
</tr>
<tr>
<td>2100</td>
<td>23380</td>
</tr>
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<td>24040</td>
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<tr>
<td>2300</td>
<td>24750</td>
</tr>
<tr>
<td>2400</td>
<td>25500</td>
</tr>
<tr>
<td>2500</td>
<td>26310</td>
</tr>
<tr>
<td>2600</td>
<td>27220</td>
</tr>
<tr>
<td>2700</td>
<td>28120</td>
</tr>
<tr>
<td>2800</td>
<td>29020</td>
</tr>
<tr>
<td>2900</td>
<td>29910</td>
</tr>
<tr>
<td>3000</td>
<td>30800</td>
</tr>
<tr>
<td>3100</td>
<td>31660</td>
</tr>
<tr>
<td>3200</td>
<td>32500</td>
</tr>
<tr>
<td>3300</td>
<td>33360</td>
</tr>
<tr>
<td>3400</td>
<td>34210</td>
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<td>3500</td>
<td>35050</td>
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<td>3700</td>
<td>36670</td>
</tr>
<tr>
<td>3800</td>
<td>37450</td>
</tr>
<tr>
<td>3900</td>
<td>38230</td>
</tr>
<tr>
<td>4000</td>
<td>38970</td>
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<tr>
<td>4100</td>
<td>39710</td>
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<tr>
<td>4200</td>
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<tr>
<td>4300</td>
<td>41120</td>
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<tr>
<td>4400</td>
<td>41800</td>
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<tr>
<td>4500</td>
<td>42460</td>
</tr>
<tr>
<td>4600</td>
<td>43110</td>
</tr>
</tbody>
</table>
11.4.2 Refined Methods of Analysis

Refined methods of deck slab analysis are permitted by the Specification. Some conditions need to be met to accurately model the behavior of deck slabs as observed in laboratory tests and in actual bridges. For example, in many solid decks, the wheel load carrying contribution of torsion is comparable to that of flexure. This effect is more significant in the end zones of skewed girder bridges due to differential deflection. Thus, flexural and torsional deformation of the deck need to be considered in the analysis. In-plane shear deformations, which, for instance, gave rise to the concept of effective width for composite bridge decks, should also be considered. On the other hand, vertical shear deformation may usually be neglected since their contribution to vertical deflection is not significant in most practical cases. Other conditions that will lead to more accurate results include:

• Locations of flexural discontinuity through which shear may be transmitted should be modeled as hinges.

• If the deck may crack and/or separate along element boundaries when loaded, Poisson’s ratio may be neglected.

• The wheel loads shall be modeled as patch loads distributed over the tire contact area specified by the Specification, extended by half of the deck depth on all four sides. This will produce significantly smaller, and more realistic, moments, especially in the case of decks supported on closely spaced components.

The Specification allows the use of the isotropic plate model to analyze bridge decks which are solid, have uniform or close to uniform depth, and whose stiffness is close to equal in all in-plane directions. This model is relatively insensitive to small variations in depth, such as those produced by superelevation, crown and haunches. In slightly cracked concrete slabs, even a large difference in the reinforcement ratio will not cause significant changes in load distribution.

In case of driving surfaces that are composite with or integrated into their supporting elements, the Specification allows the use of the orthotropic plate model. For simplicity, the flexural rigidity of the elements may be uniformly distributed along the cross-section of the deck. However, this may lead to inaccurate results in the case of systems consisting of small number of elements subjected to concentrated loads. Where the torsional stiffness of the deck is not contributed solely by a solid plate of uniform thickness, the torsional rigidity should be established by physical testing, three-dimensional analysis, or generally accepted and verified approximations.

11.4.3 Analysis of Cantilever Slabs

The following three design cases must be checked separately when designing concrete deck overhangs for the limit states indicated:

Lecture - 11-12
Design Case 1: the transverse and longitudinal forces specified in Article SA13.2 - extreme event limit state

Design Case 2: the vertical forces specified in Article SA13.2 - extreme event limit state

Design Case 3: the loads specified in Article S3.6.1 which occupy the overhang - strength limit state

To design deck overhangs for vehicular collision (Design Cases 1 and 2), crash testing is required. Analytical design procedures for Design Cases 1 and 2 for overhangs supporting concrete or post-type railings are presented in Specification Section 13 and in Lecture 17. These procedures may be used for preliminary design of new railing systems or to verify the capacity of systems that are slightly different from systems that were crash-tested.

A resistance factor $\varphi = 1.0$ is required by the Specification for the extreme event limit (S1.3.2.1).

11.5 DESIGN OF CONCRETE DECK SLABS

11.5.1 General Design Requirements

- Minimum Depth: The LRFD Specification requires that the depth of a concrete deck, excluding any provision for grinding, grooving and sacrificial surface, should not be less than 175 mm. Traditionally, some jurisdictions also apply the minimum thickness required by the optional deflection control of slab-type superstructures to concrete decks. Unless a lesser thickness can be proven satisfactory during the crash testing procedure, the minimum edge thickness for concrete deck overhangs should be taken as:

  For deck overhangs supporting a deck-mounted post system: 200 mm

  For deck overhangs supporting a side-mounted post system: 300 mm

  For deck overhangs supporting concrete parapets or barriers: 200 mm

  These thickness limits are applicable for cast-in-place and precast reinforced or prestressed decks.

  For slabs with a design span-to-depth ratio greater than 20, consideration should be given to prestressing in the direction of the span in order to control cracking. In the case of thin
slabs, tighter control on construction tolerances is required or a relatively large variation in the deck strength will be obtained.

- **Minimum Cover**: Concrete cover requirements are listed in Article S5.12.3. These requirements are based on traditional concrete mixes and without protective coating on either the concrete or the steel. A combination of special mix design, protective coatings, dry or moderate climate and the absence of corrosive chemicals may justify a reduction of these requirements, meeting the approval of the Owner.

For most decks, the Specification requires the top cover of a concrete deck to be 60 mm, assuming the deck is exposed to deicing salts and tire studs. The bottom cover is required to be 25 mm. The required cover thickness is then modified by a correction factor of 0.8 for concrete mixes with water/cement (w/c) ratio \( \#0.4 \) or a factor of 1.2 for w/c ratio \( \#0.5 \). Based on the site conditions, the cover requirements may differ from those listed above.

- **Skewed Decks**: To prevent extensive cracking of the deck, the reinforcement should be placed in a direction close to that of the principal flexural stresses. Thus, if the skew angle of the deck does not exceed 25\( ^\circ \), the primary reinforcement may be placed in the direction of the skew, otherwise, it should be placed perpendicular to the main supporting components. The 25\( ^\circ \) criterion is an arbitrary limit that was thought to provide adequate reinforcement in the direction of principal flexural stresses.

![Figure 11.5.1-1 - Reinforcement Layout](image)

### 11.5.2 Design of Stay-in-Place Formwork

The design of stay-in-place formwork should prevent excessive sagging of the formwork during construction, which would result in an unanticipated increase in the weight of the concrete slab. It is necessary to ensure adequate cover for reinforcing steel and to account for all dead load in the design. Therefore, it has been common to limit the deflection to the form span divided by 240, but not to exceed 20 mm. The Specification require that:

- Stay-in-place formwork be designed to be elastic under construction loads.
• Minimum construction loads to be considered in the analysis is the weight of the form, the concrete slab, plus $2.4 \times 10^{-3}$ MPa.

• Flexural stresses due to unfactored construction loads do not exceed the smaller of 75% of the yield strength of steel, or 65% of the 28-day compressive strength for concrete in compression, or the modulus of rupture in tension for prestressed concrete form panels.

If steel stay-in-place formwork is used, panels should be mechanically tied together at their common edges and fastened to their support. Steel formwork should not be welded to the supporting components unless otherwise shown in the contract documents. Due to the uncertainty of the strength of the adhesion between the formwork and the concrete, steel formwork should not be considered to be composite with a concrete slab.

If concrete stay-in-place formwork is used, the upper surface of the panels shall be specified to be roughened in such a manner as to ensure composite action with the cast-in-place concrete. The depth of the stay-in-place concrete should neither exceed 55% of the depth of the finished deck slab nor be less than 90 mm. This arbitrary limit was based on past experience with this type of construction. Thousands of bridges have successfully been built with a depth ratio of 43% or somewhat higher; 55% is believed to be a practical limit beyond which cracking of the cast-in-place concrete at the panel interface may be expected.

Concrete stay-in-place formwork panels may be prestressed in the direction of the design span. In this case, the strands may be considered as primary reinforcement in the deck slab and the concrete cover below the strands should not be less than 20 mm. The transfer and development lengths of the strands shall be investigated for conditions during construction and in service. Prestressing strands and/or reinforcing bars in the precast panel need not be extended into the cast-in-place concrete above the supporting beams. The age of the panel concrete at the time of placing the cast-in-place concrete should be such that the difference between the combined shrinkage and creep of the precast panel and the shrinkage of the cast-in-place concrete is minimized.

To minimize the reflective cracking in the cast-in-place concrete layer, the ends of the formwork panels is required to be supported on a continuous mortar bed, or to be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding.
11.5.3 Provisions for Precast Deck Slabs

The Specification allows the use of flexurally discontinuous decks made from precast panels. The panels may be joined together by transverse shear keys or longitudinal post tensioning. The provisions for both type of connections are presented in Article S9.7.5.

Precast decks that form the top slabs of post-tensioned girders whose cross-sections consist of single or multi-cell boxes are also allowed. Provisions of these decks are presented in Article S9.7.6.

11.5.4 Cast-In-Place Concrete Deck Design Example - Conventional Design

To facilitate the identification of the applicable articles of the Specification, where possible, the article numbers appear in parenthesis. Where engineering judgment was exercised, the rational of using certain design steps was also explained.

ASSUMPTIONS

Figure 11.5.4-1 - Cross-Section

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder spacing</td>
<td>2500 mm</td>
</tr>
<tr>
<td>Number of girders</td>
<td>5</td>
</tr>
<tr>
<td>Top cover (includes 10 mm integral</td>
<td>60 mm</td>
</tr>
<tr>
<td>wearing surface)</td>
<td></td>
</tr>
<tr>
<td>Bottom cover</td>
<td>25 mm</td>
</tr>
<tr>
<td>Steel yield strength</td>
<td>400 MPa</td>
</tr>
<tr>
<td>Concrete compressive strength</td>
<td>28 MPa</td>
</tr>
<tr>
<td>Concrete density</td>
<td>2400 kg/m³</td>
</tr>
<tr>
<td>Future wearing surface - Density</td>
<td>146 kg/m²</td>
</tr>
<tr>
<td>Assumed distance from centerline</td>
<td></td>
</tr>
<tr>
<td>of girder to design section for</td>
<td></td>
</tr>
<tr>
<td>negative moment</td>
<td>75 mm (S4.6.2.1.6)</td>
</tr>
<tr>
<td>Minimum slab thickness</td>
<td>Larger of (S + 3000)/30</td>
</tr>
<tr>
<td></td>
<td>$165 mm (S2.5.2.6.3) or</td>
</tr>
<tr>
<td></td>
<td>175 mm (S9.7.1.1) or</td>
</tr>
<tr>
<td>Minimum overhang thickness</td>
<td>200 mm (S13.7.3.1.2)</td>
</tr>
<tr>
<td></td>
<td>assuming concrete parapet</td>
</tr>
<tr>
<td>Assume Slab thickness</td>
<td>200 mm</td>
</tr>
</tbody>
</table>
(includes 10 mm integral wearing
surface)
Assume overhang total thickness  = 220 mm

Concrete Parapet:

- Mass per unit length  = 946 kg/m
- Width at base       = 510 mm
- Moment capacity at base, $M_c = 102,000$ N@m/mm
- Parapet height, $H$  = 1060 mm
- Length of parapet failure mechanism, $L_c$  = 4142 mm
- Collision load capacity, $R_w$  = 654,000 N

The listed parapet properties are those of the parapet designed in Lecture 17 of this course. The load capacity of this parapet exceeds the minimum required by the Specifications. The deck overhang region is required to be designed to have a resistance larger than the actual resistance of the concrete parapet.

**EQUIVALENT STRIP METHOD (S4.6.2)**

Moments are calculated for a deck transverse strip assuming rigid supports at web centerlines. Use same reinforcement in all deck bays.

**Design Dead load moments:**

Load factors (S3.4.1):

- Slab and Parapet:
  - Minimum = 0.9
  - Maximum = 1.25

- Future Wearing Surface:
  - Minimum = 0.65
  - Maximum = 1.5

Dead load moments for the self-weight of the deck, parapet and future wearing service are listed in Table 11.5.4-1.

**Live load effects:**

Minimum distance from center of wheel to the inside face of parapet = 300 mm (S3.6.1.3)

Minimum distance between the wheels of two adjacent trucks = 1200 mm

Dynamic load allowance = 33% (S3.6.2.1)
Load factor (Strength I) = 1.75 (S3.4.1)

Multiple presence factor:  (S3.6.1.1.2)

- Single truck = 1.20
- Two trucks = 1.00
- Three trucks = 0.85

(Note: three trucks never control for girder spacings up to 4800 mm)

Trucks were moved laterally to determine extreme moments (S4.6.2.1.6)

Fatigue: Need not be investigated for concrete slabs in multi-girder bridges (S9.5.3 and S5.5.3.1)

Resistance factors for moment:

- 0.9 strength limit state (S5.5.4.2)
- 1.0 extreme event limit state (S1.3.2.1)

Assume distance from centerline of girder to design section of negative moment = 75 mm (S4.6.2.1.6)

**Determination of design moments:**

Dead load moments for tenth points in each bay were computed by conventional continuous beam analysis of a strip of unit width. The results are given in Table 11.5.4-1.

Live load positive and negative moments can be obtained using one of the following two methods:

**Method I:** Moments per unit width, including dynamic load allowance and multiple presence factors, can be directly obtained from Table 11.4.1-2. The values in this table are the upper bound for the moments calculated using the equivalent strip method in the analysis of bridges with different number of girders and different overhang length. Based on the geometry of any particular bridge, the values from this Table 11.4.1-2 may be slightly higher than those obtained from a complete analysis of the bridge being designed.

**Method II:** Complete analysis using the equivalent strip method outlined in Article S4.6.2.1.

For this example, the design moments will be calculated using both methods. The Method II maximum total live load moments on the design strip for the bridge deck being designed are listed in Table 11.5.4-2.
Design for positive moment in the deck:

Method I:

a. Live Load

   Maximum factored positive moment per unit width based on Table 11.4.1-2 = 1.75 x 26 310 = 46 043 N.mm/mm

   This moment is applicable to all positive moment regions, say from 0.4S to 0.6S, in all bays of the deck (S4.6.2.1.1).

   By inspection, the extreme dead load positive factored moment in the positive moment region occurs at 0.4S in the second bay.

b. Dead Load

   Maximum factored dead load moments in positive moment regions based on Table 11.5.4-1:

   Deck weight:
   
   1.25 x 1261 = 1576 N@m/mm

   Parapet:
   
   1.25 x 1122 = 1403 N@m/mm

   FWS:
   
   1.5 x 289 = 433 N@m/mm

   Dead load + Live load design factored moment = 46 043+1576+1403+433 = 49 455 N@m/mm

   Note that moments are dominated by live load. The location of maximum design positive moment may vary depending on overhang length and value and distribution of dead load.

   Use minimum load factors for permanent components that produce moments with the opposite sign. Do not include the effect of the future wearing surface if it produces moments that has the opposite sign.

Method II:

a. Live Load

   Width of equivalent strip for positive moment (S4.6.2.1.3) = 660 + 0.55S = 2035 mm
b. Dead Load

Maximum factored live load moment, based on Table 11.5.4-2, occurs at 0.4S in the first bay = 1.75 x 1.33 x 40.21 x 10^6/2035 = 45 989 N@m/mm

Dead load moments are the same as calculated for Method I

Dead load + Live load design factored moment = 45 989+1576+1403+433 = 49 401 N@m/mm

It is clear that the difference between Method I and Method II is insignificant (0.1%) and using either method should not lead to any difference in the required reinforcement. The rest of the analysis for positive moment is conducted using the results obtained from Method II.

Resistance factor for flexure $\phi = 0.90$ (S5.5.4.2.1)

d_e = total thickness - bottom cover - 1/2 bar diameter - top integral wearing surface

Assume #15 bars

Bar diameter = 16 mm

Bar cross-sectional area = 200 mm^2

d_e = 200 - 25 - 16/2 - 10 = 157

$k' = M_u/\phi b d_e^2 = 49 401/(0.9 x 1.0 x 157^2) = 2.227$ N/mm^2

$\rho = 0.85 \left( \frac{f_c}{f_y} \right) \left[ 1.06 - \frac{(2k')}{(0.85 f_c^2)} \right]^{-0.00586}$

Required $A_s = \rho d_e = 0.00586 x 157 = 0.92$ mm^2/mm

Required bar spacing = 200/0.92 = 218 mm

Use #15 bars at 210 mm

Check depth of compression block:

$T = 200$ mm^2 x 400 MPa = 80 000 N

$a = \frac{80 000}{0.85 x 28 x 210} = 16$ mm

$\beta_1 = 0.85$ for $f_c' = 28$ MPa (S5.7.2.2)
c = 16.0/0.85 = 18.8 mm

\[ \frac{c}{d_e} = \frac{18.8}{157} = 0.12 < 0.42 \text{ (S5.7.3.3.1)} \]

Check for cracking under Service I Limit State (S5.7.3.4)

Allowable reinforcement service load stress for crack control:

\[ f_{sa}' = \frac{Z}{(d_c A)^3} \times (0.6 f_y' - 240 \text{ MPa}) \]

![Diagram of bottom transverse reinforcement](Figure 11.5.4-2 - Bottom Transverse Reinforcement)

\[ d_c = 33 \text{ mm} < (50 + \frac{1}{2} \text{ bar diameter}) \text{ mm} \quad \text{OK} \]

\[ A = 2 \times 33 \times 210 = 13860 \text{ mm}^2 \]

\[ Z = 23000 \text{ N/mm} \quad \text{severe exposure conditions} \]

\[ f_{sa} = 299 \text{ MPa} > 240 \text{ MPa} \]

Maximum allowable \( f_{sa} = 240 \text{ MPa} \)

**Stresses under service loads (S5.7.1):**

In calculating the transformed compression steel area, the Specification requires the use of two different values for the modular ratio when calculating the service load stresses caused by dead and live loads, \( 2n \) and \( n \), respectively. For deck design, it is customary to ignore the compression steel in the calculation of service load stresses.

Modular ratio for 28 MPa concrete = \( n = 8 \)

Assume stresses and strains vary linearly

By inspection, maximum dead load + live load service load stresses occur at 0.4S in second bay.

Dead load service load moment = 1261 + 1122 + 289 = 2672 N@m/mm

Live load service load moment = 1.33 x 40.21 x 10^6/2035 =

Lecture - 11-21
Dead load + live load service load moment = 28 951 N@m/mm

Notice that the extreme live load positive moment (40.21 \times 10^6 N@m) occurs at 0.4S of the first span, but is assumed applicable to all positive moment regions (S4.6.2.1.1).

\[ f_s = \frac{(28 951 \times 210 \times 8) \times 115.1}{26 346 000} = 212 \text{ MPa} \]

Allowable service load stresses = 240 MPa > 212 MPa

**Design for negative moment at interior girders:**

Assume distance from design section for negative moment to centerline of girders = 75 mm (S4.6.2.1.6)

**Method I:**

a. **Live Load**

   Maximum factored negative moment per unit width based on Table 11.4.1.2 = 1.75 x 25 990 = 45 483 N.mm/mm

   This moment is applicable to all design sections for negative moment (S4.6.2.1.1).
b. Dead Load

Maximum dead load design moment occur at the design sections adjacent to the middle girder.

Factored moments at centerline of girder based on Table 11.5.4-1:

Deck weight:

\[ 1.25 \times -2679 = -3349 \text{ N@m/mm} \]

Parapet:

\[ 1.25 \times -1403 = -1754 \text{ N@m/mm} \]

FWS:

\[ 1.5 \times -697 = -1046 \text{ N@m/mm} \]

Total dead load factored moment at centerline of girder =

\[-6149 \text{ N@m/mm} \]

Similarly, total factored dead load moments at 0.1S from centerline of girder =

\[-3299 \text{ N@m/mm} \]

By linear interpolation for design section at 75 mm from centerline of girder:

Design dead load factored moment =

\[-5294 \text{ N@m/mm} \]

Dead load + live load design factored negative moment =

\[-5294 - 45483 = -50777 \text{ N@m/mm} \]
Method II:

a. Live Load

Width of equivalent strip for negative moment (S4.6.2.1.3) = 1220 + 0.25S = 1845 mm

The live load negative moment is calculated at the design section to the right and to the left of each interior girder. The extreme value is applicable to all design sections (S4.6.2.1.1).

By inspection, maximum live load negative moment at the design sections for negative moment occurs at the section in the first bay adjacent to the first interior girder.

Factored live load negative moment at centerline of girder
= 1.33 x 1.75 x -39.36 x 10^6/1845 = -49 653 N@m/mm

Factored live load negative moment at 0.9S of first interior bay
= 1.33 x 1.75 x -22.75 + 10^6/1845 = -28 700 N@m/mm

By interpolation for design section at 75 mm from centerline of girder:
maximum live load design negative moment = -43 367 N@m/mm

This moment is applicable to all design sections for negative moment (S4.6.2.1.1).
b. Dead Load

As shown above, dead load factored moment at the design section for negative moment = 5294 N@m/mm.

Dead load + live load design factored negative moment = -5294 - 43367 = -48661 N@m/mm

The difference between the design negative factored moment calculated using Methods I and II is 4.3%, with Method I giving higher values. As explained earlier, Method I gives the upper bound of the moments that can be obtained for the girder spacing under consideration if different combinations of the number of girders in the cross-section and the overhang length were tried. For the subsequent analysis, the values of the moments from Method II were used.

d = total thickness - top cover - ½ bar diameter

Assume #15 bars, diameter = 16 mm, A_{bar} = 200 mm²

d = 200 - 60 - 16/2 = 132 mm

Required area of steel = 1.101 mm²/mm

Required spacing = 200/1.101 = 181.62 mm

Use #15 at 180 mm

Check depth of compression block:

T = 200 mm² x 400 MPa = 80 000 N

a = \frac{80 000}{0.85 \times 28 \times 180} \approx 18.7 \text{ mm}

β₁ = 0.85 for f'_c = 28 MPa (S5.7.2.2)

c = \frac{18.7}{0.85} = 22.0 \text{ mm}

c/d_w = \frac{22.0}{132} = 0.17 < 0.42 (S5.7.3.3.1)

Check for cracking under service limit state (S5.7.3.4).

By inspection, maximum (dead load + live load) service load negative moment stresses occur at the design sections to the right and to the left of the middle girder.

Allowable service load stresses:

f'_{sa} \leq \frac{Z}{t} #0.6 f'_y \leq 240 \text{ MPa}

(d_w A)^{\frac{1}{3}}

Concrete cover = 60 mm - 10 mm integral wearing surface = 50 mm
(Note: maximum clear cover to be used in the analysis = 50 mm)

\[ d_c = \text{clear cover} + \frac{1}{2} \text{bar diameter} \]
\[ = 50 + 8 = 58 \text{ mm} \]

\[ A = 2 \times 58 \times \text{bar spacing} = 20880 \text{ mm}^2 \]

\[ Z = 23000 \text{ N/mm} \]

\[ f_{sa} = 216 \text{ MPa} \]

As explained earlier, both dead load and live load service load stresses are calculated using a modular ratio \( n = 8 \)

Dead load service load moment at the design section for negative moment near the middle = \(-4119 \text{ N@m/mm}\)

Live load service load moment at design section in the first interior bay near the first interior girder = \(-24779 \text{ N@m/mm}\)

This moment is applicable to all negative moment design sections.

Maximum dead load + live load service load moment
\[ = -4119 - 24779 \]
\[ = -28898 \text{ N@m/mm} \]

\[ n = 8 \]

\[ I_{\text{transformed}} = 17381000 \text{ mm}^2 \]

Total dead load + live load service load stresses =
\[ \frac{(28898 \times 180 \times 8) \times 91.6}{17381000} \]
Design of the overhang:

Assume that the bottom of the deck in the overhang region is 20 mm lower than the bottom of other bays as shown in Figure 11.5.4-6. This results in a total thickness of the overhang = 220 mm. This is usually beneficial in resisting the effects of vehicular collision. However, a section in the first bay of the deck, where the thickness is smaller than that of the overhang, must also be checked.

Assumed loads:

Own weight of slab = 0.005 178 N/mm² of deck surface area

Weight of parapet = 0.946 Kg/mm x 9.8 m/sec² = 9.27 N/mm of length of parapet

Future wearing surface = 0.001 436 N/mm² of deck surface area

As required by Article SA13.4.1, there are three design cases to be checked when designing the deck overhang regions.
Design Case 1: Check overhang for horizontal vehicular collision load (SA13.4.1, Case 1):

![Figure 11.5.4-7 - Design Sections in the Overhang Region](image)

The overhang is designed to resist an axial tension force from vehicular collision acting simultaneously with the (collision + dead load) moment.

Resistance factor $\varphi = 1.0$ for extreme event limit state (S1.3.2.1). The Specification requires that load effects in the extreme event limit state be multiplied by $\eta_i \leq 1.05$ for bridges deemed important or $\eta_i \leq 0.95$ for bridges deemed not important. For this example, a value of $\eta_i = 1.0$ was used.

a. at inside face of parapet (Section A-A in Figure 11.5.4-7)

- $M_c$ = moment capacity of the base of the parapet given as 102 000 N@m/mm
- $M_{DL} = -1.25 \times 0.005178 \times (510)^2/2$
  
  $-1.25 \times 9.27 \times (510-190.7)$
  
  $= -4571$ N@m/mm

Design moment = $-4571 - 102 000 = -106 571$ N@m/mm

Design axial tensile force =

$$\frac{R_w}{L_c \%2H} \times \frac{654 000}{4142 \%2 \times 1060} \times 104.4 \text{ N/mm}$$

- $h_{slab} = 220$ mm
- $d = 152$ mm
Required area of steel = 2.12 mm²/mm

Check depth of compression block

\[ T = (2.12) (400) = 848 \text{ N/mm} \]
\[ C = 848 - 104.4 = 743.6 \text{ N} \]
\[ a = \frac{743.6}{0.85 \times 28} = 31.24 \text{ mm} \]

\[ M_n = 848 \left( \frac{31.24}{2} \right) \& 104.4 \left( \frac{152}{2} \& 31.24 \right) = 109,346 \text{ Nmm/mm} \]

Resistance factor = 1.0 for extreme event limit state (S1.3.2.1)

\[ M_R = \varphi M_n = 109,346 \text{ N@m/mm} \quad \text{OK} \]

\[ \frac{c}{d} \left( \frac{31.24}{0.85 \times 152} \right) = 0.24 \text{ steel yields by inspection} \]

b. at design section in the overhang (Section B-B in Figure 11.5.4-7)

Assume that the minimum haunch thickness is at least equal to the difference between the thickness of the interior regions of the slab and the overhang thickness, i.e., 20 mm. This means that when designing the section in the overhang at 75 mm from center of the girder, the total thickness of the slab at this point can be assumed to be 220 mm. For thinner haunches, engineering judgment should be exercised to determine the thickness to be considered at this section.

At the inside face of the parapet, the collision forces are distributed over a distance \( L_c \) for the moment and \( L_c + 2H \) for axial force. It is reasonable to assume that the distribution length will increase for sections that are at a distance from the parapet. In this example, the distribution length was increased at a 30° angle from the base of the parapet (see Figure 11.5.4-8).
Collision moment at design section = 
\[ \frac{M_c L_c}{L_c \% 2 \times 0.577 \times X} \times \frac{102,000 \times 4142}{4142 \% 2 \times 0.577 \times 665} \]
\[ = 86,056 \, N@m/mm \]

Factored dead load moment at design section = -16,348 N@m/mm

Design moment = -86,056 - 16,348 = -102,404 N@m/mm

Design tensile force = 
\[ \frac{R_w}{L_c \% 2 \times H \% 2 \times 0.577 \times X} \]
\[
\frac{654 \, 000}{4142 \%^2 \times 1060 \%^2 \times 0.577 \times 665} = 93.04 \, N/mm
\]

h slab = 220 mm

Required area of steel = 2.10 mm²/mm

\[\text{(2)}\]

c. Check dead load + collision moments at design section in first span

(Section C-C in Figure 11.5.4-7)

The total collision moment can be treated as an applied moment at the end of a continuous strip and the ratio of the moment \( M_2/M_1 \) (see Figure 11.5.4-9) can be calculated for the transverse design strip. As an approximation, it can be taken equal to the ratio of the moments produced by the parapet self-weight at the centerline of the first and second girder, i.e., 2805/9819. The collision moment per unit width at the section under consideration can then be determined by dividing the total collision moment by a distribution length determined using the 30E distribution as illustrated in Figure 11.5.4-8. Dead load at this design section can be determined by interpolation between dead load moments at centerline of girder and at 0.1S.

\[\text{Figure 11.5.4-9 - Assumed Distribution of the Collision Moment Across the Width of the Deck}\]

Collision moment at exterior girder, \( M_1 = -102 \, 000 \, N@m/mm \)

Collision moment at first interior girder, \( M_2 = 102 \, 000 \times 2805/9819 = +29 \, 138 \, N@m/mm \)

By interpolation for a section in the first interior bay at 75 mm from the exterior girder:

\[\text{Total collision moment} = -102 \, 000 + 75(102 \, 000+29 \, 138)/2500 = -98 \, 065 \, N@m\]

Using the 30E angle distribution, as shown in Figure 11.5.4-8: Design collision moment =

\[98 \, 065 \, L_c/[L_c+2x0.577x(740+75)]\]
Based on Table 11.5.4-1:

Factored dead load moment at centerline of exterior girder
= 1.25x(-4045)+1.25x(-9820)+1.5(-393)
= -17 921 N@m/mm

Factored dead load moment at 0.1S of first bay
= 1.25x(-2516)+1.25(-8557)+1.5(-35)
= -13 894 N@m/mm

By interpolation for a section 75 mm from the center of the girder:

Design factored dead load moment
= -17 921 + 75/250 (-17 921+13 894)
= -16 713 N@m/mm

Total design dead load + collision moment
= -79 918-16 713 = -96 631 N@m/mm

Resistance factor =1.0 for extreme event limit state (S1.3.2.1)

Required area of steel = 2.11 mm²/mm

Notice that if Table 11.4.1-2 was used to determine the maximum positive and negative moments in the interior regions of the deck instead of conducting complete live load analysis, the designer will still be able to calculate the moments in the deck overhang by hand. However, some assumptions will need to be made to determine approximate values for the moments near the overhang in the first interior bay of the deck.

**Design Case 2: Vertical collision force (SA13.4.1, Case 2):**

For concrete parapets, the case of vertical collision never controls

**Design Case 3: Check DL+LL (SA13.4.1, Case 3):**

Resistance factor =0.9 for strength limit state (S5.5.4.2.1)

a. Design section in the overhang (Section B-B in Figure 11.5.4-7)

Refer to Figure 11.5.4-6 for dimensions. Use multiple presence factor for a single truck = 1.2 and dynamic load allowance for truck loading = 1.33

Equivalent strip width for live load = 1140 + 0.833 x 365
= 1444 mm (S4.6.2.1.3)
Design factored moment

\[-1.25 \times 0.005 \times 178 \times (1250-75)^2/2\]

\[-1.25 \times 9.27 \times (1250-190.7-75)\]

\[-1.5 \times 0.001 \times 436 \times (1250-510-75)^2/2\]

\[-1.75 \times 1.33 \times 1.2 \times 72 \times 500 \times 365/1444\]

\[-67534\text{ N}\text{mm/mm}\]

d = 220 - top cover - ½ bar diameter

\[= 220 - 60 - 16/2 = 152\text{ mm}\]

Required area of steel = 1.33 mm²/mm \hspace{1cm} (4)

b. Check dead load + live load moments at design section in first span (Section C-C in Figure 11.5.4-7)

Assume slab thickness at this section = 200 mm (see Figure 11.5.4-6)

Calculate dead load and live load moments by using the moments listed in Tables 11.5.4-1 and 11.5.4-2, and by interpolation between moments at centerline of girder and at 0.1S.

Since that the live load negative moment is produced by a load on the overhang, use the overhang strip width at the centerline of girder.

Live load moment arm at centerline of girder = 440 mm

Equivalent strip width = 1506.52 mm (S4.6.2.1.3)

Design (dead load + live load) factored moment = 69563 N/mm/mm

Required area of steel = 1.634 mm²/mm \hspace{1cm} (5)

From the different design cases of the overhang and the adjacent region of the deck, the required area of steel in the overhang = largest of (1), (2), (3) (4) and (5) = 2.12 mm²/mm

Provided top reinforcement in the slab in regions other than the overhang region = #15 at 180 mm = 200/180 = 1.11 mm²/mm.

1.11 mm²/mm provided < 2.12 mm²/mm required, therefore, additional reinforcement is required in the overhang.

Bundle 1 #15 to each top bar in the deck in the overhang region.

Check depth of compression block:

\[T = 400 \times 400 = 160000\text{ N}\]
\[
a = \frac{160 \text{,}000}{0.85 \times 28 \times 180} = 37.3 \text{ mm}
\]

\[
\beta_1 = 0.85 \text{ for } f'_c = 28 \text{ MPa (S5.7.2.2)}
\]

\[
c = \frac{37.3}{0.85} = 43.9 \text{ mm}
\]

Among Sections A, B and C of Figure 11.5.4-7, Section C has the least slab thickness. Hence, the ratio \(c/d_a\) is more critical at this section.

\[
d_a \text{ at section C} = 132 \text{ mm}
\]

Maximum \(c/d_a = 43.9/132 = 0.33 < 0.42 \text{ (S5.7.3.3.1)}\)

Provided area of steel = 2# 15 at 180 mm = 400/180 = 2.22 mm\(^2\)/mm > 2.12 mm\(^2\)/mm

Cracking under service load in the overhang needs to be checked, but it does not control the design in most cases.

Determine the point in the first bay of the deck where the additional bars are no longer needed by determining the point where both (dead load + live load) moment and (dead load + collision) moments are less than or equal to the moment of resistance of the deck slab without the additional top reinforcement.

Negative moment resistance of the deck slab reinforced with #15 at 180 mm is 49,292 N@mm/mm for strength limit state (resistance factor = 0.9), or 54,769 N.mm/mm for the extreme event limit state (resistance factor = 1.0)

Theoretical cutoff point from centerline of exterior girder = 723 mm

Extend the additional bars beyond this point for a distance equal to the cut-off length. In addition, check that the provided length measured beyond the design section for moment is larger than the development length (S5.11.1.2.1).

**Cut-Off Length Requirement (S5.11.1.2)**

Cut-off length = 240 mm

Required length past the centerline of exterior girder = 723 + 240 = 963 mm

**Development length (S5.11.2)**

Basic development length:
larger of: \( \frac{0.02 A_b f_y}{\sqrt{f_{cr}^2}} \), \( \frac{0.02 \times 200 \times 400}{\sqrt{28}} \), 302 mm

or 0.06 \( d_b \) \( f_y \) = 0.06 \( d_b \) 16 \( f_y \) = 384 mm

or 300 mm

Basic development length = 384 mm

Correction factors:

- Epoxy-coated bars = 1.2 (S5.11.2.1.2)
- Two bundled bars = 1.0 (S5.11.2.3)
- Spacing > 150 mm = 0.8 (S5.11.2.1.3)

Development length = 384 x 1.2 x 1.0 x 0.8 = 369 mm

Required length of additional bars past the centerline of exterior girder = 75 + 369 = 444 mm < 963 mm provided

**LONGITUDINAL REINFORCEMENT**

Bottom distribution reinforcement (S9.7.3.2)

Percentage of longitudinal reinforcement = \( \frac{3840}{\sqrt{5}} \) #67%

Figure 11.5.4.10 - Length of the overhang additional bars
S = 2500-75-75 = 2350 mm (S9.7.2.3)

Percentage = \frac{3840}{\sqrt{2350}} \approx 79\% > 67\%

Use 67\% of transverse reinforcement

Transverse reinforcement = \#15 at 210 mm = 0.95 mm²/mm

Required longitudinal reinforcement = 0.67 \times 0.95 = 0.64 mm²/mm

Use \#15 bars, diameter = 16 mm, area = 200 mm²

Required spacing = \frac{200}{0.64} = 313 mm

Assume maximum allowed spacing = 300 mm

Use \#15 at 300 mm

**Deck top longitudinal reinforcement over intermediate supports of the girders**

1. Simple span precast girders made continuous for live load: design according to Article S5.14.1.2.7

2. Continuous steel girders: Design according to Article S6.10.1.2

Assume continuous steel girders (S6.10.1.2)

Required area of steel = 1\% of the deck cross-section

= 0.01 \times 200 = 2.0 mm²/mm

2/3 of steel in top layer = 1.33 mm²/mm

1/3 of steel in bottom layer = 0.67 mm²/mm

Maximum allowed spacing = 150 mm (S6.10.1.2)

Use \#15 at 150 mm in top and bottom layers

A_s provided = 1.33 mm²/mm > A_s required
Figure 11.5.4-11 - Deck Reinforcement
Table 11.5.4-2 - Total Live Load Moments N@m (x1 000 000)**

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**Includes a multiple presence factor of 1.2 for single truck and 1.0 for two trucks. Does not include dynamic load allowance.
Table 11.5.4-1 - Dead Load Moments N@m/mm

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Dead Load of FWS

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Dead Load of Slab

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11.5.5 Cast-in-Place Concrete Deck Design Example - Empirical Method

The same design example that was designed using the traditional method is redesigned using the empirical method design. For the general requirements that need to be satisfied for the empirical method to be applicable, refer to Article S9.7.2.

Maximum allowed effective span to apply the empirical method = 4100 mm (S9.7.2.4)

Assuming that the deck is supported on steel girders that have 300 mm wide flanges, the effective length of slab

= girder spacing - ½ flange width = 2500 - 150 = 2350 mm (S9.7.2.3) < 4100 mm OK

The ratio of effective span to design depth has to be between 6 and 18 (S9.7.2.4). Assume slab thickness = 200 mm of which 10 mm is an integral wearing surface. Effective span/design depth = 2350/(200-10) = 12.37 > 6 OK

Minimum required reinforcement in each direction of the bottom layer = 0.570 mm/mm (S9.7.2.5)

Cross-sectional area of # 15 bars = 200 mm²

Maximum required spacing of # 15 bars = 200/0.570 = 350 mm

Maximum allowed spacing in the specifications = 450 mm (S9.7.2.5)

Assume maximum spacing permitted by the owner is 300 mm
Therefore, use # 15 bars @ 300 mm in both directions of bottom reinforcement layer

Minimum permitted reinforcement in each direction of top reinforcement layer = 0.380 mm/mm (S9.7.2.5)

Maximum permitted spacing of # 15 bars = 200/0.380 = 526 mm

Assume maximum allowable spacing is 300 mm
Therefore, use # 15 bars @ 300 mm in both directions of top reinforcement layer
11.6 OVERVIEW OF METAL DECKS

11.6.1 Metal Grid Decks

Grid decks are those floor systems composed of main elements that span between beams, stringers or cross-beams, and secondary members that interconnect the main elements and span between them. The main and secondary elements are required to be securely joined together and may form a rectangular or diagonal pattern.

Force effects may be determined using one of the following methods:

- the approximate methods specified in Article S4.6.2.1, as applicable
- orthotropic plate theory,
- equivalent grillage, or
- use of design aids provided by the manufacturers, if the performance of the deck is documented and supported by sufficient technical evidence.

Filled or partially filled grid decks are formed when a metal grid or other metal structural system is filled completely or partially with concrete, respectively. These decks may be modeled for analysis as either an orthotropic plate, or an equivalent grillage. In this case, flexural and torsional rigidities may be obtained by other accepted and verified approximate methods or by physical testing. Mechanical shear transfer devices, including indentations, embossment, sand coating of surface, and other appropriate means may be used to enhance the composite action between elements of the grid and the concrete fill. If a filled or partially filled deck is considered to be composite with its supporting components for the purpose of designing those components, the effective width of slab in composite section shall be as specified in Table S4.6.2.1.3-1, reproduced herein as Table 11.4.1-1 of this lecture.

A minimum 38 mm thick structural overfill should be provided where possible. The weight of the concrete fill is assumed to be carried solely by the metal portion of the deck. The transient loads and superimposed permanent loads may be assumed to be supported by the grid bars and concrete fill acting compositely. A structural overfill may be considered as a part of the composite structural deck.

Filled and partially filled grid decks have better potential for composite action with the supporting components due to their considerable in-plane rigidity. Thus, these grids are required to be attached to supporting components by welding or by shear studs using details similar to Figure 11.6.1-1. The preferred method of shear transfer is by welded studs encased in a concrete haunch. The
connection is designed to resist the large shear forces developed at the interface between the deck and the supporting components due to the composite action.

![Diagram of shear connection](image)

**Figure 11.6.1-1 - An Acceptable Shear Connection of Partially- and Fully-Filled Grid Decks to Beams**

For partially filled grid, the internal connection among the elements of the steel grid within the concrete fill need not be investigated for fatigue. The welded internal connection among those elements of the steel grid which are not within the concrete fill shall be considered as Category "E" details, unless documentation is provided to the contrary.

Open grid floors, consisting of either an unfilled metal grid or other metal structural system, are required to be connected to the supporting components by welding or by mechanically fastening at each main element. This requirement is the result of long-term experience which indicates that even where there is apparently an insignificant degree of composite action between the deck and its supporting components, high stresses may develop at their interface, resulting in local failures and separation of the deck. Provisions for weld and edge treatment are provided in Article S9.8.2.2.

Unfilled grid floors may be made composite with a reinforced concrete slab which sets on top of the unfilled metal deck. Composite
action between the concrete slab and the grid deck is ensured either by providing shear connectors, or by other effective means capable of resisting horizontal and vertical components of interface shears. Composite action between the metal grid and the supporting components should be ensured by mechanical shear connectors as indicated in Figure 11.6.1-2. Provisions for designing this type of decks are provided in Article S9.8.2.4.2.

Figure 11.6.1-2 - An Acceptable Shear Connection of Unfilled Grid Decks Composite with Reinforced Concrete Slabs to Beams

11.6.2 Orthotropic Steel Decks

Steel decks that consist of a deck plate stiffened and supported by longitudinal ribs and transverse floorbeams are considered orthotropic decks. The deck plate acts as a common flange of the ribs, the floorbeams and the main longitudinal components of the bridge. To ensure the structural integrity of the deck, the connection of the deck to floorbeam should be designed for full composite action, whether or not the wearing surface is considered composite for the design of the steel components. In addition, connections suitable to develop composite action between the deck and the main longitudinal components should be provided where practical.

Following the traditional assumption, a 45E distribution of the tire pressure may be assumed to occur in all directions from the
The wearing surface should be regarded as an integral part of the total orthotropic deck system. Past experience should be considered in selection of a wearing surface and in determining its long-term contribution to the structural system. Wearing surfaces, acting compositely with the deck plate, may reduce deformations and stresses in orthotropic decks. Thus, the Specification requires that the wearing surface be bonded to the top of the deck plate. However, the contribution of a wearing surface to the stiffness of the members of an orthotropic deck may only be considered if structural and bonding properties are satisfactorily demonstrated over the temperature range of -30°C to +50°C. This is required because the effectiveness of the wearing surface is dependent upon its temperature-dependent elastic modulus and bond characteristics. Acceptable methods of analysis and design provisions for orthotropic decks are presented in Articles S9.8.3.4 through S9.8.3.6.

11.7 OVERVIEW OF WOOD DECKS AND DECK SYSTEMS

11.7.1 Design requirements

The following design requirements are applicable to wood decks and deck systems which are currently being designed and built in the United States and which have demonstrated acceptable performance. The supporting components may be metal, concrete or wood.

Wood decks may be analyzed using one of the following methods:

- the approximate equivalent strip method specified in Article S4.6.2.1,
- orthotropic plate theory, or
- equivalent grillage model.

In wood decks with closely spaced supporting components, the assumption of infinitely rigid supports upon which approximate methods of analysis are based, is not valid. Therefore, the Specification requires that the orthotropic plate or the equivalent grid be used to model the deck system, including the supporting components, if the spacing of the supporting components is less than either 910 mm or 6.0 times the nominal depth of the deck.

Wood deck floor systems need to be designed for shear. In transverse decks, maximum shear is computed at a distance from the support equal to the depth of the deck. In longitudinal decks, maximum shear is computed in accordance with the provisions of
Article S8.7. Shear effects may be neglected in the design of stress-laminated decks.

Generally, thermal expansion has not presented problems in wood deck systems. Except for stress-laminated decks and tightly placed glued-laminated panels, most wood decks inherently contain gaps at the butt joints which can absorb thermal movements. Therefore, thermal effects may be neglected in plank decks and spike-laminated decks, but need to be considered for stress-laminated and glued-laminated panel decks made continuous over more than 120,000 mm. In case of considering thermal effects, the coefficient of thermal expansion of wood parallel to its fibers shall be taken as $3.6 \times 10^{-6}$ per degree Celsius.

For skewed bridges with transverse decks, placing the laminations on the skew is the easiest and most practical method for small skew angles. Cutting the ends of the laminations on the skew provides a continuous straight edge. The Specification allows this arrangement only for decks with skew angle less than 25 degrees. The 25-degree arbitrary limit is chosen to keep the direction of the fibers close to that of the principal flexural stresses. For larger skew angles, the transverse laminations shall be placed normal to the supporting components, and the free ends of the laminations at the ends of the deck shall be supported by either a diagonal beam or by other suitable means. For longitudinal decks, except for stress-laminated wood, any skew angle can generally be accommodated by offsetting each adjacent lamination on the skew.

11.7.2 Glued-Laminated Decks

Glued-laminated timber panel decks consist of a series of panels, prefabricated with water-resistant adhesives, that are tightly abutted along their edges. This form of deck is appropriate only for roads having low to medium volumes of commercial vehicles. The requirements of the Specification are based on the results of laboratory tests and on past experience. Therefore, the Specification provisions are applicable to the range of spans of the decks built to-date, i.e., transverse deck panels 900 to 1800 mm wide and longitudinal deck panels 1050 to 1350 mm wide.

Based upon current practices, where panels are attached to wood supports, the tie-downs consist of metal brackets that are bolted through the deck and attached to the sides of the supporting component. Lag screws or deformed shank spikes may be used to tie panels down to wood supports. Where panels are attached to steel beams, metal clips are used as tie-downs. These clips extend over the beam flange which are bolted through the deck.

Panels of glued-laminated wood may be used for bridge decks and may be placed parallel or perpendicular to traffic. Where panels are parallel to traffic, transverse stiffener beams with the number and strength specified in Article S9.9.4.3.1 are required to interconnect the panels. Despite the fact that the transverse stiffener beams assure...
interpanel shear transfer of loads, some relative deflection will take place. Under frequent heavy loads, this relative deflection will cause reflective cracking of bituminous wearing surfaces.

Where panels are perpendicular to traffic, mechanical fasteners, splines, dowels or stiffener beams may be used for interconnection of panels. Provisions for each type of connection are provided in Article S9.9.4.3.2.

If panels are non-interconnected they will likely cause reflective cracking in the wearing surface at the butt joints, even under relatively low levels of loading. In order to avoid the extensive maintenance the wearing surface may require, it is appropriate to use non-interconnected panels only for roads having low volumes of commercial vehicles.

11.7.3 Stress-Laminated Decks

Stress-laminated decks consist of a series of wood laminations that are placed edgewise and post-tensioned together, normal to the direction of the lamination. The structural performance of these decks relies on friction, due to transverse prestress, between the surfaces of the laminations to transfer force effects. Unlike spiked or bolted connections in wood, the friction-based performance of stress-laminated decks does not deteriorate with time under the action of repeated heavy loads. The majority of decks of this type include laminations which are 50 to 75 mm thick. The increased load distribution and load sharing qualities of this type of deck, coupled with its improved durability under the effects of repeated heavy vehicles, make it the best choice among the several wood decks for high volume road application. For skewed decks, the prestressing can be arranged as shown in Figures 11.7.3-1 and 11.7.3-2. However, stress-laminated decks are not permitted where the skew angle exceeds 45°.
Laminations are required to be nailed together prior to post-tensioning. Since nailing is only a temporary construction convenience in stress-laminated decks, it should be kept as close to minimum requirements as possible. Excessive nailing may inhibit the build-up of elastic strains during transverse stressing which could subsequently contribute to decreasing its effectiveness. Nailing requirements are specified in Article S9.9.5.2.

Post-tensioning bars pass through holes in the laminations. Empirical limitations on hole size and spacing are provided in Article S9.9.5.2.
S9.9.5.4. The intent of these limitations is to minimize the negative effects of the holes on the performance of the deck. Only drilled holes are permitted. Punched holes can seriously affect the performance of the laminates by breaking the wood fibers in the vicinity of the holes. Therefore, only drilled holes are permitted.

To allow for the use of lamination lengths that are less than the deck length, butt joints are permitted. However, reducing or eliminating the occurrence of butt joints and/or distributing butt joints uniformly will result in improved performance. The Specification requires that not more than one butt joint occurs in any four adjacent laminations within a 1200 mm distances (see Figure 11.7.3-3).

![Figure 11.7.3-3 - Minimum Spacing of Lines of Butt Joints](image)

Stress-laminated decks have a tendency to develop curvature perpendicular to the laminates when they are transversely stressed. This occurs because of unavoidable misalignments which can cause the prestressing to become eccentric to portions of the deck resulting in bending of the deck. Therefore, a tie-down more effective than toenailing or drift pins is required. Tie-downs using bolts or lag screws ensure proper contact of the deck with the supporting members. Each tie-down shall consist of a minimum of two 20 mm diameter bolts for decks up to and including 300 mm deep, and two 27 mm diameter bolts for decks more than 300 mm deep. Decks need to be tied-down at every support and the maximum spacing of the tie-downs along each support should not exceed 900 mm.

Provisions for design of prestressing systems and railings for stress-laminated decks are provided in Article S9.9.5.6.

11.7.4 Spike-Laminated Decks

Spike-laminated decks consist of a series of lumber laminations that are placed edgewise between supports and spiked together on their wide face with deformed spikes of sufficient length to fully penetrate four laminations. Butt splicing the laminations is not permitted within their unsupported length. The use of spike-laminated
decks should be limited to secondary roads with low truck volumes, i.e., ADTT significantly less than 100 trucks per day. The spikes shall be placed in lead holes that are bored through pairs of laminations at each end and at intervals not greater than 300 mm in an alternating pattern near the top and bottom of the laminations, as shown in Figure 11.7.4-1.

Figure 11.7.4-1 - Spike Layout for Spike-Laminated Decks

Deck panels are required to be interconnected if practical. The panels may be interconnected with mechanical fasteners, splines, dowels or stiffener beams to transfer shear between the panels. Stiffener beams, comparable to those specified for glued-laminated timber panels, are recommended. Use of an adequate stiffener beam enables the spike-laminated deck to approach the serviceability of glue-laminated panel construction and the provisions for interconnected glued-laminated decks will be applicable.

If non-interconnected decks are used, they should be limited to secondary and rural roads. With time, the deck may begin to
delaminate in the vicinity of the edge-to-edge panel joints. The load
distribution provisions given for the non-interconnected panels are
intended for use in the evaluation of existing non-interconnected panel
decks and interconnected panel decks in which the interconnection is
no longer effective.

11.7.5 Plank Decks

Wood plank decks consist of a series of lumber planks placed
flatwise on supports. Butt joints must be placed over supports. The
joints in adjacent blanks are staggered a minimum of 900 mm. This
type of deck has been used on low volume roads with little or no
heavy vehicles, and it is usually economical. However, these decks
provide no protection against moisture penetrating the supporting
members, they do not readily accept and/or retain a bituminous
wearing surface, and usually require continuous maintenance if used
by heavy vehicles.

To ensure the integrity of the deck, blanks need to be tied
down to the supporting elements. Two nails of minimum length equal
to twice the plank thickness should be used as a tie-down for each
plank supported on wood beams. In case of steel beams, planks
should be bolted to the beams or nailed to wood nailing strips. The
strips should be at least 100 mm thick and their width should exceed
that of the beam flange. The strips should be secured with A307 bolts
at least 16 mm in diameter placed through the flanges, spaced not
more than 1200 mm apart and no more than 450 mm from the ends
of the strips.

11.7.6 Wearing Surfaces for Wood Decks

Experience has shown that unprotected wood deck surfaces
are vulnerable to wear and abrasion and/or may become slippery
when wet. Therefore, wood decks must be provided with a wearing
surface. In general, wearing surfaces for wood decks are required to
be continuous and no nails, except in wood planks, are to be used to
fasten them to the deck. Based on past experience, bituminous
wearing surfaces are recommended for wood decks. To enhance
adhesion, the surface of wood decks should be free of surface oils.
Proper placement of the wearing surface also prevents excessive
bleeding of the preservative treatment through the wearing surface
which may seriously reduce the adhesion. The plans and
specifications should clearly state that the deck material be treated
using the empty cell process followed by an expansion bath or
steaming.

Plant mix asphalt with a minimum compacted depth of 50 mm
can be used on wood decks. An approved tack coat needs to be
applied to wood decks prior to the application of the asphalt wearing
surface. The application of the tack coat greatly improves the
adhesion of asphalt wearing surfaces. The tack coat may be omitted
when a geotextile fabric is used, subject to the recommendations of
the manufacturer.
Due to the smooth surface of individual laminations and glued-laminated decks, it is beneficial to provide a positive connection between the wood deck and the wearing surface in order to ensure proper performance. This connection may be provided mechanically or with a geotextile fabric.

Chip seal can also be used as a wearing surface on wood decks. In such case, a minimum of two layers should be provided. Chip seal wearing surfaces have a good record as applied to stress-laminated decks because the behavior of these decks approaches that of solid slabs. Other laminated decks may have offset laminations creating irregularities on the surface, and it is necessary to provide an adequate depth of wearing surface to provide proper protection to the wood deck.
LECTURE 12 - STEEL DESIGN - I

12.1 OBJECTIVE

12.2 NEW PROVISIONS IN LRFD SPECIFICATION NOT CONTAINED IN LFD SPECIFICATION

12.2.1 Deflection Limitations

Deflection requirements for other than orthotropic decks are now optional (see Article S2.5.2.6.2), as are requirements for limits on span-to-depth ratios (see Article S2.5.2.6.3).

However, flange stress controls of Articles S6.10.5, summarized below, and S6.10.10.2, which are similar in intent, are mandatory and provide limits on permanent deformation in lieu of deflection limitations. Load Combination Service II in Table S3.4.1-1 applies to this check. For compact members, the investigation of permanent deflection may be based on moment redistribution.

Flange stresses in positive and negative bending are not to exceed:

- for both steel flanges of composite sections:

  \[ f_f \# 0.95 R_b R_h F_{yf} \]  \hspace{1cm} (12.2.1-1)

- for both flanges of non-composite sections:

  \[ f_f \# 0.80 R_b R_h F_{yf} \]  \hspace{1cm} (12.2.1-2)

where:

\( R_b \) = load-shedding factor specified in Article 12.7.3

\( R_h \) = flange-stress reduction factor specified in Article 12.7.3

\( f_f \) = elastic flange stress caused by the factored loading (MPa)

\( F_{yf} \) = yield stress of the flange (MPa)

A resistance factor is not applied since the specified limit is a serviceability criterion for which the resistance factor is 1.0.

These provisions are intended to apply to the design live load specified in Article S3.6.1.1. If this criterion were to be applied to a
permit load situation, a reduction in the load factor for live load should be considered.

This limit state check is intended to prevent objectionable permanent deflections due to expected severe traffic loadings which would impair rideability. It corresponds to the overload check in the 1992 AASHTO Standard Specification and is merely an indicator of successful past practice, the development of which is described in Vincent (1969).

Under the load combinations specified in Table S3.4.1-1, the criterion for control of permanent deflections does not govern for composite non-compact sections, therefore, it need not be checked for those sections. This may not be the case under a different set of load combinations.

Article S6.10.10.2 provides comparable controls for beams and girders designed by inelastic procedures, such as the mechanism method or the unified autostress method.

12.2.2 Fatigue

12.2.2.1 GENERAL


These provisions are based upon two principles of fatigue of welded steel details:

- If all of the stress ranges that a welded steel detail experiences in its lifetime are less than the constant-amplitude fatigue threshold (i.e., the maximum stress range is less than the threshold), the detail will not experience fatigue crack growth; otherwise

- the fatigue life of the detail can be estimated considering an effective (weighted average of sorts) stress range, which represents all of the varying magnitudes of stress range experienced by the detail during its lifetime (Fisher, et al, 1983).

These two principles result in two branches in the flow of fatigue design, infinite life design and finite life design.

Further, an explicit check of whether any of the tensile live load stresses experienced by the detail overcome the compressive dead load stresses is specified in Article S6.6.1.2.1 prior to the consideration of the fatigue provisions.
Fatigue details for bridges with higher truck traffic volumes are designed for infinite life. This practice is carried over from both the Standard Specifications and the Guide Specifications.

Bridges with lower truck traffic volumes are designed for the fatigue life required by the estimated site-specific traffic volumes projected for their lifetimes.

12.2.2.2 FATIGUE LOAD

On the load side of the LRFD equation, the specified load condition for fatigue is a single truck, the current HS20 truck with a fixed rear axle spacing of 9.0 m. The load effects (e.g., moments, shear, etc.) caused by this truck are of lesser magnitudes than those caused by the combination of a truck and a uniformly distributed load applied for strength design. Further, the truck occupies a single lane on the bridge at a time, not multiple lanes as in strength design.

This fatigue load produces a lower calculated stress range than that of the Standard Specifications. This reduction in calculated stress range is offset by an increase in the number of cycles to be considered. The number of cycles to be considered is the number of cycles due to the trucks anticipated to cross the bridge in the most heavily traveled lane during its design life, as calculated by Equation 12.2.2.2-1. This will be a large number of cycles compared to the design cycles in the 15th Edition of AASHTO. Both the lower stress range and the increased number of cycles are more reflective of the actual conditions experienced by the bridge. In this consideration, these fatigue provisions resemble the Guide Specifications.

\[ N = (365) (75) n (ADTT)_{SL} \]  
\[ (12.2.2.2-1) \]

where:

\[ n = \text{number of stress range cycles per truck passage given in Table 12.2.2.2-1} \]

\[ (ADTT)_{SL} = \text{single-lane ADTT} \]
Table 12.2.2.2-1 - Cycles per Truck Passage, n

<table>
<thead>
<tr>
<th>Longitudinal Members</th>
<th>Span Length</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&gt; 12 000 mm</td>
<td>#12 000 mm</td>
<td></td>
</tr>
<tr>
<td>Simple-Span Girders</td>
<td>1.0</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>Continuous Girders</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1) near interior support</td>
<td>1.5</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>2) elsewhere</td>
<td>1.0</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>Cantilever Girders</td>
<td>5.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trusses</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse Members</td>
<td>Spacing</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 6000 mm</td>
<td>#6000 mm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>2.0</td>
<td></td>
</tr>
</tbody>
</table>

In the absence of better information, the single-lane average daily truck traffic shall be taken as:

\[ ADTT_{SL} = p \times ADTT \]  

(12.2.2.2-2)

where:

\[ ADTT \] = the number of trucks per day in one direction averaged over the design life

\[ ADTT_{SL} \] = the number of trucks per day in a single-lane averaged over the design life

\[ p \] = taken as specified in Table 12.2.2.2-2
Table 12.2.2.2-2 - Fraction of Truck Traffic in a Single Lane, \( p \)

<table>
<thead>
<tr>
<th>Number of Lanes Available to Trucks</th>
<th>( p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>0.85</td>
</tr>
<tr>
<td>3 or more</td>
<td>0.80</td>
</tr>
</tbody>
</table>

The single-lane ADTT is that for the traffic lane in which the majority of the truck traffic crosses the bridge. On a typical bridge with no nearby entrance/exit ramps, the shoulder lane carries most of the truck traffic.

Since future traffic patterns on the bridge are uncertain, the frequency of the fatigue load for a single lane is assumed to apply to all lanes.

Research has shown that the Average Daily Traffic, ADT, including all vehicles, i.e., cars and trucks, is physically limited to about 20,000 vehicles per lane per day under normal conditions. This limiting value of traffic should be considered when estimating the ADTT. The ADTT can be determined from the ADT by multiplying by the fraction of trucks in the traffic. In lieu of site-specific fraction of truck traffic data, the values of Table 12.2.2.2-3 may be applied for routine bridges.

Table 12.2.2.2-3 - Fraction of Trucks in Traffic

<table>
<thead>
<tr>
<th>Class of Highway</th>
<th>Fraction of Trucks in Traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Interstate</td>
<td>0.20</td>
</tr>
<tr>
<td>Urban Interstate</td>
<td>0.15</td>
</tr>
<tr>
<td>Other Rural</td>
<td>0.15</td>
</tr>
<tr>
<td>Other Urban</td>
<td>0.10</td>
</tr>
</tbody>
</table>

In the LRFD Specification, 75% of the stress range due to the passage of the fatigue load (the current HS20 truck with a fixed rear axle spacing) is considered representative of the effective stress range. One-hundred-fifty percent of the stress range of the fatigue load is considered representative of the maximum stress range. Only the 75% is specified in the load combinations table of the Loads and Load Factors Section of the Specification. The 150% is accounted for by modifying the resistance side of the equation.
12.2.2.3 FATIGUE RESISTANCE

On the resistance side of the equation, while the provisions of the LRFD Specification appear quite different than those of the Standard Specifications, they are little changed in terms of effect.

The nominal fatigue resistance is taken as:

\[
(\Delta F)_n \left( \frac{A}{N} \right)^{\frac{1}{3}} \cdot \frac{1}{2} (\Delta F)_{TH}
\]  

(12.2.2.3-1)

where:

A = constant taken from Table 12.2.2.3-1

\((\Delta F)_{TH}\) = constant-amplitude fatigue threshold taken from Table 12.2.2.3-1

Table 12.2.2.3-1 - Detail Category Constant, A, and Thresholds

<table>
<thead>
<tr>
<th>DETAIL CATEGORY</th>
<th>CONSTANT, A TIMES 10^{11} (MPa^{3})</th>
<th>THRESHOLD (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>82.0</td>
<td>165</td>
</tr>
<tr>
<td>B</td>
<td>39.3</td>
<td>110</td>
</tr>
<tr>
<td>BN</td>
<td>20.0</td>
<td>82.7</td>
</tr>
<tr>
<td>C</td>
<td>14.4</td>
<td>69.0</td>
</tr>
<tr>
<td>CN</td>
<td>14.4</td>
<td>82.7</td>
</tr>
<tr>
<td>D</td>
<td>7.21</td>
<td>48.3</td>
</tr>
<tr>
<td>E</td>
<td>3.61</td>
<td>31.0</td>
</tr>
<tr>
<td>EN</td>
<td>1.28</td>
<td>17.9</td>
</tr>
<tr>
<td>M164 (A325M) Bolts in Axial Tension</td>
<td>5.61</td>
<td>214</td>
</tr>
<tr>
<td>M253 (A490M) Bolts in Axial Tension</td>
<td>10.3</td>
<td>262</td>
</tr>
</tbody>
</table>

If the maximum stress range is less than the constant-amplitude fatigue threshold, then all of the stress ranges experienced by the detail are less than the threshold and fatigue crack growth is not expected. In this case, the detail is said to have infinite life. The maximum stress range is assumed to be twice the effective stress range. Therefore, since Equation 12.2.2.3-1 is a check on the effective stress range, the constant-amplitude threshold in the
equation is divided by two. The constant-amplitude fatigue threshold of the LRFD Specification is the allowable stress range for over two million cycles of the Standard Specifications.

If the maximum stress range is greater than the constant-amplitude fatigue threshold, fatigue crack growth is expected and the detail must be checked to insure that the necessary finite life can be achieved. Since an infinite number of different finite fatigue lives or numbers of design cycles are now possible, the tables of limited numbers of allowable stress ranges based upon the several numbers of design cycles (i.e., 100 000; 500 000; 2 000 000 and over 2 000 000 cycles) have been replaced by equations for each design category. These equations are represented by Equation 12.2.2.3-1 above and its accompanying tables. The resistances represented by these equations are, however, identical to those of the tables of the Standard Specifications as shown below.

![Figure 12.2.2.3-1 - Stress Range Versus Number of Cycles](image)

In the past, the decision was made to design bridges with higher daily truck traffic volumes to insure infinite life. In the Standard Specifications, infinite life for such bridges is mandated by footnote "c" to Table 10.3.2A. This decision was partially based upon the notion that high daily traffic volumes are difficult to predict accurately. Twenty-five hundred trucks per day was arbitrarily chosen to represent the volume beyond which infinite life is desirable.

The philosophy of infinite life for higher truck traffic volume bridges will be maintained in the LRFD Specification.

12.2.2.4 SPECIAL REQUIREMENTS FOR GIRDER WEBS

New provisions are provided to control out-of-plane flexing and fatigue cracking of the web under repeated live loading.
Webs without longitudinal stiffeners shall satisfy the following requirements:

- If \( \frac{2D_c}{t_w} \leq 5.70 \sqrt{\frac{E}{F_{yw}}} \), then:
  \[
  f_{cf} \geq F_{yw}
  \]  
  (12.2.2.4-1)

- If \( \frac{2D_c}{t_w} > 5.70 \sqrt{\frac{E}{F_{yw}}} \), then:
  \[
  f_{cf} \geq 32.5E \left( \frac{t_w}{2D_c} \right)^2
  \]  
  (12.2.2.4-2)

where:

- \( f_{cf} \) = maximum compressive elastic flexural stress in the compression flange due to the unfactored permanent load and the fatigue loading, taken as being indicative of the maximum flexural stress in the web (MPa)
- \( F_{yw} \) = yield strength of the web (MPa)
- \( D_c \) = depth of the web in compression (mm)
- \( t_w \) = web thickness (mm)

The elastic bend-buckling capacity of the web given by Equation 2 is based on an elastic buckling coefficient, \( k \), equal to 36.0. This value is between the theoretical \( k \) value for bending-buckling of 23.9 for simply supported boundary conditions at the flanges and the theoretical \( k \) value of 39.6 for fixed boundary conditions at the flanges (Timoshenko and Gere 1961). This intermediate \( k \) value is used to reflect the rotational restraint offered by the flanges. The specified web slenderness limit of 5.70 \( (E/F_{yw})^{1/2} \) is the web slenderness at which the section reaches the yield strength according to Equation 2.

Longitudinal stiffeners theoretically prevent bend-buckling of the web; thus, the provisions in this article do not apply to sections with longitudinally stiffened webs.

For the loading and load combination applicable to this limit state, it is assumed that the entire cross-section will remain elastic and, therefore, \( D_c \) can be determined as specified in Article S6.10.3.1.4a.

The shear force in unstiffened webs and in webs of hybrid sections is already limited to either the shear yielding or the shear buckling force at the strength limit state by provisions which are summarized in Article 12.7.6. Consequently, these requirements need not be checked for those sections.
Webs of homogeneous sections with transverse stiffeners and with or without longitudinal stiffeners shall be proportioned to satisfy:

\[ v_c = 0.58 C F_{yw} \]  

(12.2.2.4-3)

where:

- \( F_{yw} \) = yield strength of the web (MPa)
- \( C \) = ratio of shear buckling stress to shear yield strength as specified in Article 12.7.6.3
- \( v_c \) = maximum elastic shear stress in the web due to the unfactored permanent load and the fatigue loading (MPa)

### 12.2.3 Resistance Factors

The resistance factors, \( \phi \), for the strength limit state are:

- for flexure .......................................................... \( \phi_1 = 1.00 \)
- for shear ............................................................ \( \phi_v = 1.00 \)
- for axial compression, steel only ...................... \( \phi_c = 0.90 \)
- for axial compression, composite .................. \( \phi_c = 0.90 \)
- for tension, fracture in net section ................ \( \phi_u = 0.80 \)
- for tension, yielding in gross section .......... \( \phi_y = 0.95 \)
- for bearing on pins, in reamed, drilled or bolted holes and milled surfaces ........ \( \phi_b = 1.00 \)
- for bolts bearing on material ....................... \( \phi_{bb} = 0.80 \)
- for shear connectors ........................................ \( \phi_{sc} = 0.85 \)
- for A325M and A490M bolts in tension .......... \( \phi_t = 0.80 \)
- for A307 bolts in tension ............................... \( \phi_1 = 0.80 \)
- for A307 bolts in shear ................................. \( \phi_s = 0.65 \)
- for A325M and A490M bolts in shear .......... \( \phi_s = 0.80 \)
- for block shear ............................................... \( \phi_{bs} = 0.80 \)
- for weld metal in complete penetration welds:
  - shear on effective area ......................... \( \phi_{e1} = 0.85 \)
  - tension or compression normal to effective area \( \phi = \text{base metal} \)
    - tension or compression parallel to axis of the weld \( \phi = \text{base metal} \)
- for weld metal in partial penetration welds:
  - shear parallel to axis of weld ............... \( \phi_{e2} = 0.80 \)
  - tension or compression parallel to axis of weld \( \phi = \text{base metal} \)
  - compression normal to the effective area \( \phi = \text{base metal} \)
  - tension normal to the effective area .... \( \phi_{e1} = 0.80 \)
- for weld metal in fillet welds:
  - tension or compression parallel to axis of the weld \( \phi = \text{base metal} \)
  - shear in throat of weld metal ............... \( \phi_{s2} = 0.80 \)
- for resistance during pile driving ............... \( \phi = 1.00 \)
All resistance factors for the extreme event limit state, except for bolts, shall be taken as 1.0.

12.2.4 Diaphragm Spacing

The 7620 mm spacing limit in the current AASHTO Specification has been eliminated. A structural investigation and analysis to determine need and spacing of diaphragms is now required.

12.2.5 Pins

New requirements are provided for pins subjected to combined flexure and shear.

12.3 TENSILE RESISTANCE

The net area, \( A_n \), of a member is the sum of the products of thickness and the smallest net width of each element. The width deducted for all holes; standard oversize and slotted holes, shall be taken as 2 mm greater than the hole size specified in Article S6.13.2.4.2. The net width shall be determined for each chain of holes extending across the member along any transverse, diagonal, or zigzag line.

The net width for each chain shall be determined by subtracting from the width of the element the sum of the widths of all holes in the chain and adding the quantity \( s^2/4g \) for each space between consecutive holes in the chain, where:

\[
\begin{align*}
    s & = \text{pitch of any two consecutive holes (mm)} \\
    g & = \text{gage of the same two holes (mm)} \\
\end{align*}
\]

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

The development of the "\( s^2/4g \)" rule for estimating the effect of a chain of holes on the tensile resistance of a section is described in McGuire (1968). Although it has theoretical shortcomings, it has been used for a long time and has been found to be adequate for ordinary connections.

In designing a tension member, it is conservative and convenient to use the least net width for any chain together with the full tensile force in the member. It is sometimes possible to achieve an acceptable, slightly less conservative, design by checking each possible chain with a tensile force obtained by subtracting the force removed by each bolt ahead of that chain, i.e., closer to mid-length of the member, from the full tensile force in the member. This approach
assumes that the full force is transferred equally by all bolts at one end.

The resistance to fracture at a net section now includes a reduction factor, \( U \), to account for shear lag.

Members and splices subjected to axial tension shall be investigated for two conditions:

- yield on the gross section, i.e., Equation 12.3-1, and
- fracture on the net section, i.e., Equation 12.3-2.

The determination of the net section requires consideration of:

- the gross area from which deductions will be made, or reduction factors applied, as appropriate
- deductions for all holes in the design cross-section,
- correction of the bolt hole deductions for the stagger rule,
- application of the reduction factor \( U \), specified below to account for shear lag, and
- application of an 85% maximum area efficiency factor for splice plates and other splicing elements.

The factored tensile resistance, \( P_r \), shall be taken as the lesser of the values given by Equations 12.3-1 and 12.3-2.

\[
P_r = \varphi_y P_{ny} = \varphi_y F_y A_g \quad (12.3-1)
\]

\[
P_r = \varphi_u P_{nu} = \varphi_u F_u A_n U \quad (12.3-2)
\]

where:

- \( P_{ny} \) = nominal tensile resistance for yielding in gross section (N)
- \( F_y \) = yield strength (MPa)
- \( A_g \) = gross cross-sectional area of the member (mm\(^2\))
- \( P_{nu} \) = nominal tensile resistance for fracture in net section (N)
- \( F_u \) = tensile strength (MPa)
- \( A_n \) = net area of the member as described above (mm\(^2\))
- \( U \) = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements, and as specified below for other cases.

Lecture - 12-11
The reduction factor, $U$, does not apply when checking yielding on the gross section because yielding tends to equalize the non-uniform tensile stresses caused over the cross-section by shear lag.

Due to strain hardening, a ductile steel loaded in axial tension can resist a force greater than the product of its gross area and its yield strength prior to fracture. However, excessive elongation due to uncontrolled yielding of gross area not only marks the limit of usefulness, it can precipitate failure of the structural system of which it is a part. Depending on the ratio of net area to gross area and the mechanical properties of the steel, the component can fracture by failure of the net area at a load smaller than that required to yield the gross area. General yielding of the gross area and fracture of the net area both constitute measures of component strength. The relative values of the resistance factors for yielding and fracture reflect the different reliability indices deemed proper for the two modes.

The part of the component occupied by the net area at fastener holes generally has a negligible length relative to the total length of the member. As a result, the strain hardening is quickly reached and, therefore, yielding of the net area at fastener holes does not constitute a strength limit of practical significance, except, perhaps, for some built-up members of unusual proportions.

For welded connections, $A_n$ is the gross section less any access holes in the connection region.

In the absence of more refined analysis or tests, the reduction factors below may be used to account for shear lag in connections.

For bolted connections, the following values of $U$ may be used:

- For rolled I-shapes with flange widths not less than two-thirds the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than three fasteners per line in the direction of stress,

$$U = 0.90$$
• For all other members having no fewer than three fasteners per line in the direction of stress,

$$U = 0.85$$

• For all members having only two fasteners per line in the direction of stress,

$$U = 0.75$$

When a tension load is transmitted by fillet welds to some, but not all, elements of a cross-section, the weld strength shall control.

A component subjected to tension and flexure must satisfy Equations 12.3-3 and 12.3-4.

If \( \frac{P_u}{P_r} < 0.2 \), then

\[
\frac{P_u}{2.0P_r} \% \left( \frac{M_{ux}}{M_{rx}} \% \frac{M_{uy}}{M_{ry}} \right) \# 1.0
\]

(12.3-3)

If \( \frac{P_u}{P_r} \geq 0.2 \), then

\[
\frac{P_u}{P_r} \% 8.0 \left( \frac{M_{ux}}{M_{rx}} \% \frac{M_{uy}}{M_{ry}} \right) \# 1.0
\]

(12.3-4)

where:

- \( P_r \) = factored tensile resistance (N)
- \( M_{rx}, M_{ry} \) = factored flexural resistances about the x and y axes, respectively (N@m)
- \( M_{ux}, M_{uy} \) = moments about the x and y axes, respectively, resulting from factored loads (N@m)
- \( P_u \) = axial force effect resulting from factored loads (N)

Interaction equations in tension and compression members are a design simplification. Such equations involving exponents of 1.0 on the moment ratios are usually conservative. More exact, nonlinear interaction curves are also available and are discussed in Galambos (1988). If these interaction equations are used, additional investigation of service limit state stresses is necessary to avoid premature yielding.
The stability of a flange subjected to a net compressive stress due to the tension and flexure shall be investigated for local buckling.

12.4 COMPRESSIVE RESISTANCE

The axial compressive load, $P_u$, and concurrent moments, $M_{ux}$ and $M_{uy}$, calculated for the factored loadings by elastic analytical procedures must satisfy the following relationships:

If $\frac{P_u}{P_r} < 0.2$, then

$$\frac{P_u}{2.0P_r} \left( \frac{M_{ux}}{M_{rx}} \frac{M_{uy}}{M_{ry}} \right) \# 1.0$$

(12.4-1)

If $\frac{P_u}{P_r} \geq 0.2$, then

$$\frac{P_u}{P_r} \left( \frac{M_{ux}}{M_{rx}} \frac{M_{uy}}{M_{ry}} \right) \# 1.0$$

(12.4-2)

$P_r$ = factored compressive resistance (N)

$M_{rx}$ = factored flexural resistance about the x axis (N@m)

$M_{ry}$ = factored flexural resistance about the y axis (N@m)

$M_{ux}$ = factored flexural moment about the x axis calculated as specified below (N@m)

$M_{uy}$ = factored flexural moment about the y axis calculated as specified below (N@m)

$M_{ux}$ and $M_{uy}$, moments about axes of symmetry, may be determined by either:

- a second order elastic analysis that accounts for the magnification of moment caused by the factored axial load, or
- the approximate single step adjustment specified in Article S4.5.3.2b.

These equations are identical to the provisions in the AISC LRFD Specification (1986). They were selected for use in that Specification after comparing it and a number of alternative formulations with the results of refined inelastic analyses of 82 frame sidesway cases, Kanchanalai (1977). $P_u$, $M_{ux}$ and $M_{uy}$ are simultaneous axial and flexural forces on cross-sections determined by analysis under factored loads. The maximum calculated moment
in the member in each direction should be considered, including the second order effects. Where maxima occur on different cross-sections, each should be checked.

12.5 NON-COMPOSITE COMPRESSION MEMBERS

New provisions for compressive resistance, adopted from AISC LRFD Specification, are contained in Article S6.9.4.1.

For members that satisfy the width/thickness requirements specified in Table 12.5-1, the nominal compressive resistance, \( P_n \), is taken as:

\[
\text{If } \lambda \# 2.25, \text{ then } P_n = 0.66 \lambda F_y A_s \tag{12.5-1}
\]

\[
\text{If } \lambda > 2.25, \text{ then } P_n = \frac{0.88 F_y A_s}{\lambda} \tag{12.5-2}
\]

for which:

\[
\lambda' \left( \frac{K R}{r_s \pi} \right)^2 \frac{F_y}{E}
\]

where:

\[
A_s = \text{ gross cross-sectional area (mm}^2)\]

\[
F_y = \text{ yield strength (MPa) } \]

\[
E = \text{ modulus of elasticity (MPa) } \]

\[
K = \text{ effective length factor } \]

\[
R = \text{ unbraced length (mm) } \]

\[
r_s = \text{ radius of gyration about the plane of buckling (mm) } \]

For steel piles under axial load only, \( P_n \) shall not exceed the following:

- for H-Piles ........................................... 0.78 \( F_y A_s \)
- for pipe piles ....................................... 0.87 \( F_y A_s \)

The limits on \( P_n \) for steel piles under only axial load are intended to account for unintended eccentricities. The use of these reduced axial capacities is consistent with the resistance factors specified in Section S12.
Pile Damage Factors - Further Reduction (see Article C10.5.5)

- Easy Driving Anticipated ................................ 1.00
- Moderate Driving Anticipated ........................... 0.875
- Difficult Driving Anticipated ............................. 0.75

These equations are identical to the column design equations of the AISC LRFD Specification (1986). Both are essentially the same as column strength curve 2P of SSRC (1988). They incorporate an out-of-straightness criterion of L/1500. The development of the mathematical form of these equations is described in Tide (1985), and the structural reliability they are intended to provide is discussed in SSRC (1988).

Width/Thickness ratios for plates, tubes and built-up members are to satisfy:

\[
\frac{b}{t} \# k \sqrt{\frac{E}{F_y}} \tag{12.5-3}
\]

where:

- \(k\) = plate buckling coefficient as specified in Table 12.5-1
- \(b\) = width of plate as specified in Table 12.5-1 (mm)
- \(t\) = plate thickness (mm)

Wall thickness of tubes shall satisfy:

- for circular tubes: \[\frac{D}{t} \# 2.8 \sqrt{\frac{E}{F_y}} \tag{12.5-4}\]
- for rectangular tubes: \[\frac{b}{t} \# 1.7 \sqrt{\frac{E}{F_y}} \tag{12.5-5}\]

where:

- \(D\) = diameter of tube (mm)
- \(b\) = width of face (mm)
- \(t\) = thickness of tube (mm)
Table 12.5-1 - Limiting Width-Thickness Ratios

<table>
<thead>
<tr>
<th>Plates Supported Along One Edge</th>
<th>k</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flanges and Projecting Legs or Plates</td>
<td>0.56</td>
<td>- Half-flange width of I-sections</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Full-flange width of channels</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Distance between free edge and first line of bolts or welds in plates</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Full-width of an outstanding leg for pairs of angles in continuous contact</td>
</tr>
<tr>
<td>Stems of Rolled Tees</td>
<td>0.75</td>
<td>- Full-depth of tee</td>
</tr>
<tr>
<td>Other Projecting Elements</td>
<td>0.45</td>
<td>- Full-width of outstanding leg for single angle strut or double angle strut with separator</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Full projecting width for others</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plates Supported Along Two Edges</th>
<th>k</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Box Flanges and Cover Plates</td>
<td>1.40</td>
<td>- Clear distance between webs minus inside corner radius on each side for box flanges</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Distance between lines of welds or bolts for flange cover plates</td>
</tr>
<tr>
<td>Webs and Other Plate Elements</td>
<td>1.49</td>
<td>- Clear distance between flanges minus fillet radii for webs of rolled beams</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Clear distance between edge supports for all others</td>
</tr>
<tr>
<td>Perforated Cover Plates</td>
<td>1.86</td>
<td>- Clear distance between edge supports</td>
</tr>
</tbody>
</table>
12.6 COMPOSITE COMPRESSION MEMBERS

New provisions for axially loaded composite columns are contained in Article S6.9.5.1. The provisions of Article S6.12.2.3 apply to composite columns in flexure. These types of members are used infrequently in bridges and, hence, are not covered here in detail.

12.7 I-SECTIONS IN FLEXURE

12.7.1 General

These provisions apply to flexure of rolled or fabricated straight steel I-sections, symmetrical about the vertical axis in the plane of the web.

Hybrid sections consisting of a web with a specified minimum yield strength lower than one or both flanges may be designed under these specifications. Sections with a higher strength steel in the web than in the flanges are permitted, but should not be considered hybrid sections.

The provisions of these articles apply to:

• composite and non-composite compact sections,
• composite and non-composite non-compact sections.

Flexural members are to be designed for:

• the strength limit state,
• the service limit state control of permanent deflections summarized in Article 12.2.1,
• the fatigue and fracture limit state requirements for details and the fatigue requirements for webs as summarized in Article 12.2.2,
• constructibility as specified in Article S6.10.3.2.

Provisions now require that, generally, the yield strength of steel may not exceed 485 MPa or, where plastic capacity is used, 345 MPa. For steels exceeding 345 MPa, moment redistribution provisions of Article S6.10.4.4 and inelastic analysis procedures of Article S6.10.10 are not permitted.

New limits on weak axis moment of inertia of the compression flange of a steel section are provided. Flexural components shall be proportioned such that:
0.1 \# \frac{I_{yc}}{I_y} \# 0.9 \quad (12.7.1-1)

where:

\[ I_y = \text{moment of inertia of the steel section about the vertical axis in the plane of the web (mm}^4) \]

\[ I_{yc} = \text{moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (mm}^4) \]

The ratio of \( I_{yc}/I_y \) determines the location of the shear center of a singly symmetric section. Girders with ratios outside of the limits specified behave more like a "T" section with the shear center located at the intersection of the larger flange and the web.

Provisions for web slenderness, compression flange slenderness and compression flange bracing requirements are essentially the same as contained in the AASHTO Load Factor Design Specification.

Web slenderness ratios for compact-composite members are now based on the depth of the web in compression at the plastic moment rather than the total web depth.

All webs are to be proportioned such that:

\[ \frac{2D_c}{t_w} \# 6.77 \sqrt{\frac{E}{f_c}} \# 200 \quad \text{without longitudinal stiffeners} \]

\[ \quad (12.7.1-2) \]

\[ \frac{2D_c}{t_w} \# 13.54 \sqrt{\frac{E}{f_c}} \# 400 \quad \text{with longitudinal stiffeners} \]

\[ (12.7.1-3) \]

where:

\[ D_c = \text{depth of the web in compression in the elastic range (mm)} \]

\[ f_c = \text{stress in the compression flange due to the factored loading under investigation (MPa)} \]

**Flange Proportions:**

Compression flanges shall be proportioned such that:
\[ b_f \geq 0.3D_c \]  \hspace{1cm} (12.7.1-4)

where:

\[ b_f \quad = \quad \text{width of the compression flange (mm)} \]

Tension flanges shall be proportioned such that:

\[ \frac{b_t}{2t_t} \leq 12.0 \]  \hspace{1cm} (12.7.1-5)

The limit on compression flanges acts to control bend buckling of the web and the preferable limit is actually 0.4 \( D_c \). Furthermore, it is desirable to proportion the compression flange thickness to be greater than 1.5 times the web thickness.

The limit on the tension flange thickness acts to limit distortion of the flange induced by welding.

The yield moment, \( M_y \), of a composite section is needed only for the strength limit state investigation of the following types of composite sections:

- Compact positive bending sections in continuous spans - Article S6.10.4.2.2a.
- Negative bending sections - designed by the alternative formula Article S6.10.4.2.3.
- Hybrid negative bending sections for which the neutral axis is more than 10\% of the web depth from mid-depth of the web - Article S6.10.4.3.1c.
- Compact homogeneous sections with stiffened webs subjected to combined moment and shear values exceeding specified limits - Article S6.10.7.3.3a.
- Non-compact sections used at the last plastic hinge location in inelastic designs - Article S6.10.10.1.2c.

The yield moment, \( M_y \), of a composite section is determined as the sum of the moments applied separately to the steel, short-term and long-term composite sections to cause first yielding in either steel flange when any web yielding in hybrid sections is disregarded.

A procedure for calculating the yield moment is presented in Appendix A of the LRFD Specification.

\( M_p \) shall be calculated as the first moment of plastic forces about the plastic neutral axis. Plastic forces in steel portions of a cross-section shall be calculated using the yield strengths of the flanges, the web and reinforcing steel, as appropriate. Plastic forces in concrete portions of the cross-section which are in compression
may be based on the rectangular stress block as specified in Article S5.7.2.2. Concrete in tension shall be neglected.

The position of the plastic neutral axis is determined by the equilibrium condition that there is no net axial force. The plastic moment of a composite section in positive flexure can be calculated by the following procedure:

1. Calculate the element forces and use them to determine whether the plastic neutral axis is in the web, top flange, or slab,
2. Calculate the location of the plastic neutral axis within the element determined in the first step,
3. Sum moments about the plastic neutral axis to calculate $M_p$.

Equations for the five cases most likely to occur in practice are given in Appendix A of the LRFD Specification.

The forces in the longitudinal reinforcement may be conservatively neglected. To do this, set $P_{rb}$ and $P_{rt}$ equal to 0 in the equations in Appendix A of the LRFD Specification.

The plastic moment of a composite section in negative flexure can be calculated by an analogous procedure.

The yield moment, $M_y$, of a non-composite section is determined as the moment required to cause first yielding in either flange when any web yielding in hybrid sections is disregarded.

The plastic moment, $M_p$, of a non-composite section is determined as the resultant moment of the fully plastic stress distribution acting on the section.

The plastic moment of non-composite sections may be calculated by eliminating the terms pertaining to the concrete slab and longitudinal reinforcement from the equations in the Specification, Appendix A, for composite sections.

The depth of web in compression at the plastic moment shall be determined as:

If: $F_{yw} A_w \$ *F_{yc} A_c \& F_{yt} A_t^*$, then

$$D_{cp} = \frac{D}{2A_w F_{yw}} \left( F_{yt} A_t \% F_{yw} A_w \& F_{yc} A_c \right) \quad (12.7.1-6)$$

otherwise:

$$D_{cp} = D \quad (12.7.1-7)$$

where:
D = web depth (mm)

\( A_t = \) area of the tension flange (mm\(^2\))

\( A_c = \) area of the compression flange (mm\(^2\))

\( A_w = \) area of the web (mm\(^2\))

\( F_{yt} = \) specified minimum yield strength of the tension flange (MPa)

\( F_{yc} = \) specified minimum yield strength of the compression flange (MPa)

\( F_{yw} = \) specified minimum yield strength of the web (MPa)

If the inequality is satisfied, the neutral axis is in the web. If it is not, the neutral axis is in the flange and \( D_{cp} \) is equal to the depth of the web.

For continuous spans with compact sections in positive moment areas and non-compact sections at interior supports, new restrictions on the positive moment resistance are contained in Article S6.10.4.2.2a and are summarized below.

### 12.7.2 Compact Composite Sections

The design provisions for compact composite sections are summarized in Figure 12.7.2-1 and 12.7.2-2.

Compact section provisions are not applicable to steel with \( F_y \) greater than 345 MPa since their inelastic rotation capacity is not fully established. For steels with \( F_y \) greater than 345 MPa, the provisions for non-compact sections are applicable.
For composite sections in positive flexure, Articles 6.10.4.1.3, 6.10.4.1.4, 6.10.4.1.6a, 6.10.4.1.7 and 6.10.4.1.9 are considered to be automatically satisfied.
Figure 12.7.2-2 - I-Sections in Flexure Flow Chart When the Optional Q Formula is Considered

For composite sections in positive flexure, Articles 6.10.4.1.3, 6.10.4.1.4, 6.10.4.1.6a, 6.10.4.1.7 and 6.10.4.1.9 are considered to be automatically satisfied.
Some further explanation of the requirements for positive flexure is needed. For simple spans and continuous spans with compact interior support sections, the nominal positive flexural resistance is:

- If $D_p > D'$, then:
  \[
  M_n = \frac{M_p}{D_p} \]
  (12.7.2-1)

- If $D' < D_p < 5D'$, then:
  \[
  M_n' = \frac{5}{4} M_p \left\{ 0.85 M_y \left( \frac{D_p}{D} \right) + 0.85 M_y \left( \frac{D}{D_p} \right) \right\} \]
  (12.7.2-2)

where:

- $D_p$ = distance from the top of the slab to the neutral axis at the plastic moment (mm)
- $D'$ = depth at which a composite section reaches its theoretical plastic moment capacity when the maximum strain in the concrete slab is at its theoretical crushing strain (mm) which are to be approximated as $\beta (d + t_s + t_h)/7.5$
- $\beta = 0.90$ where $F_y = 250$ MPa
  - $\beta = 0.70$ where $F_y = 345$ MPa
  - $\beta = 0.70$ where $F_y = 485$ MPa
- $M_y$ = moment capacity at first yield of the long-term composite positive moment section, N@m

For the purpose of the calculation of $M_n$ using Equation 2, $M_y$ may be taken as product of the long-term composite section modulus with respect to the tension flange and the yield strength of the steel.

For continuous spans with non-compact interior support sections, the nominal flexural resistance may be determined by either Method "A" or Method "B", specified herein, but shall not be taken greater than the applicable value of $M_n$ computed from either Equation 1 or Equation 2.

- Method "A" - Approximate Analysis:
  \[
  M_n = 1.3 R_h M_y \]
  (12.7.2-3)

- Method "B" - More Refined Analysis:
  \[
  M_n = R_h M_y \%A * M_{np} \& M_{cp} \]
  (12.7.2-4)

for which:
$M_{np} - M_{cp}$ for interior spans shall be taken as the smaller value at either end of the span.

where:

$M_{np} =$ nominal flexural resistance at an interior support (N\(\cdot\)m)

$M_{cp} =$ moment due to the factored loadings at an interior support concurrent with the maximum positive flexural moment at the cross-section under consideration (N\(\cdot\)m)

$A =$ for end spans, the distance from the end support to the location of the cross-section in the span divided by the span length. For interior spans, $A$ shall be taken as 1.0

$M_y =$ yield moment (N\(\cdot\)m)

$R_h =$ flange-stress reduction factor from Article 12.7.3

The ductility requirement herein is equivalent to the requirement given in the AASHTO (1995). The ratio of $D_p$ to $D'$ is limited to a value of 5.0 to ensure that the tension flange of the steel section reaches strain hardening prior to crushing of the concrete slab. $D'$ is defined as the depth at which the composite section reaches its theoretical plastic moment capacity $M_p$ when the maximum strain in the concrete slab is at its theoretical crushing strain. The term $(d+t_s+t_h)/7.5$ in the definition of $D'$, hereafter referred to as $D^*$, was derived by assuming that the concrete slab is at the theoretical crushing strain of 0.3% and the tension flange is at the assumed strain-hardening strain of 1.2%.

From the results of a parametric analytical study of over 400 different composite steel sections, including unsymmetrical, as well as symmetrical steel sections. It was determined that sections utilizing 250 MPa steel reached $M_p$ at a ratio of $D_p/D^*$ equal to approximately 0.9, and sections utilizing 345 MPa steel reached $M_p$ at a ratio of $D_p/D^*$ equal to approximately 0.7. Thus, 0.9 and 0.7 are specified as the values to use for the $\beta$ factor, which is multiplied by $D^*$ to compute $D'$ for 250 MPa and 345 MPa yield strength steels.

Compression flange slenderness and bracing should not be investigated for the strength limit state of composite sections in positive flexure.

Where Equation 12.7.2-5 is used, the concurrent positive flexural moment shall not exceed $R_h M_y$ for the factored loading which produces the maximum negative flexural moment at the adjacent support.
Alternatively, the actual effects of the positive flexural yielding can be accounted for by using the inelastic procedures specified in Article S6.10.10.1.

The shape factor \((M_p/M_y)\) for composite sections in positive bending can be as high as 1.5. Therefore, a considerable amount of yielding is required to get from \(M_y\) to \(M_p\), and this yielding reduces the effective stiffness of the section. In continuous spans, the reduction in stiffness can shift moment from positive flexural regions to negative flexural regions. Therefore, the actual moments in negative flexural regions may be higher than those predicted by an elastic analysis. Negative flexural sections would have to have the capacity to sustain these higher moments, unless some limits were placed on the extent of yielding of the positive moment section. This latter approach is used in the Specification.

The live loading patterns causing the maximum elastic moments in negative flexural sections are different from those causing maximum moments in positive flexural sections. When the loading pattern causing maximum positive flexural moments are applied, the concurrent negative flexural moments are usually well below the flexural resistance of the sections in those regions. Therefore, the specifications conservatively allow additional moment above \(M_y\) to be applied to positive flexural sections of continuous spans with non-compact interior support sections. Compact interior support sections have sufficient capacity to sustain the higher moments caused by the reduction in stiffness of the positive flexural region. Thus, the nominal flexural resistance of positive flexural sections in members with compact interior support sections is specified to be \(M_p\) with no additional limit.

New alternative formulas for flexural resistance given below apply to constant depth members without longitudinal stiffeners in negative flexure that exceed either the compression flange slenderness requirements specified in Articles S6.10.4.1.3, S6.10.4.1.4, or S6.10.4.1.1, or the web slenderness requirements specified in Article S6.10.4.1.2 or S6.10.4.1.1.

The nominal flexural resistance, \(M_n\), of sections satisfying the requirements of Articles S6.10.4.1.8 web and slenderness, S6.10.4.1.7 for compression flange bracing is taken as the lesser of:

\[
M_n = M_p, \text{ or } (12.7.2-5)
\]

\[
M_n' = 1 \& \left( 1 \& \frac{0.7}{(M_p/M_y)} \right) \left( \frac{Q_p \& Q_{fl}}{Q_p \& 0.7} \right) M_p \quad (12.7.2-6)
\]

for which:

\[
Q_p = 5.47(M_p/M_y) - 3.13 \text{ for unsymmetrical sections}
\]
If \( \frac{b_f}{2t_f} \geq 0.382 \sqrt{\frac{E}{F_{yc}}} \), then

\[
Q_{ll'} = \frac{30.5}{\sqrt{2D_{cp}/t_w}}
\]

(12.7.2-7)

Otherwise:

\[
Q_{ll'} = \frac{4.45}{\left( \frac{b_f}{2t_f} \right)^2 \sqrt{\frac{2D_{cp}/t_w}{F_{yc}}}}
\]

(12.7.2-8)

where:

\[
M_p = \text{plastic moment (N@m)}
\]

\[
M_y = \text{yield moment (N@m)}
\]

\[
F_{yc} = \text{specified minimum yield strength of the compression flange (MPa)}
\]

The nominal flexural resistance given by Equation 12.7.2-6 is based on the inelastic buckling strength of the compression flange, and results from a fit to available experimental data. The equation considers the interaction of the web and compression flange slenderness in the determination of the resistance of the section by using a flange buckling coefficient, \( k_f = 4.92/(2D_{cp}/t_w)^{1/2} \), in computing the \( Q_{ll'} \) parameter in Equation 12.7.2-8. \( Q_{ll'} \) is the ratio of the buckling capacity of the flange to the yield strength of the flange. The buckling coefficient given above was based on the test results reported in Johnson (1985), as well as data from other available composite and non-composite steel beam tests. A similar buckling coefficient is given in Section B5.2 and Appendix B5.2 of AISC (1989). Equation 12.7.2-7 is specified to compute \( Q_{ll'} \) if the compression flange slenderness is less than the value specified in Article S6.10.4.1.3 to effectively limit the increase in the bending resistance at a given web slenderness with a reduction in the compression flange slenderness below this value. Equation 12.7.2-7 is obtained by substituting the compression flange slenderness limit from Article S6.10.4.1.3 in Equation 12.7.2-8.

Equation 12.7.2-6 represents a linear fit of the experimental data between a flexural resistance of \( M_p \) and \( 0.7 M_y \). The \( Q_p \) parameter, defined as the web and compression flange slenderness to reach a flexural resistance of \( M_p \), was derived to ensure the
equation yields a linear fit to the experimental data. Equation 12.7.2-6 was derived to determine the maximum flexural resistance and does not necessarily ensure a desired inelastic rotation capacity. Sections in negative flexure which are required to sustain plastic rotations may be designed according to the procedures specified in Article S6.10.10. If elastic procedures are used and Equation 12.7.2-6 is not used to determine the nominal flexural resistance, the resistance shall be determined according to the procedures specified in Article S6.10.5.3.3 for negative moment regions of non-compact composite sections.

Compression flanges of sections providing the flexural resistance above must satisfy:

\[
\frac{b_f}{2t_f} \geq 2.52 \left( \frac{E}{F_{yc}} \right) \left( \frac{D_{cp}}{t_w} \right) \quad (12.7.2-9)
\]

12.7.3 Non-Compact Composite Sections

New and expanded provisions for web and compression flange slenderness, and for compression flange bracing are contained in Articles 6.10.2.2, 6.10.4.1.4 and 6.10.4.1.9.

The specified web slenderness limit corresponds to the upper limit for transversely stiffened webs in the AASHTO (1992). This limit defines an upper bound below which fatigue due to excessive lateral web deflections is not a consideration, Yen and Mueller (1966), and Mueller and Yen (1968).

The specified web slenderness limit for longitudinally stiffened webs is retained from the Allowable Stress Design portion of the AASHTO (1992). Since the flange-stress reduction factor, \(R_b\), specified in Article S6.10.4.3.2a is assumed to be 1.0 for longitudinally stiffened sections, local bend-buckling of the web needs to be prevented. Using an elastic plate buckling coefficient, \(k\), equal to 129.4 for longitudinally stiffened webs yields an allowable web slenderness slightly below that specified in Equation 12.7.3-2, Dubas (1948). However, flanges provide additional rotational restraint to the web so the use of the higher value is justified.

The limit on compression flange slenderness includes the effect of the web slenderness.

The limit on \(b/2t\) is the largest ratio for which the flanges can be expected to reach yield, subject to various factors, before local buckling.

The compression flange bracing requirement defines the maximum unbraced length for which a composite negative flexural
section can reach the specified minimum yield strength times the applicable flange stress reduction factors, under a uniform moment, before the onset of lateral torsional buckling. Under a moment gradient, sections with larger unbraced lengths can still reach the yield strength. This larger allowable unbraced length may be determined by setting Equation 12.7.3-9 equal to \( R_b R_h F_{yc} \), and solving the equation for \( L_b \) using the calculated value of the moment gradient factor \( C_b \) given herein.

The nominal flexural resistance of the compression flange, in terms of stress, shall be determined as:

- For composite sections in positive flexure in their final condition:
  \[
  F_n = R_b R_h F_{yc}
  \]
  (12.7.3-1)

- For all other sections in their final condition and for constructibility:
  \[
  F_n = R_b R_h F_{cr}
  \]
  (12.7.3-2)

where:

\[
F_{cr} = \frac{1.904E}{\left(\frac{b_f}{2t_f}\right)^2 \left(\frac{2D_c}{t_w}\right)} \leq F_{yc}
\]

without longitudinal web stiffeners

\[
F_{cr} = \frac{0.166E}{\left(\frac{b_f}{2t_f}\right)^2} \leq F_{yc}
\]

with longitudinal web stiffeners

where:

- \( b_f \) = width of the compression flange (mm)
- \( D_c \) = depth of the web in compression in the elastic range (mm)
- \( R_h \) = hybrid factor
- \( R_b \) = load-shedding factor
- \( F_{yc} \) = specified minimum yield strength of the compression flange (MPa)
The nominal flexural resistance of the tension flange, in terms of stress, shall be determined as:

\[ F_n = R_h R_{hy} F_{yt} \]  \hspace{1cm} (12.7.3-3)

where:

- \( F_{yt} \) = specified minimum yield strength of the tension flange (MPa)

**Hybrid Factor**

The hybrid factor accounts for the nonlinear variation of stresses caused by yielding of the lower strength steel in the web of a hybrid beam. The formulas defining this factor are the same as those given in the AASHTO (1992), and are based on experimental and theoretical studies of composite and non-composite beams and girders, ASCE (1968), Schilling (1968), and Schilling and Frost (1964). The factor applies to non-compact sections in both shored and unshored construction.

For homogeneous sections, \( R_h \) shall be taken as 1.0.

For hybrid sections in which the stress in both flanges under the factored loading does not exceed the yield strength of the web, the hybrid factor \( R_h \) shall be taken as 1.0.

The reduction factor should not be applied to compact sections because the effect of the lower-strength material in the web is accounted for in calculating the plastic moment as shown below.

For positive flexural resistance of composite hybrid sections in positive flexure, the hybrid reduction factor shall be taken as:

\[ R_h' = 1 \& \left[ \frac{\beta \psi (1 \& \rho)^2 (3 \& \psi \% \rho \psi)}{6 \% \beta \psi (3 \& \psi)} \right] \]  \hspace{1cm} (12.7.3-4)

where:

- \( \rho \) = \( \frac{F_{yw}}{F_{yb}} \)
- \( \beta \) = \( \frac{A_w}{A_{fb}} \)
- \( \psi \) = \( \frac{d_n}{d} \)
- \( d_n \) = distance from outer fiber of bottom flange to neutral axis of transformed short-term composite section (mm)
- \( d \) = depth of steel section (mm)
For negative flexure, where the neutral axis of composite hybrid sections, determined as specified in Article S6.10.3.1.4a, is located within 10% of the web depth from mid-depth of the web, the hybrid factor is:

\[ R_h' = \frac{12 \% \beta (3 \rho \beta^3)}{12 \% 2\beta} \]  

(12.7.3-5)

where:

\[ \rho = \frac{F_{yw}}{f_{fl}} \]

\[ \beta = \frac{2A_w}{A_{tf}} \]

\[ f_{fl} = \text{lesser of either the specified minimum yield strength, or the stress due to the factored loading in either flange (MPa)} \]

\[ A_{tf} = \text{total area of both steel flanges and the longitudinal reinforcement included in the section (mm}^2) \]

For other composite hybrid sections in negative flexure, the hybrid factor shall be taken as:

\[ R_h' = \frac{M_{yr}}{M_y} \]  

(12.7.3-6)

where:

\[ M_y = \text{yield resistance in terms of moment, when web yielding is disregarded (N\text@m)} \]

\[ M_{yr} = \text{yield resistance in terms of moment, when web yielding is accounted for (N\text@m)} \]

The approximate method illustrated in Figure 12.7.3-1 may be used in determining the yield moment resistance, \( M_{yr} \), when web yielding is accounted for. The solid line connecting \( F_{yf} \) with \( f_r \) represents the distribution of stress at \( M_y \) if web yielding is neglected. For unshored construction, this distribution can be obtained by first applying the proper permanent load to the steel section, then applying...
the proper permanent load and live load to the composite section, and combining the two stress distributions. The dashed lines define a triangular stress block whose moment about the neutral axis is subtracted from \( M_y \) to account for the web yielding at a lower stress than the flange. \( M_y \) may be determined as specified earlier. Thus,

\[
M_{yr} \ll M_y - aP \quad (12.7.3-7)
\]

for which:

\[
P \ll \frac{(F_{yf} & F_{yw})}{2} t_w h_w \quad (12.7.3-8)
\]

where:

\[F_{yf} = \text{specified minimum yield strength of the bottom flange (MPa)}\]
\[F_{yw} = \text{specified minimum yield strength of the web (MPa)}\]
\[t_w = \text{web thickness (mm)}\]
\[a = \text{distance from the elastic neutral axis to the centroid of the stress block (mm)}\]
\[f_r = \text{stress in reinforcing steel at } M_y \text{ (MPa)}\]
\[h_w = \text{depth of yielded web (mm)}\]

Figure 12.7.3-1 is specifically for the case where the elastic neutral axis is above mid-depth of the web and web yielding occurs only below the neutral axis. However, the same approach can be used if web yielding occurs both above and below the neutral axis or only above the neutral axis. The moment due to each triangular stress block due to web yielding must be subtracted from \( M_y \).

This approach is approximate because web yielding causes a small shift in the location of the neutral axis. The effect of this shift on \( M_{yr} \) is almost always small enough to be neglected. The exact value of \( M_{yr} \) can be calculated from the stress distribution by accounting for yielding, Schilling (1968).
Figure 12.7.3-1 - Stress Distribution in Hybrid Girder

**Load Shedding Factors, $R_b$ for Compression Flanges**

This factor accounts for the nonlinear variation of stresses caused by local buckling of slender webs subjected to flexural stresses. It does not apply if a suitable longitudinal stiffener is provided to control this local buckling, or if the web slenderness is below $\lambda_b (E/f_c)^{1/2}$, which corresponds to the theoretical elastic bend-buckling stress of the web. The formula defining this factor is based on extensive experimental and theoretical studies, SSRC (1988). Equation 12.7.3-7 represents the exact formulation for the factor $R_b$ given by Basler (1961).

The value of $\lambda_b$ reflects different assumptions of support provided to the web by the flanges. The value for members with a compression flange area less than the tension flange area is based on the theoretical elastic bend-buckling coefficient, $k$, of 23.9 for simply-supported boundary conditions at the flanges. The value for members with a compression flange area equal to or greater than the tension flange area is based on a value of $k$ between the value for simply-supported boundary conditions and the theoretical $k$ value of 39.6 for fixed boundary conditions at the flanges, Timoshenko and Gere (1961).

**Load-Shedding Factor, $R_b$**

**Compression Flanges**

If:

$$\frac{2D_c}{t_w} \#\lambda_b \sqrt{\frac{E}{f_c}}$$  \hspace{1cm} (12.7.3-9)

Or if one or two longitudinal web stiffeners are provided and:
\[
\frac{D}{t_w} #1.01 \sqrt[ ]{\frac{E_k}{f_c}} \tag{12.7.3-10}
\]

Then, \( R_b \) shall be taken as 1.0.

Otherwise,

\[
R_b' = 1 &\left( \frac{a_r}{1200 \% 300 a_r} \right) \left( \frac{2D_c \lambda_b a_r}{t_w} \sqrt[ ]{\frac{E}{f_c}} \right) \tag{12.7.3-11}
\]

for which:

If \( \frac{d_s}{D_c} \geq 0.4 \) then:

\[
k' = 5.17 \left( \frac{D}{d_s} \right)^2 \leq 9.0 \left( \frac{D}{D_c} \right)^2 \leq 7.2 \tag{12.7.3-12}
\]

If \( \frac{d_s}{D_c} < 0.4 \) then:

\[
k' = 11.64 \left( \frac{D}{D_c \lambda_d} \right)^2 \leq 9.0 \left( \frac{D}{D_c} \right)^2 \leq 7.2 \tag{12.7.3-13}
\]

\[
a_r' = \frac{2D_c t_w}{A_c} \tag{12.7.3-14}
\]

where:

\[
\lambda_b = \begin{cases} 5.76 & \text{for sections where } D_c \text{ is less than or equal to } D/2 \\ 4.64 & \text{for sections where } D_c \text{ is greater than } D/2 \end{cases}
\]

\[f_c = \text{stress in the compression flange due to the factored loading under investigation (MPa)}\]

\[A_c = \text{area of the compression flange (mm}^2)\]

\[d_s = \text{distance from the centerline of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener to the inner surface or leg of the compression-flange element (mm)}\]
\(D_c\) = depth of the web in compression in the elastic range (mm)

\(R_b\) is 1.0 for tension flanges because the increase in flange stresses due to web buckling occurs primarily in the compression flange and the tension flange stress is not significantly increased by the web buckling, Basler (1961).

**Lateral-Torsional Buckling**

The nominal flexural resistance of sections in negative flexure in terms of stress, \(F_n\), is determined as the lesser of Equation 12.7.3-2 or:

If \(L_b \# 4.44 \frac{r_t}{E} \), then:

\[
F_n = C_b R_b R_h F_{yc} \left[ 1.33 + 0.187 \left( \frac{L_b}{r_t} \right) \sqrt{\frac{F_{yc}}{E}} \right] \\
\# R_b R_h F_{yc}
\]  

(12.7.3-15)

If \(L_b > 4.44 \frac{r_t}{E}\), then:

\[
F_n = C_b R_b R_h \left[ \frac{9.86E}{(L_b/r_t)^2} \right] \# R_b R_h F_{yc}
\]  

(12.7.3-16)

for which:

for unbraced cantilevers or for members where the moment within a significant portion of the unbraced segment exceeds the larger of the segment end moments:

\[C_b = 1.0\]  

(12.7.3-17)

for all other cases:

\[C_b = 1.75 - 1.05 (P_5/P_h) + 0.3(P_5/P_h)^2 \# 2.3\]  

(12.7.3-18)

where:

\[C_b = \text{moment gradient correction factor}\]

\[P_R = \text{force in the compression flange at the brace point with the lower force due to the factored loading (N)}\]
$P_h = \text{force in the compression flange at the brace point with the higher force due to the factored loading (N)}$

$L_b = \text{distance between points bracing the compression flange (mm)}$

$r_t = \text{radius of gyration of a notional section comprised of the compression flange of the steel section, plus one-third of the web in compression, taken about the vertical axis (mm)}$

$F_{yc} = \text{specified minimum yield strength of the compression flange (MPa)}$

$R_b, R_h = \text{flange-stress reduction factors discussed above}$

$(P_r/P_h)$ is negative if $P_h$ is a tensile force.

The provisions for lateral-torsional buckling of composite sections in this article differ from those specified for non-composite sections because they attempt to handle the complex general problem of lateral-torsional buckling of a constant or variable depth section with stepped flanges constrained against lateral displacement at the top flange by the composite concrete slab.

The equations provided in this article are based on the assumption that only the flexural stiffness of the compression flange will prevent the lateral displacement of that element between brace points, which ignores the effect of the restraint offered by the concrete slab, Basler and Thurlimann (1961). As such, the behavior of a compression flange in resisting lateral buckling between brace points is assumed to be analogous to that of a column. These simplified equations, developed based on this assumption, are felt to yield conservative results for composite sections under the various conditions listed above.

The effect of the variation in the compressive force along the length between brace points is accounted for by using the factor $C_b$, which has a minimum value of 1.0 when the flange compressive force, and corresponding moment, is constant over the unbraced length. As the force at one of the brace points is progressively reduced, $C_b$ becomes larger, and is taken as 1.75 when this force is 0.0. If the force at the end is then progressively increased in tension, $C_b$ continues to increase until it reaches the maximum permitted value of 2.3.

If the cross-section is constant between brace points, $M_5/M_h$ is expressed in terms of $P_5/P_h$, and may be used in calculating $C_b$. It is conservative and convenient to use the maximum moments from the moment envelope at both brace points in this ratio, or in computing $P_5/P_h$, although the actual behavior depends on the concurrent moments at these points. Under a moment gradient, $P_h$ will become...
a tensile force beyond the point of contraflexure. For this case, the ratio of \( \frac{P}{P_b} \) shall be taken as negative in computing \( C_b \).

An alternative formulation for \( C_b \) is given by the following formula, AISC (1991):

\[
C_b' = \frac{12.5P_{\text{max}}}{2.5P_{\text{max}} \%3P_A \%4P_B \%3P_c}
\]  \hspace{1cm} (12.7.3-19)

where:

\( P_{\text{max}} \) = absolute value of the maximum compression flange force in the unbraced segment (N)

\( P_A \) = absolute value of the compression flange force at the quarter point of the unbraced segment (N)

\( P_B \) = absolute value of the compression flange force at the centerline of the unbraced segment (N)

\( P_c \) = absolute value of the compression flange force at the three-quarter point of the unbraced segment (N)

This formulation gives improved results for the cases of non-linear moment gradients and moment reversal.

The effect of a variation in the lateral stiffness properties, \( r_t \), between brace points can be conservatively accounted for by using the minimum value that occurs anywhere between the brace points. Alternatively, a weighted average \( r_t \) could be used to provide a reasonable, but somewhat less conservative, answer.

The alternative formulas for flexural resistance of Article S6.10.4.2.3 applies to constant depth members without longitudinal stiffeners in negative flexure and which exceed the compression flange slenderness requirements of Article S6.10.4.1.4.

12.7.4 Compact Non-Composite Sections

Requirements regarding yield strength of steel, and provisions for web and compression flange slenderness and for compression flange bracing are the same as for the negative flexure provisions of compact composite sections.

The restriction on positive moment resistance of continuous spans with non-compact support sections is not applicable because of the lower non-composite shape factor.
**12.7.5 Non-Compact Non-Composite Sections**

Non-composite construction (as a final stage) is generally discouraged by the specification. However, provisions to evaluate the capacity of a non-composite section under dead load, prior to the curing of the deck are given in Article S6.10.4.2.6.

Requirements regarding yield strength of steel, and provisions for web and compression flange slenderness are now addressed within the general provisions for flexural resistance.

Requirements to control lateral-torsional buckling require that if the unbraced length of a compression flange exceeds $L_p$ as given below, the nominal flexural resistance based on the stability of the compression flange is determined from Equations 12.7.5-1, 12.7.5-2 or 12.7.5-3, as applicable, not to exceed the capacity given by Equation 12.7.3-2.

If either a longitudinal stiffener is provided or:

\[
\frac{2D_c}{t_w} \geq \lambda_b \sqrt{\frac{E}{F_{yc}}} \text{, then:}
\]

\[
M_{n'} = 3.14E C_b R_n \left( \frac{I_{yc}}{L_b} \right) \sqrt{0.772 \left( \frac{J}{I_{yc}} \right) \% 0.87 \left( \frac{d}{L_b} \right)^2} \tag{12.7.5-1}
\]

If a longitudinal stiffener is not provided, and if:

\[
\frac{2D_c}{t_w} > \lambda_b \sqrt{\frac{E}{F_{yc}}} \text{, then}
\]

If $L_p < L_b$, then:

\[
M_{n'} = C_b R_n R_h M_y \left[ 1 + 0.5 \left( \frac{L_p}{L_b} \right) \right] \# R_p R_h M_y \tag{12.7.5-2}
\]

If $L_b > L_r$, then:

\[
M_{n'} = C_b R_n R_h \left( \frac{L_r}{L_b} \right)^2 \# R_p R_h M_y \tag{12.7.5-3}
\]

for which:
\[ J' = \frac{D t_w^3}{3} \Sigma \frac{b f_i^3}{3} \]

\[ L_p' = 1.76 r^1 \sqrt{\frac{E}{F_{yc}}} \]

\[ L_r' = 4.44 \sqrt{\frac{l_{yc} d E}{S_{xc} F_{yc}}} \]

where:

- \( \lambda_b = 5.76 \) for members where \( D_c \leq D/2 \)
- \( \lambda_b = 4.64 \) for members where \( D_c > D/2 \)
- \( C_b \) = moment gradient correction factor given above
- \( I_{yc} \) = moment of inertia of the compression flange about the vertical axis in the plane of the web (mm\(^4\))
- \( S_{xc} \) = section modulus about the horizontal axis of the section to the compression flange (mm\(^3\))
- \( M_y \) = yield moment for the compression flange (N\(\cdot\)m)
- \( R_b \) and \( R_h \) = flange stress reduction factors as specified
- \( rN \) = minimum radius of gyration of the compression flange about the vertical axis (mm)
- \( t_w \) = thickness of web (mm)
- \( t_f \) = thickness of a flange (mm)
- \( F_{yc} \) = yield strength of compression flange (MPa)

Much of the discussion of the lateral buckling formulas in Article 12.7.3 also applies to this article. The formulas of this article are simplifications of the formulas presented in AISC (1986), and Kitipornchai and Trahair (1980) for the lateral buckling capacity of unsymmetrical girders.

The formulas predict the lateral buckling moment within approximately 10% of the more complex Trahair equations for sections satisfying the proportions specified in Article S6.10.1.1. The formulas treat girders with slender webs differently than girders with
stocky webs. For sections with stocky webs with a web slenderness less than or equal to $\lambda_b (E/F_{yc})^{1/2}$, or with longitudinally stiffened webs, bend-buckling of the web is theoretically prevented. For these sections, the St. Venant torsional stiffness and the warping torsional stiffness are included in computing the elastic lateral buckling moment given by Equation 12.7.5-1. For sections with thinner webs or without longitudinal stiffeners, cross-sectional distortion is possible; thus, the St. Venant torsional stiffness is ignored for these sections. Equation 12.7.5-3 is the elastic lateral torsional buckling moment given by Equation 12.7.5-1 with $J$ taken as 0.0.

Equation 12.7.5-2 represents a straight line estimate of the inelastic lateral buckling resistance between $R_b$, $R_{th}$, $M_y$, and $0.5 \: R_b$, $R_{th}$, $M_y$. A straight line transition similar to this is not included for sections with stocky webs or longitudinally stiffened webs because the added complexity is not justified.

A discussion of the derivation of the value of $\lambda_b$ may be found in Article SC6.10.4.3.2a.

12.7.6 Shear Resistance

12.7.6.1 GENERAL

A flow chart for shear capacity of I-sections is shown below:
12.7.6.2 UNSTIFFENED WEBS

The nominal shear resistance of unstiffened webs of hybrid and homogeneous girders shall be taken as:

If \( \frac{D}{t_w} \leq 2.46 \sqrt{\frac{E}{F_{yw}}} \), then:

\[
V_n = V_p = 0.58 F_{yw} D t_w
\]  

(12.7.6.2-1)

If \( 2.46 \sqrt{\frac{E}{F_{yw}}} < \frac{D}{t_w} \neq 3.07 \sqrt{\frac{E}{F_{yw}}} \), then:

\[
V_n = 1.48 t_w^2 \sqrt{EF_{yw}}
\]  

(12.7.6.2-2)

If \( \frac{D}{t_w} > 3.07 \sqrt{\frac{E}{F_{yw}}} \), then:

\[
V_n = \frac{4.55 t_w^3 E}{D}
\]  

(12.7.6.2-3)

where:

- \( F_{yw} \) = specified minimum yield strength of the web (MPa)
- \( D \) = web depth (mm)
- \( t_w \) = thickness of web (mm)

Equation 12.7.6.2-1 is equal to the web area times the assumed shear yield strength of \( F_{yw}/3^{1/2} \). The \( D/t_w \) limit was determined by setting the shear buckling stress, calculated assuming a buckling coefficient, \( k \), equal to 5.0, equal to the yield strength of the web, \( F_{yw} \), in Formula 35 of Cooper, et al, (1978).

When \( D/t_w > 3.07(E/F_{yw})^{1/2} \), the nominal shear resistance of the web is based on elastic shear buckling. Basler (1961) suggests taking the proportional limit as 80% of \( F_{yw} \). This corresponds to \( D/t_w = (2.46/0.8) (E/F_{yw})^{1/2} \). Thus, when \( D/t_w > 3.07(E/F_{yw})^{1/2} \), the nominal shear resistance is determined from Equation 12.7.6.2-3, which is equal to the web area times the elastic shear buckling stress given by
Formula 6 of Cooper, et al, (1978), calculated assuming a buckling coefficient, $k$, equal to 5.0.

Equation 12.7.6.2-2 represents a straight-line transition between the two web slenderness limits.

12.7.6.3 STIFFENED WEBS

The nominal shear resistance of transversely or transversely and longitudinally stiffened interior and end web panels is specified herein for homogeneous and hybrid sections. The total web depth, $D$, shall be used in determining the nominal shear resistance of web panels with longitudinal stiffeners. Transverse stiffeners shall be spaced using the maximum shear in a panel.

Longitudinal stiffeners divide a web panel into subpanels. The shear resistance of the entire panel can be taken as the sum of the shear resistance of the subpanels, Cooper (1967). However, the contribution of the longitudinal stiffener at a distance of $2D_v/5$ from the compression flange is relatively small. Thus, it is conservatively recommended that the influence of the longitudinal stiffener be neglected in computing the nominal shear resistance of the web plate.

For web panels without longitudinal stiffeners, transverse stiffeners are required if:

$$\frac{D}{t_w} > 150 \quad (12.7.6.3-1)$$

The spacing of transverse stiffeners, $d_o$, must satisfy:

$$d_o \#D \left[ \frac{260}{(\frac{D}{t_w})^2} \right] \quad (12.7.6.3-2)$$

Transverse stiffeners are required on web panels with a slenderness ratio greater than 150 in order to facilitate handling of sections without longitudinal stiffeners during fabrication and erection. The spacing of the transverse stiffeners is arbitrarily limited by Equation 12.7.6.3-2, Basler (1961). Substituting a web slenderness of 150 into Equation 12.7.6.3-2 results in a maximum transverse stiffener spacing of 3D, which corresponds to the maximum spacing permitted for web panels without longitudinal stiffeners. For higher web slenderness ratios, the maximum allowable spacing is reduced to less than 3D.

The requirement in Equation 12.7.6.3-2 is not needed for web panels with longitudinal stiffeners since maximum transverse stiffener spacing is already limited to 1.5D.
Stiffened interior web panels of homogeneous sections may develop post-buckling shear resistance due to tension-field action, Basler (1961). The action is analogous to that of the tension diagonals of a Pratt truss. The nominal shear resistance of these panels can be computed by summing the contributions of beam action and of the post-buckling tension-field action. The resulting expression is given in Equation 12.7.6.3-1, where the first term in the bracket relates to either the shear yield or shear buckling force and the second term relates to the post-buckling tension-field force.

The nominal shear resistance of interior web panels of compact sections complying with the requirements above is:

If $M_u \leq 0.5 \phi_f M_p$, then:

$$V_n' = \frac{V_p}{C} \left[ \frac{0.87(1+C)}{\sqrt{1 + \left( \frac{d_o}{D} \right)^2}} \right]$$  \hspace{1cm} (12.7.6.3-3)

If $M_u > 0.5 \phi_f M_p$, then:

$$V_n' = RV_p \left[ \frac{C}{\sqrt{1 + \left( \frac{d_o}{D} \right)^2}} \right] \# CV_p$$  \hspace{1cm} (12.7.6.3-4)

for which:

$$R' = 0.6 \% 0.4 \left( \frac{M_r & M_u}{M_r & 0.75 \phi_f M_y} \right) \# 1.0$$  \hspace{1cm} (12.7.6.3-5)

$$V_p' = 0.58 F_{yw} D t_w$$  \hspace{1cm} (12.7.6.3-6)

where:

$M_u$ = maximum moment in the panel under consideration due to the factored loads (N@m)

$V_n$ = nominal shear resistance (N)

$V_p$ = plastic shear force (N)

$M_r$ = factored flexural resistance (N@m)

$\phi_f$ = resistance factor for flexure
When both shear and flexural moment are high in a stiffened interior panel under tension-field action, the web plate must resist the shear and also participate in resisting the moment. Panels whose resistance is limited to the shear buckling or shear yield force are not subject to moment-shear interaction effects. Basler (1961) shows that stiffened web plates in non-compact sections are capable of resisting both moment and shear, as long as the shear force due to the factored loadings is less than 0.6\(\phi_v V_n\) or the flexural stress in the compression flange due to the factored loading is less than 0.75\(\phi_f F_y\).

For compact sections, flexural resistances are expressed in terms of moments rather than stresses. For convenience, a limiting moment of 0.5\(\phi_f M_p\) is defined rather than a limiting moment of 0.75\(\phi_f M_y\) in determining when the moment-shear interaction occurs by using an assumed shape factor (\(M_p/M_y\)) of 1.5. This eliminates the need to compute the yield moment to simply check whether or not the interaction effect applies. When the moment due to factored loadings exceeds 0.5\(\phi_f M_p\), the nominal shear resistance is taken as \(V_n\), given by Equation 12.7.6.3-2, reduced by the specified interaction factor, \(R\).

Both upper and lower limits are placed on the nominal shear resistance in Equation 12.7.6.3-2 determined by applying the interaction factor, \(R\). The lower limit is either the shear yield or shear buckling force. Sections with a shape factor below 1.5 could potentially exceed \(V_n\), according to the interaction equation at moments due to the factored loadings slightly above the defined limiting value of 0.5\(\phi_f M_p\). Thus, for compact sections, an upper limit of 1.0 is placed on \(R\).

To avoid the interaction effect, transverse stiffeners may be spaced so that the shear due to the factored loadings does not exceed the larger of:

- 0.60 \(\phi_v V_n\) where \(V_n\) is given by Equation 12.7.6.3-3,
- the factored shear buckling or shear yield resistance equal to \(\phi_C V_p\).

The coefficient, \(C\), is equal to the ratio of the elastic shear buckling stress of the panel, computed assuming simply-supported boundary conditions, to the shear yield strength assumed to be equal to \(F_{yw}/(3)^{0.5}\). Equation 12.7.6.3-7 is applicable only for \(C\) values not exceeding 0.8, Basler (1961). Above 0.8, \(C\) values are given by Equation 12.7.6.3-6 until a limiting slenderness ratio is reached where

\[
M_y = \text{yield moment (N\text@m)}
\]
\[
D = \text{web depth (mm)}
\]
\[
d_o = \text{stiffener spacing (mm)}
\]
\[
C = \text{ratio of the shear buckling stress to the shear yield strength}
\]
the shear buckling stress is equal to the shear yield strength and \( C = 1.0 \). Equation 12.7.6.3-8 for the shear buckling coefficient is a simplification of two exact equations for \( k \) that depend on the panel aspect ratio.

The ratio, \( C \), is given below:

If \( \frac{D}{t_w} < 1.10 \sqrt[2]{\frac{E_k}{F_{yw}}} \), then

\[
C = 1.0 \quad \text{(12.7.6.3-5)}
\]

If \( 1.10 \sqrt[2]{\frac{E_k}{F_{yw}}} \# \frac{D}{t_w} \# 1.38 \sqrt[2]{\frac{E_k}{F_{yw}}} \), then:

\[
C' = \frac{1.10 D}{t_w} \sqrt[2]{\frac{E_k}{F_{yw}}} \quad \text{(12.7.6.3-6)}
\]

If \( \frac{D}{t_w} > 1.38 \sqrt[2]{\frac{E_k}{F_{yw}}} \), then:

\[
C' = \frac{1.52}{\left(\frac{D}{t_w}\right)^2} \left(\frac{E_k}{F_{yw}}\right) \quad \text{(12.7.6.3-7)}
\]

for which:

\[
k' = \frac{5}{\left(\frac{d_o}{D}\right)^2} \quad \text{(12.7.6.3-8)}
\]

The nominal shear resistance of interior web panels of non-compact sections complying with the requirements for maximum stiffener spacing is:

If \( f_u \# 0.75 \varphi F_y \), then:
If \( f_u > 0.75 \phi_f F_y \), then:

\[
V_n' > RV_p \left[ C \frac{0.87 (1 + C)}{\sqrt{1 \% \left( \frac{d_o}{D} \right)^2}} \right] \quad (12.7.6.3-10)
\]

for which:

\[
R > 0.6 \% 0.4 \left( \frac{F_r \& f_u}{F_r \& 0.75 \phi_f F_y} \right) \# 1.0 \quad (12.7.6.3-11)
\]

where:

- \( f_u \) = flexural stress in the compression or tension flange due to the factored loading, whichever flange has the maximum ratio of \( f_u \) to \( F_r \) in the panel under consideration (MPa)
- \( C \) = ratio of shear buckling stress to the shear yield strength
- \( F_r \) = factored flexural resistance of the compression flange for which \( f_u \) was determined (MPa)

The nominal shear resistance of an end panel is limited to the shear buckling or shear yield force given by:

\[
V_n = CV_p \quad (12.7.6.3-12)
\]

for which:

\[
V_p = 0.58 F_{yw} D t_w \quad (12.7.6.3-13)
\]

where:

- \( C \) = ratio of the shear buckling stress to the shear yield strength
- \( V_p \) = plastic shear force (N)
The transverse stiffener spacing for end panels without a longitudinal stiffener is not to exceed 1.5D. The transverse stiffener spacing for end panels with a longitudinal stiffener is not to exceed 1.5 times the maximum subpanel depth.

The transverse stiffener spacing requirements are applicable to hybrid sections.

Tension-field action is not permitted for hybrid sections. Thus, the nominal shear resistance is limited to either the shear yield or the shear buckling force given by:

\[ V_n = C V_p \]  
(12.7.6.3-14)

### 12.7.7 Stiffeners

Provisions for transverse intermediate, bearing and longitudinal stiffeners are summarized below.

#### 12.7.7.1 TRANSVERSE INTERMEDIATE STIFFENERS

Transverse stiffeners consist of plates or angles welded or bolted to either one or both sides of the web.

Stiffeners not used as connection plates are required to have a tight fit at the compression flange, but need not be in bearing with the tension flange.

Stiffeners used as connecting plates for diaphragms or cross-frames should be connected by welding or bolting to both flanges.

The distance between the end of the web-to-stiffener weld and the near edge of the web-to-flange fillet weld should not be less than 4\(t_w\) or more than 6\(t_w\).

The requirements in this article are intended to prevent local buckling of the transverse stiffener.

The width, \(b_t\), of each projecting stiffener element must satisfy:

\[ 50 \% \frac{d}{30} \leq b_t \leq 0.48 t_p \sqrt{\frac{E}{F_{ys}}} \]  
(12.7.7.1-1)

and

\[ 16.0 t_p \leq b_t \leq 0.25 b_f \]  
(12.7.7.1-2)

where:

\[ d = \text{steel section depth (mm)} \]
tp = thickness of the projecting element (mm)

F_{ys} = specified minimum yield strength of the stiffener (MPa)

b_t = full-width of compression or tension flange (mm)

The moment of inertia of any transverse stiffener must satisfy:

\[ I_t \geq d_o t_w^3 J \]  \hspace{1cm} (12.7.7.1-3)

for which:

\[ J = 2.5 \left( \frac{D_p}{d_o} \right)^2 \times 2.0 \times 0.5 \]  \hspace{1cm} (12.7.7.1-4)

where:

\[ I_t \] = moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (mm$^4$)

\[ t_w \] = web thickness (mm)

\[ d_o \] = transverse stiffener spacing (mm)

\[ D_p \] = web depth for webs without longitudinal stiffeners or maximum subpanel depth for webs with longitudinal stiffeners (mm)

For the web to adequately develop the tension field, the transverse stiffener must have sufficient rigidity to cause a node to form along the line of the stiffener. For ratios of \((d_o/D_p)\) less than 1.0, much larger values of \(I_t\) are required as discussed in Timoshenko (1961). Each subpanel of a subdivided panel behaves as a separate panel. Therefore, the depth of the subpanel is used in this requirement. This requirement should also be investigated for web panels not required to develop a tension field.

Transverse stiffeners used in conjunction with longitudinal stiffeners must also satisfy:

\[ I_t \geq \left( \frac{b_t}{b_{R_t}} \right) \left( \frac{D}{3.0d_o} \right) I_R \]  \hspace{1cm} (12.7.7.1-5)

where:

\[ b_t \] = projecting width of transverse stiffener (mm)
Lateral loads along the length of a longitudinal stiffener are transferred to the adjacent transverse stiffeners as concentrated reactions, Cooper (1967). Equation 12.7.7.1-3 gives a relationship between the moments of inertia of the longitudinal and transverse stiffeners to ensure the latter does not fail under the concentrated reactions. Equation 12.7.7.1-3 is equivalent to Equation 10-111 in the 1992 AASHTO.

Transverse stiffeners need sufficient area to resist the vertical component of the tension field. The formula for the required stiffener area below can give a negative result. In that case, the required area is 0.0. A negative result indicates that the web alone is sufficient to resist the vertical component of the tension field. The stiffener then need only be proportioned for stiffness and satisfy the projecting width requirements. For web panels not required to develop a tension field, this requirement need not be investigated.

Transverse intermediate stiffeners required to carry the forces imposed by tension-field action of the web must satisfy:

\[
A_s \geq \left[ 0.15B \frac{D}{t_w} (1 \& C) \left( \frac{V_u}{V_r} \right) \& 18 \right] \frac{F_{yw}}{F_{cr}} t_w^2 \quad (12.7.7.1-6)
\]

\[
F_{cr} = \frac{0.311 E}{\left( \frac{b_t}{t_p} \right)^2} \leq F_{ys} \quad (12.7.7.1-7)
\]

where:

\( V_r \) = factored shear resistance (N)

\( V_u \) = shear due to factored loads at the strength limit state (N)

\( A_s \) = stiffener area; total area of both stiffeners for pairs (mm²)

\( B \) = 1.0 for stiffener pairs

\( B \) = 1.8 for single angle stiffeners

\( B \) = 2.4 for single plate stiffeners
C = ratio of the shear buckling stress to the shear yield strength

$F_{yw}$ = specified minimum yield strength of the web (MPa)

$F_{ys}$ = specified minimum yield strength of the stiffener (MPa)

12.7.7.2 BEARING STIFFENERS

Bearing reactions and other concentrated loads, either in the final state or during construction, shall be resisted by bearing stiffeners.

Inadequate provision to resist concentrated loads has resulted in failures, particularly in temporary construction.

Bearing stiffeners should be placed on webs of rolled beams at all bearing locations and other points of concentrated loads where:

$$V_u > 0.75\varphi_b V_n$$  \hspace{1cm} (12.7.7.2-1)

where:

$\varphi_b$ = resistance factor for bearing

$V_u$ = shear due to the factored loads (N)

$V_n$ = nominal shear resistance (N)

Bearing stiffeners should be placed on the webs of plate girders at all bearing locations and at all locations supporting concentrated loads.

Bearing stiffeners consist of one or more plates or angles welded or bolted to both sides of the web. The connections to the web are to be designed to transmit the full bearing force due to the factored loads.

The stiffeners should extend the full-depth of the web and, as closely as practical, to the outer edges of the flanges.

Each stiffener should be either milled to fit against the flange through which it receives its reaction, or attached to that flange by a full penetration groove weld.

If an owner chooses not to utilize bearing stiffeners where specified in this article, the web crippling provisions of AISC (1986) should be used to investigate the adequacy of the component to resist a concentrated load.

The width, $b_i$, of each projecting stiffener element must satisfy:
To bring bearing stiffener plates tight against the flanges, part of the stiffener must be clipped to clear the web-to-flange fillet weld. Thus, the area of direct bearing is less than the gross area of the stiffener. The bearing resistance is based on this bearing area and the yield strength of the stiffener.

The factored bearing resistance, $B_r$, shall be taken as:

$$B_r = \varphi_b A_{pn} F_{ys}$$  \hspace{1cm} (12.7.7.2-3)

where:

- $F_{ys} = \text{specified minimum yield strength of the stiffener (MPa)}$
- $A_{pn} = \text{area of the projecting elements of the stiffener outside of the web-to-flange fillet welds, but not beyond the edge of the flange (mm}^2\text{)}$
- $\varphi_b = \text{resistance factor for bearing}$

The factored axial resistance, $P_r$, is determined as specified for compression members. The radius of gyration shall be computed about the mid-thickness of the web. The end restraint against column buckling provided by the flanges allows for the use of a reduced effective length, therefore, the effective length may be 0.75$D$, where $D$ is the web depth.

The web of hybrid girders is not included in the computation of the radius of gyration because the web may be yielding due to longitudinal flexural stress. At end supports where the moment is NIL, the web may be included.

For stiffeners bolted to the web, the effective column section shall consist of the stiffener elements only.

A portion of the web is assumed to act in combination with the bearing stiffener plates.

For stiffeners welded to the web, the effective column section shall consist of all stiffener elements, plus a centrally located strip of web extending not more than 9$t_w$ on each side of the outer projecting elements of the group, if more than one pair of stiffeners is used.
The strip of web is not included in the effective section at interior supports of continuous span hybrid members if:

\[
\frac{F_{yw}}{F_{yf}} < 0.70
\]  \hspace{1cm} (12.7.7.2-4)

where:

\[
F_{yw} = \text{specified minimum yield strength of the web (MPa)}
\]
\[
F_{yf} = \text{higher of the specified minimum yield strengths of the flanges (MPa)}
\]

### 12.7.7.3 LONGITUDINAL STIFFENERS

Where required, longitudinal stiffeners should consist of either a plate welded longitudinally to one side of the web, or a bolted angle, and shall be located a distance of \(2D_c/5\) from the inner surface of the compression flange, where \(D_c\) is the depth of the web in compression at the section with the maximum compressive flexural stress.

Theoretical and experimental studies have indicated that the optimum location of one longitudinal stiffener is \(2D_c/5\) for bending and \(D/2\) for shear. Tests have also shown that longitudinal stiffeners located at \(2D_c/5\) can effectively control lateral web deflections under flexure, Cooper (1967). The distance \(2D_c/5\) is specified because shear is always accompanied by moment and because a properly proportioned longitudinal stiffener reduces lateral web deflections caused by shear.

Because \(D_c\) may vary along the length of the span, it is specified here that the stiffener be located based on the \(D_c\) computed at the section with the largest compressive flexural stress. Thus, the stiffener may not be located at its optimum location at other sections with a lower stress and a different \(D_c\). Research is presently underway to attempt to better define the resistance for sections with longitudinal stiffeners at locations other than the optimum location.

Continuous longitudinal stiffeners placed on the opposite side of the web from the transverse intermediate stiffeners are preferred. If longitudinal and transverse stiffeners must be placed on the same side of the web, it is preferable that the longitudinal stiffener not be interrupted for the transverse stiffener. If the longitudinal stiffener must be interrupted, it should only be interrupted in compression regions. Furthermore, if the longitudinal stiffener is interrupted, its area should not be included when calculating section properties.

Where the transverse stiffener is interrupted, copes should be provided for the longitudinal stiffener-to-web welds to avoid intersecting welds, and the transverse stiffener should be fillet or groove welded to the longitudinal stiffener to assure it will function properly in resisting web buckling and supporting the longitudinal stiffener. All interruptions must be carefully designed with respect to fatigue, Schilling (1986).
The projecting width, \( b_R \) of the stiffener must satisfy:

\[
b_R \geq 0.48 t_s \sqrt{\frac{E}{F_{ys}}} \tag{12.7.7.3-1}
\]

where:

\( t_s \) = thickness of stiffener (mm)

\( F_{yc} \) = specified minimum yield strength of the stiffener (MPa)

This requirement is intended to prevent local buckling of the longitudinal stiffener. The yield strength of the adjacent compression flange is conservatively used rather than the yield strength of the stiffener to prevent overstressing the stiffener if it has a lower yield strength than the compression flange.

The section properties of the stiffener shall be based on an effective section consisting of the stiffener and a centrally located strip of the web not exceeding 18\( t_w \).

The moment of inertia requirement is to ensure the stiffener will have adequate rigidity to force a horizontal line of nil deflection in the web panel. The radius of gyration requirement is to ensure the longitudinal stiffener will be rigid enough to withstand the axial compressive stress without lateral buckling. A partially restrained end condition is assumed for the stiffener acting as a column. It is also assumed in the development of Equation 12.7.7.3-2 that the eccentricity of the load and initial out-of-straightness cause a 20% increase in stress in the stiffener.

Longitudinal stiffeners must satisfy:

\[
I_R \geq 2.4 \left( \frac{d_o}{D} \right)^2 \sqrt{D t_w^3} \tag{12.7.7.3-2}
\]

\[
r \geq 0.234 d_o \sqrt{\frac{F_{ys}}{E}} \tag{12.7.7.3-3}
\]

where:

\( I_R \) = moment of inertia of the longitudinal stiffener and the web strip about the edge in contact with the web (mm\(^4\))

\( r \) = radius of gyration of the longitudinal stiffener and the web strip about the edge in contact with the web (mm)

\( D \) = web depth (mm)
do = transverse stiffener spacing (mm)

tw = web thickness (mm)

F_{ys} = specified minimum yield strength of the stiffener (MPa)

A longitudinal stiffener meeting the requirements above will have sufficient area to anchor the tension field. Therefore, no additional area requirement is given for longitudinal stiffeners.

12.7.8 Constructibility

New provisions for flexural and shear resistance of composite sections which are non-composite during construction are contained in Article S6.10.3.2.

12.7.9 Inelastic Analysis Procedures

Provisions similar to the Alternate Load Factor Design Guide Specifications of AASHTO are contained in Article S6.10.10 for both the mechanism and unified autostress methods. Deflection limitations at the service limit state are also imposed.

12.7.10 Steel Plate Girder Design Example

12.7.10.1 BRIDGE DESCRIPTION

Bridge Geometry:

Total length = 72 000 mm (distance between centerlines of bearing at abutments)
Number of spans = 2
Length of spans = 36 000 mm - 36 000 mm
Number of lanes = 2
Number of girders = 4
Lane width = 3600 mm
Shoulder width = 1560 mm
Overhang = 800 mm
Total deck width = 11 200 mm
Girder type = Straight, constant depth plate girder, homogeneous
Skew angle = No skew
Cross-frame spacing = 7200 mm

Cross-section Geometry:

Girder:

Spacing = 3200 mm
Web depth = 1425 mm
Slab/Wearing Surface:

Slab Thickness = 210 mm (10 mm of which is considered non-structural)
Haunch Thickness = 0 mm
Reinforcing (top) = 4700 mm² (within effective slab width for negative moment)
Reinforcing (bottom) = 2400 mm² (within effective slab width for negative moment)
SIP Forms = 0.72 kN/m²
FWS = 1.44 kN/m²

Parapets (PL-3):

Width at base = 440 mm
Weight = 7.61 kN/m

Materials:

Structural steel = 345 MPa
Reinforcing steel = 400 MPa
\( E_s \) = 200 000 MPa
Slab concrete = 28 MPa
\( E_c \) = 25 400 MPa
\( n \) = 8
Steel Density = 7850 kg/m³
Concrete Density = 2400 kg/m³

Figure 12.7.10.1-1 - Bridge Cross-Section Geometry
12.7.10.2 ASSUMPTIONS

C Interior girder design
C $ADTT = 500$; $ADTT_{SL} = 425$
C Weight of attachments (stiffeners, cross-frames, etc.) taken into account by increasing girder weight by 5 percent
C Live load applied to “n” section, superimposed dead loads applied to “3n” section
C Composite, positive flexural stiffness properties used for entire length of girder for superimposed dead load and live load analysis (SC6.10.1.3)
C Ignore contribution of slab reinforcing, except for negative flexure
C Strength II loading case (permit live load) not considered
C Strength IV loading case will not control ($DL/LL << 7.0$; see C3.4.1)
C Unshored construction
C Slab pouring sequence will not be considered

12.7.10.3 LIVE LOAD VEHICLES

C Design Vehicle (Application per Article S3.6.1.3.1)
C Fatigue Vehicle (Article S3.6.1.4.1)
C Deflection Vehicle (Article S3.6.1.3.2)
C Live and permanent load analysis will be performed using PennDOT’s Continuous Beam Analysis (CBA) program

12.7.10.4 PLATE SIZES

Table 12.7.10.4-1 - Plate Sizes (all dimensions in mm)

<table>
<thead>
<tr>
<th>Section</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>6000</td>
<td>17400</td>
<td>2400</td>
<td>4800</td>
<td>2400</td>
<td>3000</td>
</tr>
<tr>
<td>Web Depth</td>
<td></td>
<td></td>
<td>1425</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Web Thickness</td>
<td>12</td>
<td></td>
<td>10</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Top Flange Width</td>
<td>300</td>
<td></td>
<td>460</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Flange Thickness</td>
<td>13</td>
<td>16</td>
<td>25</td>
<td>42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom Flange Width</td>
<td>300</td>
<td></td>
<td>460</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom Flange Thickness</td>
<td>18</td>
<td>30</td>
<td>21</td>
<td>23</td>
<td>31</td>
<td>48</td>
</tr>
</tbody>
</table>

Figure 12.7.10.4-1 - Cross-section References
Calculate the effective flange width, \( b_e \) (S4.6.2.6)

The effective flange width, \( b_e \) (effective slab width for composite action), can be a function of the effective span length which is taken as the actual span length for simple spans and as the distance between points of permanent load contraflexure for continuous spans, as appropriate for either positive or negative flexure.

Therefore, for a two-span continuous girder the effective span length \( L_{eff} \) is:

*For positive moment regions:*

\[
C \quad L_{eff} = 0.7 \times \text{(Span length)} = 0.7 \times 36000 = 25200 \text{ mm}
\]

*For negative moment regions:*

\[
C \quad L_{eff} = 2 \times 0.3 \times \text{(Span length)} = 0.6 \times 36000 = 21600 \text{ mm}
\]

For an interior girder, the effective slab width is then taken as the least of the following:

\[
\frac{1}{4}(L_{eff}), \quad 12t_w + \text{MAX}(t_w, b_n/2), \quad \text{Average beam spacing, } S
\]

So, in the positive moment region (Sections A, B and C):

\[
\begin{align*}
C \quad \frac{1}{4}(L_{eff}) &= 0.25 \times 25200 = 6300 \\
C \quad 12t_w + \text{MAX}(t_w, b_n/2) &= 12 \times 200 + \text{MAX}(200, 300/2) = 2550 \\
C \quad S &= 3200
\end{align*}
\]

And, in the negative moment region (Sections D, E and F):

\[
\begin{align*}
C \quad \frac{1}{4}(L_{eff}) &= 0.25 \times 21600 = 5400 \\
C \quad 12t_w + \text{MAX}(t_w, b_n/2) &= 12 \times 200 + \text{MAX}(200, 460/2) = 2630 \\
C \quad S &= 3200
\end{align*}
\]
12.7.10.5 SECTION PROPERTIES

Table 12.7.10.5-1 - Permanent Loads and Section Properties
(dimensions in mm unless stated otherwise)

<table>
<thead>
<tr>
<th>Section</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of Range</td>
<td>6000</td>
<td>23000</td>
<td>25400</td>
<td>30600</td>
<td>33000</td>
<td>36000</td>
</tr>
<tr>
<td>Steel Area</td>
<td>26400</td>
<td>30000</td>
<td>27300</td>
<td>32190</td>
<td>40010</td>
<td>55650</td>
</tr>
<tr>
<td>Steel + Details</td>
<td>2.13 kN/m</td>
<td>2.43 kN/m</td>
<td>2.21 kN/m</td>
<td>2.60 kN/m</td>
<td>3.24 kN/m</td>
<td>4.50 kN/m</td>
</tr>
<tr>
<td>Slab, Haunch, SIP</td>
<td>18.12 kN/m</td>
<td>18.12 kN/m</td>
<td>18.12 kN/m</td>
<td>18.12 kN/m</td>
<td>18.12 kN/m</td>
<td>18.12 kN/m</td>
</tr>
<tr>
<td>Total DL1</td>
<td>20.25 kN/m</td>
<td>20.55 kN/m</td>
<td>20.33 kN/m</td>
<td>20.72 kN/m</td>
<td>21.36 kN/m</td>
<td>22.62 kN/m</td>
</tr>
</tbody>
</table>

DL2 Parapet | 3.81 kN/m |

DL2 FWS, Util | 3.72 kN/m |

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Note: The member proportions should be checked per Article S6.10.2 which requires that the ratio of I_{y,c}/I_{y} (moment of inertia of compression flange divided by moment of inertia of the steel section, both taken about a vertical axis) be between 0.1 and 0.9. For Sections A through F above, this ratio ranged from 0.30 to 0.59. The web and flange also need to be checked for conformance with Articles S6.10.2.2 and S6.10.2.3.
12.7.10.6 DISTRIBUTION FACTORS

Use of The Multiple Presence Factors

Longitudinal Stiffness, $K_g$:

$K_g$, was based on a weighted average for the entire length of the bridge. The resulting value of $K_g$ was computed as $3.05 \times 10^{11}$ mm$^4$.

Single Lane Loaded:

Flexure:

$DFM_{SL} = 0.06 \% \left( \frac{S}{4300} \right)^{0.4} \% \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_g}{L t_s^3} \right)^{0.1} = 0.49$

Shear:

$DFV_{SL} = 0.36 \% \frac{S}{7600} = 0.78$

Multiple Lanes Loaded:

Flexure:

$DFM_{ML} = 0.075 \% \left( \frac{S}{2900} \right)^{0.6} \% \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_g}{L t_s^3} \right)^{0.1} = 0.73$

Shear:

$DFV_{ML} = 0.2 \% \frac{S}{3600} \& \frac{S}{10700}^{2.0} = 1.0$

Fatigue:

Flexure:

$DFM_{fatigue} = DFM_{SL}/1.2 = 0.41$

Shear:

$DFV_{fatigue} = DFV_{SL}/1.2 = 0.65$

Deflection (assuming all girders deflect equally; see Article 2.5.2.6.2):

$DF_\Delta = N_L/N_B = 2/4 = 0.5$
Dynamic Load Allowance (IM) - See Table S3.6.2.1-1:

Fatigue and Fracture Limit State:

IM = 1.15

Other Limit States:

IM = 1.33

12.7.10.7 GENERAL LOAD FACTORS, \( \eta \) (Articles S1.3.3, S1.3.4, S1.3.5)

The LRFD Specification allows the owner/engineer to account for several parameters that, in the past, were not generally considered when factoring the design loads. These include ductility, redundancy and operational importance.

A. Ductility, \( \eta_D \) = 1.00 for Strength Only
B. Redundancy, \( \eta_R \) = 1.00 for Strength Only
C. Importance, \( \eta_I \) = 0.95 for Strength Only
   (would also apply to extreme event if seismic loads were to be investigated)
D. Total (\( \eta_D \eta_R \eta_I \)):
   Strength: \[ \eta = 0.95 \times 0.95 = 0.90 \text{ (OK)} \]
   Fatigue, Service: \[ \eta = 1.00 \times 0.95 = 0.95 \text{ (OK)} \]

12.7.10.8 LOAD CALCULATIONS (Permanent and Live)

Permanent Loads:

Non-Composite Loads:

C Steel Girder, Section A:

\[ A = 26 \, 400 \, \text{mm}^2 \]
\[ A = \left( \frac{26 \, 400 \, \text{mm}^2}{1 \, \text{E6} \, \text{mm}^2} \right) \left( \frac{1 \, \text{m}^2}{7850 \, \text{kg/m}^3} \right) \left( \frac{9.81 \, \text{m/s}^2}{1 \, \text{kN}} \right) \]
\[ 2.03 \, \text{kN/m} \]

Use “detail factor” of 1.05 to account for stiffeners, cross-frames, etc.:

\[ 2.03 \times 1.05 = 2.13 \, \text{kN/m} \]

Calculations should be repeated for Sections B through F, but are not shown here. See the summary of dead loads and section properties in Table 12.7.10.5.1.
C Slab:

\[ A = S t = (3200 \text{ mm})(210) = 672\,000 \text{ mm}^2 \]

\[ (67\,200 \text{ mm}^2) \left( \frac{m^2}{1E6 \text{ mm}^2} \right) (2400 \text{ kg/m}^3)(9.81 \text{ m/s}^2) \left( \frac{1 \text{kN}}{1000 \text{ N}} \right) \]

\[ 15.82 \text{ kN/m} \]

C SIP Forms:

\[ (3.2 \text{ m}) (0.72 \text{ kN/m}^2) = 2.30 \text{ kN/m} \]

C Haunch:

Assumed zero haunch

Composite Loads:

C Parapet (equally distributed to all girders):

\[ (7.61 \text{ kN/m})(2 \text{ parapets})/(4 \text{ girders}) = 3.81 \text{ kN/m} \]

C Future Wearing Surface (FWS equally distributed to all girders):

\[ (10.32 \text{ m})(1.44 \text{ kN/m}^2)/(4 \text{ girders}) = 3.72 \text{ kN/m} \]

Utilities:

Assumed no utility loads

PennDOT’s Continuous Beam Analysis Program (CBA) was used to compute the unfactored moments, shears and deflections due to the permanent loads. The results are tabulated in Table 1 and shown in Figure 1.

Live Loads:

PennDOT’s Continuous Beam Analysis Program (CBA) was used to compute the LRFD live loads for the HL93 loading and the fatigue live loading. The results are tabulated in Table 2 and shown in Figure 1. The deflections shown adjacent to the LRFD live load moments and shears correspond to the deflection vehicle loading given in Article S3.6.1.3.2 rather than the design vehicle.
Figure 12.7.10.8-1 - Unfactored Moment and Shear Diagrams
## Unfactored Load Effects

Table 12.7.10.8-1 - Permanent Loads (Moments in kN@, Shears in kN, Deflections in mm):

<table>
<thead>
<tr>
<th>X (m)</th>
<th>DL1 M</th>
<th>DL1 V</th>
<th>DL1 Defl</th>
<th>DL2P M</th>
<th>DL2P V</th>
<th>DL2P Defl</th>
<th>DL2W M</th>
<th>DL2W V</th>
<th>DL2W Defl</th>
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<td>0.0</td>
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<td>82.5</td>
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Table 12.7.10.8-2 - Unfactored Live Loads (Moments in kN\#m, Shears in kN, Deflections in mm):

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<th>X (m)</th>
<th>LRFD Live Load (including DF and IM)</th>
<th>LRFD Fatigue (including DF and IM)</th>
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</tbody>
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12.7.10.9 LIVE LOAD DEFLECTION (Article S2.5.2.6)

For a steel girder bridge without pedestrian loads, the recommended deflection limit is the span length/800. Thus, for this example the maximum live load deflection is:

\[
\Delta_{\text{max}} = \frac{\text{Span}}{800} \times \frac{36000}{800} = 45 \text{ mm}
\]

The maximum computed deflection has been determined to be 26 mm which is less than the recommended allowable deflection.
12.7.10.10 SPECIFICATION CHECKS

1. Flexural capacity
   - Critical Sections: **14.4 m, 36.0 m** (points of maximum positive and negative moment)
   - Web fatigue stress limits (flexure and shear)

2. Shear (stiffened web) - Show a check at 0 and 36.0 meters (distance measured centerline of bearing at the abutment)

3. Fatigue (at selected points for various detail categories)

4. Shear connectors

5. Transverse stiffeners and bearing stiffeners

6. Wind loads

12.7.10.10.1 Specification Check: Flexure

**Compute Flexural Capacity at Point of Maximum Positive Moment**

\[(x = 14.4 \text{ m})\]

**Check Compactness:**

Web Slenderness (S6.10.4.1.2)

\[
\frac{2D_{cp}}{t_w} \# 3.76 \sqrt{\frac{E}{F_{yc}}} \quad (S6.10.4.1.2-1)
\]

\[D_{cp} = 0, \text{ therefore, the section is compact}\]

Compression flange slenderness and compression flange bracing:
For the final stage of loading, i.e., at the strength limit state, the compression flange is assumed to be fully braced by the concrete slab. However, the compression flange bracing and slenderness requirements should be checked during a construction analysis using the appropriate loading condition.

**Compute the Flexural Capacity of a Compact Section:**

Compactness must be checked at the interior support to determine the applicable flexural capacity equation for the section of interest.

**Check Compactness at the Pier \((x = 36 \text{ m})\):**

Web Slenderness (S6.10.4.1.2)

\[
\frac{2D_{cp}}{t_w} \# 3.76 \sqrt{\frac{E}{F_{yc}}} \quad (S6.10.4.1.2-1)
\]
where:

\[ D_{cp} = 986 \]
\[ t_w = 10 \]

\[ \frac{2D_{cp}}{t_w} = \frac{2(986)}{10} = 197 \]

\[ 3.76 \sqrt{\frac{E}{F_{yc}}} \times 3.76 \sqrt{\frac{200000}{345}} = 90.5 < 197 \]

therefore, the section at the interior support is non-compact.

Ductility Check:

Because the factored stress in the tension flange exceeds the yield stress of the flange, the ductility check of Article S6.10.4.2.2b applies:

\[ \left( \frac{D_p}{D^\dagger} \right) \#5 \]

(S6.10.4.2.2b-1)

in which:

\[ D^\dagger = \beta \left( \frac{d}{t_s} \frac{d}{t_n} \right) \frac{7.5}{7.5} \]

where:

\[ \beta = 0.9 \text{ for } F_y = 250 \text{ MPa} \]
\[ = 0.7 \text{ for } F_y = 345 \text{ MPa} \]

\[ d = D + t_c + t_n = 1425 + 13 + 30 = 1468 \]

\[ t_s = 200 \]
\[ t_n = 0 \text{ (no haunch assumed)} \]

Calculate the depth to the plastic neutral axis from the top of the slab, \( D_p \) (see Appendix SA6.1, assuming PNA is in the slab above \( P_{rb} \))

\[ D_p \times \sqrt{\frac{P_{rb}}{P_s} \%P_c \%P_w \%P_t \& P_{rt}} \]

The longitudinal reinforcing steel in the slab was neglected for positive flexural resistance in this example (as is the practice by some DOT's), therefore, the forces \( P_{rt} \) and \( P_{rb} = 0 \).

\[ D_p \times \sqrt{\frac{0 \times 345(12)(300) \times 345(12)(1425) \times 345(30)(300) & 0}{0.85(28)(2550)(200)}} \]

\[ ' \text{ 170} \]
And the upper limit on the depth of web in compression at the plastic moment is calculated as:

\[
\beta \left( \frac{d \% t_s \% t_h}{7.5} \right) \left( 0.7 \times \frac{1468 \% 200 \% 0}{7.5} \right) \left\{ 156 \text{ (OK)} \right\}
\]

\[
\frac{D_p}{D_i} = 170 \times \frac{1}{156} \quad 1.09 < 5 \text{ (OK)}
\]

The Specification allows one of two methods for computing the flexural capacity in such cases, Method “A” and Method “B.” In this case, the simpler Method “A” has been selected. Thus, the flexural capacity shall be determined by the following:

- If \( D_p \neq D' \), then:
  \[ M_n = M_p \]  \quad \text{(S6.10.4.2.2a-1)}

- If \( D' < D_p \neq 5D' \), then:
  \[ M_n = \frac{5M_p \times 0.85M_y \times 0.85M_p}{4} \times \left( \frac{D_p}{D_i} \right) \]  \quad \text{(S6.10.4.2.2a-2)}

Because the section is non-compact, \( M_n \) shall not be greater than:

\[ M_n \leq 1.3R_hM_y \]  \quad \text{(S6.10.4.2.2a-3)}

where:

\[ R_h = 1.0 \text{ (non-hybrid section; see S6.10.4.3.1a)} \]

**Compute the yield moment, \( M_y \):**

Table 12.7.10.10.1-1 - Factored Moment (values in kN\(\cdot\)m; Strength I):

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>(\eta_y)</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>MDL1</td>
<td>1582.7</td>
<td>(0.95)(1.25)</td>
<td>1879</td>
</tr>
<tr>
<td>MDL2P</td>
<td>314.1</td>
<td>(0.95)(1.25)</td>
<td>373</td>
</tr>
<tr>
<td>MDL2W</td>
<td>306.7</td>
<td>(0.95)(1.50)</td>
<td>437</td>
</tr>
<tr>
<td>LL</td>
<td>2770.5</td>
<td>(0.95)(1.75)</td>
<td>4606</td>
</tr>
</tbody>
</table>

\[ M_y ' = M_{DL1} \% M_{DL2} \% S_{LL} \left( F_{yf} \& \frac{M_{DL1}}{S_{DL1}} \& \frac{M_{DL2}}{S_{DL2}} \right) \]  \quad \text{(See SA6.2)}

Compute the yield moment for each flange and the minimum yield moment controls:
\[ M_{ycf} = 1879 \times 10^{6} \times 1.364 \times 10^{8} \left( \frac{345 \times 1879 \times 10^{6}}{1.083 \times 10^{7}} \times \frac{810 \times 10^{6}}{4.495 \times 10^{7}} \right) \times 10^{6} \]

' 23622 kN@

\[ M_{yhf} = 1879 \times 10^{6} \times 2.203 \times 10^{7} \left( \frac{345 \times 1879 \times 10^{6}}{1.490 \times 10^{7}} \times \frac{810 \times 10^{6}}{2.027 \times 10^{7}} \right) \times 10^{6} \]

' 6631 kN@

\[ M_{ymin} = 6631 \text{ kN@} \]
\[ M_{u} = 1879 + 810 + 4606 = 7295 \text{ kN@} \]
\[ D_{p} = 170 \]
\[ D' = 156 \]
\[ 5D' = 780 \]

156 < 170 < 780

Therefore:

\[ M_{n} = \frac{5(9988) \times 0.85(6631)}{4} \times \frac{0.85(6631) \times 9988}{4} \times \frac{170}{156} \]

= 9890 kN@

1.3 \( R_{n} \) \( M_{u} = (1.3)(1.0)(6631) = 8620 \text{ kN@} \)

9890 kN@ > 8620 kN@, therefore, \( M_{n} = 8620 \text{ kN@} \)

8620 kN@ > \( M_{u} = 7295 \text{ kN@} \) (OK)

Check Service Limit State Control of Permanent Deflections at Location of Maximum Service II Flexural Stress (\( x = 14.4 \text{ m} \)).

Article S6.10.5:

The Service II load combination corresponds to the "overload check" in the 1992 AASHTO Standard Specifications. It was determined that for this problem, the maximum flexural stress for the Service II load combination was located in the bottom flange at a distance 14.4 meters from the support at the abutment. Figure 1 shows the Service II flexural stress in each of the flanges due to the positive and negative live load effects.
For a composite section, the stress in each flange is limited by the following equation.

\[ f_r \leq 0.95 F_y \]  

(S6.10.5.2-1)

So,

\[ f_r \leq 0.95(345) = 328 \text{ MPa} \]

and the maximum stress at 14.4 meters is 300 MPa, therefore, the girder satisfies the Service II limit state requirements. It should also be noted that the live load factor for the Service II limit state is 1.30. However, if the Service II limit state is to be checked for a permit vehicle, the Owner may wish to select a reduced live load factor (SC6.10.3.1).
Check the Web Fatigue Stresses at the Point of Maximum Positive Moment (x = 14.4 m), Article S6.10.6.3:

This provision requires the calculation of the depth of web in compression resulting from the maximum fatigue loading (including the unfactored permanent load stresses). Note: the load factor for the fatigue live load is 0.75 and the general load factor, $\eta$, does not apply for the fatigue limit state. Also, Article S6.10.6.2 requires that twice the factored fatigue be used for checking the web fatigue stress. This applies to both shear and flexural fatigue stresses due to the fatigue vehicle.

Calculate the stress in each flange due to the unfactored dead load plus twice the factored fatigue live load.

$$f_c = \frac{M_{DL1}}{S_{cDL1}} \% \frac{M_{DL2}}{S_{cDL2}} \% \frac{2.0M_{LLf}}{S_{cLL}}$$

$$f_t = \frac{M_{DL1}}{S_{tDL1}} \% \frac{M_{DL2}}{S_{tDL2}} \% \frac{2.0M_{LLf}}{S_{tLL}}$$

$$D_c = \frac{df_c}{(f_c \% f_t)} \% \frac{1468(169)}{(169 \% 193)} \% 672 \text{ mm}$$

Check:

$$\frac{2D_c}{t_w} \# \frac{5.70}{\sqrt{\frac{E}{F_{yw}}}}$$

$$\frac{2D_c}{t_w} = \frac{2(672)}{12} \# 112$$

$$5.70 \sqrt{\frac{E}{F_{yw}}} = 5.70 \frac{200000}{345} \# 137 > 112$$

Therefore,

$$f_{cr} \# F_{yw} \quad (S6.10.6.3-1)$$

$$F_{yw} = 345 \text{ MPa}$$

So,

$$f_{cr} \# 345 \text{ MPa} \quad (OK)$$

**Web Fatigue Shear Stress at X = 14.4 m (S6.10.6.4)**

In addition to the web fatigue check for flexure, Article S6.10.6.4 contains a limit on web fatigue shear stresses. Because this is the point of maximum moment, the shear stress is low and will not control. This check will be illustrated at the point of maximum negative moment.
Compute Flexural Capacity at Point of Maximum Negative Moment \((x = 36.0 \text{ m})\)

Table 12.7.10.1.1-2 - Factored Moments (Strength I; values in kN-m):

<table>
<thead>
<tr>
<th>Force Effect Unfactored Values</th>
<th>(\eta\gamma)</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>(M_{DL1})</td>
<td>-4043</td>
<td>(0.95)(1.25)</td>
</tr>
<tr>
<td>(M_{DL2P})</td>
<td>-696</td>
<td>(0.95)(1.25)</td>
</tr>
<tr>
<td>(M_{DL2W})</td>
<td>-680</td>
<td>(0.95)(1.50)</td>
</tr>
<tr>
<td>(M_{LL})</td>
<td>-3274</td>
<td>(0.95)(1.75)</td>
</tr>
</tbody>
</table>

Compactness:

The section at the interior support was already determined to be non-compact (see calculations for positive moment section).

Compute the stresses in the flanges due to the factored loads (Strength I):

Compression flange:

\[
f_c = \left( \frac{M_{DL1}}{S_{cDL1}} \% \frac{M_{DL2}}{S_{cDL2}} \% \frac{M_{LL}}{S_{cLL}} \right) \gamma \quad \text{340 MPa}
\]

Tension flange:

\[
f_t = \left( \frac{M_{DL1}}{S_{tDL1}} \% \frac{M_{DL2}}{S_{tDL2}} \% \frac{M_{LL}}{S_{tLL}} \right) \gamma \quad \text{321 MPa}
\]

Depth of Web in Compression at Elastic Moment, \(D_c\):

Article S6.10.3.1.4 and the accompanying commentary provides some guidance for the calculation of the depth of web in compression at the elastic design moment. Below is a variation on the method for calculation of \(D_c\) as given in SC6.10.3.1.4a.

If the section at the location under consideration reaches the yield moment under the factored loads, the value of \(D_c\) will be based on the yield moment, i.e., the depth of the web in compression in the elastic range. The stress in the compression flange is taken as the least of the yield stress and the stress based on the factored loads.

\[
M_y = M_{DL1} \% M_{DL2} \% M_{AD}
\]

\[
M_y = M_{DL1} \% M_{DL2} \% S_{LL} \left( F_{yf} & \frac{M_{DL1}}{S_{DL1}} & \frac{M_{DL2}}{S_{DL2}} \right)
\]

(computed for each flange, the minimum \(M_y\) controls)
The depth of the web in compression, $D_c$, can be calculated based on a linear stress distribution between the compression and tension flanges at the yield moment. Thus, $D_c$ is calculated by the following expression:

$$D_c = \frac{df_c}{(f_c \% f_t)} & t_c$$

Because the compression flange has not yielded, the depth of web in compression, $D_c$, is based on the factored stresses:

Compute $D_c$:

$$D_c = \frac{df_c}{(f_c \% f_t)} & t_c = (1425 \% 42 \% 48)(340) \frac{(340 \% 321)}{48} = 731 \text{ mm}$$

Check the web slenderness as required by S6.10.2.2:

$$\frac{2D_c}{t_w} \# 6.77 \sqrt{\frac{E}{f_c}} \quad (S6.10.2.2-1)$$

If the above slenderness is not met, longitudinal stiffeners would be required and the web slenderness would be limited to $13.54(E/f_c)^{0.5}$.

$$\frac{2D_c}{t_w} = \frac{2(731)}{10} = 146$$

$$6.77 \sqrt{\frac{E}{f_c}} = 6.77 \sqrt{\frac{200000}{340}} = 164 > 146 \quad (OK)$$

Check compression flange slenderness:

$$\frac{b_f}{2t_f} \# 12 \quad (S6.10.4.1.4-1)$$

$$\frac{b_f}{2t_f} = \frac{460}{2(48)} = 4.79 \quad (OK)$$
Check compression flange bracing:  

\[ L_b \leq 1.76r_t \sqrt{\frac{E}{F_{yc}}} \]  

where \( r_t \) is the minimum (within the braced length) radius of gyration of the compression flange plus one third of the depth of web in compression, \( D_c \). Aside from the analysis point being considered, the nearest adjacent brace point for the compression flange is located at a distance of 28.8 meters (Section "D") from the centerline of bearing at the abutment. The needed data for that point is as follows:

Table 12.7.10.10.1-3 - Factored Moments at \( X = 28.8 \text{ m} \) (Strength I, values in kN\( \cdot \)m):

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>( \eta \gamma )</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>MDL1</td>
<td>1089</td>
<td>(0.95)(1.25)</td>
<td>1293</td>
</tr>
<tr>
<td>MDLP</td>
<td>162</td>
<td>(0.95)(1.25)</td>
<td>192</td>
</tr>
<tr>
<td>MDLW</td>
<td>158</td>
<td>(0.95)(1.50)</td>
<td>225</td>
</tr>
<tr>
<td>MLL</td>
<td>1555</td>
<td>(0.95)(1.75)</td>
<td>2585</td>
</tr>
</tbody>
</table>

Stresses and section dimensions at \( X = 28.8 \text{ m} \) (Section "D"):

\( f_c = 224 \text{ MPa} \)
\( f_t = 205 \text{ MPa} \)
\( D_c = 741 \text{ mm} \) (based on factored stresses above)
\( t_{fc} = 23 \text{ mm} \)
\( b_{fc} = 460 \text{ mm} \)
\( t_w = 10 \text{ mm} \)

\[ r_t \leq \frac{I_{cf/\gamma/3D_c}}{A_{cf/\gamma/3D_c}} \]

The moment of inertia of 1/3 of the depth of web in compression about the vertical axis is typically negligible, therefore:

\[ 1.76r_t \sqrt{\frac{E}{F_{yc}}} \leq 1.76(120)\sqrt{\frac{200000}{345}} \leq 5085 \text{ mm} \]

But, the unbraced length, \( L_b \), was assumed to be 7200 mm. Therefore, lateral torsional buckling controls.
Lateral Torsional Buckling of Compression Flanges (S6.10.4.2.5):

Check

\[ L_b \# 4.44 \sqrt[4]{\frac{E}{F_{yc}}} \]

\[ 4.44 \sqrt[4]{\frac{E}{F_{yc}}} \quad 4.44(120) \sqrt[4]{\frac{200000}{345}} \quad 12828 \text{ mm (OK)} \]

Therefore, Equation S6.10.4.2.5a-1 applies:

\[ F_n \cdot C_b R_b R_h F_{yc} \left[ 1.33 \& 0.187 \left( \frac{L_b}{r_t} \right) \sqrt{\frac{F_{yc}}{E}} \right] \# R_b R_h F_{yc} \]

where:

\[ C_b' = 1.75 \& 1.05 \left( \frac{P_l}{P_h} \right) \% 0.3 \left( \frac{P_l}{P_h} \right)^2 \# 2.3 \] \hspace{1cm} (S6.10.4.2.5a-4)

At \( X = 28.8 \text{ m} \):
\( f_c = 224 \text{ MPa} \)
\( A_{cf} = (23)(460) = 10580 \text{ mm}^2 \) (Section “D”)

thus,
\[ P_l = (224 \text{ N/mm}^2) (10580 \text{ mm}^2) \]
\[ = 2.37E6 \text{ N} \]

For the brace point located at the analysis point (\( x = 36 \text{ m}; \) Section “F”):
\( f_c = 340 \text{ MPa} \)
\( A_{cf} = (48)(460) = 22080 \text{ mm}^2 \)

thus,
\[ P_h = (340 \text{ N/mm}^2) (22080 \text{ mm}^2) \]
\[ = 7.50E6 \text{ N} \]

\[ C_b' = 1.75 \& 1.05 \left( \frac{2.37}{7.50} \right) \% 0.3 \left( \frac{2.37}{7.50} \right)^2 \# 1.45 \# 2.3 \]

Therefore, use \( C_b = 1.45 \)

Compute the flange stress reduction factor, \( R_b \):
Check \( \frac{2D_c}{t_w} \# \lambda_b \sqrt{\frac{E}{f_c}} \)  
(S6.10.4.3.2a-1)

\[
\frac{2D_c}{t_w} \cdot \frac{2(731)}{10} = 146
\]

\( \lambda_b \) ' 5.76 if \( D_c \# \frac{D}{2} \)

\( \lambda_b \) ' 4.64 if \( D_c > \frac{D}{2} \)

therefore, \( \lambda_b \) ' 4.64

\( D_c = 731 \text{ mm} \)

\[
\frac{D}{2} = \frac{1425}{2} = 713 \text{ mm}
\]

\[
\lambda_b \sqrt{\frac{E}{f_c}} \cdot 4.64 \sqrt{\frac{200000}{345}} \quad 112 < 146
\]

Therefore, \( R_b \leq 1.0 \)

So,

\[
R_b \cdot 1 \cdot \left( \frac{a_r}{1200 \%300 a_r} \right) \left( \frac{2D_c}{t_w} \# \lambda_b \sqrt{\frac{E}{f_c}} \right)
\]

where:

\[
a_r = \frac{2D_c t_w}{A_{fc}} = \frac{2(731)(10)}{22080} = 0.66
\]

(S6.10.4.3.2a-6)

\[
R_b = 1 \& \left( \frac{0.66}{1200 \%300(0.66)} \right)(146 \& 112) = 0.98
\]

for the compression flange only. \( R_b \) for the tension flange is 1.0 (S6.10.5.4.2b).

\[
F_n = (1.45)(0.98)(1.0)(345)
\]

\[
\cdot 424 \text{ MPa} \# R_b R_n F_{yc}
\]

So,

\[
R_b R_n F_{yc} = (0.997)(1.0)(345) = 344 \text{ MPa}
\]

controls.

\[
F_r = \phi_r F_n = (1.0)(344) = 344 \text{ MPa}
\]
and the required flexural resistance, $F_{u}$, is 340 MPa which is approximately one percent less than the actual resistance of 344 MPa, therefore, this section is adequate for flexure at the pier.

**Check the Web Fatigue Stresses at the Point of Maximum Negative Moment ($x = 36$ m), Article S6.10.6.3:**

**Flexural Fatigue Stress:**

Table 12.7.10.1-4 - Stress Due to the Unfactored Dead Load Plus Twice the Factored Fatigue Live Load

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>$\gamma$</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{DL1}$</td>
<td>-4043</td>
<td>(1.0)</td>
<td>-4043</td>
</tr>
<tr>
<td>$M_{DL2P}$</td>
<td>-696</td>
<td>(1.0)</td>
<td>-696</td>
</tr>
<tr>
<td>$M_{DL2W}$</td>
<td>-680</td>
<td>(1.0)</td>
<td>-680</td>
</tr>
<tr>
<td>$M_{LL}$</td>
<td>-541</td>
<td>(2)(0.75)</td>
<td>-812</td>
</tr>
</tbody>
</table>

$$f_c ' = \left( \frac{M_{DL1}}{S_{cDL1}} \right) \% \left( \frac{M_{DL2}}{S_{cDL2}} \right) \% \left( \frac{2.0M_{LL}}{S_{cLL}} \right)' = 179 \text{ MPa}$$

$$f_t ' = \left( \frac{M_{DL1}}{S_{tDL1}} \right) \% \left( \frac{M_{DL2}}{S_{tDL2}} \right) \% \left( \frac{2.0M_{LL}}{S_{tLL}} \right)' = 180 \text{ MPa}$$

$$d = 42 + 1425 + 48 = 1515$$

$$t_{fc} = 48$$

$$D_c = \frac{df_c}{(f_c \% f_t)} \& t_{fc} = \frac{(1515)(179)}{(179 \% 180)} \& 48 = 707 \text{ mm}$$

Check:

$$\frac{2D_c}{t_w} \# 5.70 \sqrt{\frac{E}{F_{yw}}}$$

$$\frac{2D_c}{t_w} = \frac{2(707)}{10} = 141$$

$$5.70 \sqrt{\frac{E}{F_{yw}}} = 5.70 \sqrt{\frac{200000}{345}} = 137 < 141$$

Therefore,

$$32.5E \left( \frac{t_w}{2D_c} \right)^2 \quad \text{(S6.10.6.3-2)}$$
So,

\[ f_{cf} \leq 32.5 \times 000 \left( \frac{10}{2(707)} \right)^2 = 325 \text{ MPa} \]

325 MPa > 179 MPa  (OK)

**Check Web Fatigue Shear Stress at X = 36.0 m (S6.10.6.4)**

Transversely stiffened homogeneous sections with or without longitudinal stiffeners must satisfy:

\[ v_{cf} \geq 0.58 C F_{yw} \]  

(S6.10.6.4-1)

The shear stress, \( v_{cf} \), is calculated on the basis of the shear due to the unfactored permanent load and twice the factored fatigue load (per Article S6.10.4.2):

\[ \begin{align*}
V_{DL1} &= -490 \text{ kN} \\
V_{DL2} &= -88 \text{ kN} - 86 \text{ kN} = -174 \text{ kN} \\
V_{fatigue} &= 0.75(2.0)(-219) = -329 \text{ kN}
\end{align*} \]

therefore, \( V_{Total} = -993 \text{ kN} \)

\[ \frac{v_{cf}}{A_{web}} \left( \frac{993 \text{ kN}(1000 \text{ N/kN})}{(1425)(10)} \right) = 69.7 \text{ MPa} \]

The maximum stiffener spacing, \( d_o \), is assumed for the calculation of the web buckling coefficient, \( C \). \( d_o \), therefore, is taken as the least of:

\[ d_o \geq D \left( \frac{260}{(D/t_w)} \right)^{\frac{1}{3}} \]  

(S6.10.7.3.2-2)

and,

\[ d_o \geq 3D \]  

(see S6.10.7.1)

where:

\[ \begin{align*}
D &= \text{ web depth } = 1425 \text{ mm} \\
t_w &= \text{ web thickness } = 10 \text{ mm}
\end{align*} \]

So,

\[ d_o \geq 1425 \left( \frac{260}{(1425/10)} \right)^{\frac{1}{3}} \approx 4744 \]

But, the maximum allowable stiffener spacing for an interior panel is controlled by three times the depth of the web or 4275 mm
Calculate the stiffener spacing coefficient, $k$:

$$k' = 5 \% \cdot \frac{5}{\left( \frac{d_o}{D} \right)^2} \quad (S6.10.7.3.3a-8)$$

$$k' = 5 \% \cdot \frac{5}{\left( \frac{4275}{1425} \right)^2} = 5.56$$

To determine the appropriate equation for the ratio of shear buckling stress to shear yield strength, $C$, we need to compare the web slenderness to the limiting values:

Check \( \frac{D}{t_w} \# 1.10 \sqrt{\frac{E_k}{F_{yw}}} \)

\[ \frac{D}{t_w} = \frac{1425}{10} = 142.5 \]

\[ 1.10 \sqrt{\frac{E_k}{F_{yw}}} = 142.5 \]

\[ 1.10 \sqrt{\frac{E_k}{F_{yw}}} = 62.4 \]

Check \( \sqrt{\frac{E_k}{F_{yw}}} \# 1.38 \sqrt{\frac{E_k}{F_{yw}}} \)

\[ 1.38 \sqrt{\frac{E_k}{F_{yw}}} = 78.3 < 142.5 \]

Therefore,

$$C = \frac{1.52}{E_k} \left( \frac{F_{yw}}{D} \right)^2 \quad \frac{1.52}{F_{yw}} \left( \frac{200000(5.56)}{345} \right) = 0.241$$

\[ (S6.10.7.3.3a-7) \]

And the fatigue web shear stress limit is then:

$$v_{c,\text{max}} = 0.58 CF_{yw} \quad (S6.10.4.4-1)$$

$$= 0.58(0.241)(345)$$

$$= 48.3 \text{ MPa} < 69.7 \text{ MPa} \text{ (No Good)}$$
Thus, this provision must be checked with a more realistic stiffener spacing which has been determined as a part of the shear design (further along in this example) to be 1000 mm at this analysis point. So,

\[
k' = 5 \%
\frac{5}{\left(\frac{1000}{1425}\right)^2} = 15.15
\]

\[
1.10 \sqrt{\frac{E_k}{F_{yw}}} = 1.10 \sqrt{\frac{200000(15.15)}{345}} = 103.1 < 142.5
\]

Check

\[
1.38 \sqrt{\frac{E_k}{F_{yw}}} \cdot \frac{D}{t_w} \cdot 1.38 \sqrt{\frac{E_k}{F_{yw}}} = 1.38 \sqrt{\frac{200000(15.15)}{345}} = 129 < 142.5
\]

Therefore,

\[
C = \left(\frac{D}{t_w}\right)^2 \cdot \frac{E_k}{F_{yw}} = 1.52 \left(\frac{200000(15.15)}{345}\right) = 0.657
\]

And the fatigue web shear stress limit is then:

\[
v_{cfmax} = 0.58 \cdot C F_{yw}
\]

\[
= 0.58(0.657)(345) = 132 \text{ MPa} > 69.7 \text{ MPa (OK)}
\]
12.7.10.10.2 Specification Check: Shear

**Compute and Check the Shear Capacity, \( V_r \), at the abutment:**
\((x = 0 \text{ m})\)

Table 12.7.10.10.2-1 - Factored Shear Load (Strength I; values in kN):

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>( \eta \gamma )</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_{DL1} )</td>
<td>256.4</td>
<td>(0.95)(1.25)</td>
<td>304.5</td>
</tr>
<tr>
<td>( V_{DL2P} )</td>
<td>49.3</td>
<td>(0.95)(1.25)</td>
<td>58.5</td>
</tr>
<tr>
<td>( V_{DL2W} )</td>
<td>48.1</td>
<td>(0.95)(1.50)</td>
<td>68.5</td>
</tr>
<tr>
<td>( V_{LL} )</td>
<td>532.1</td>
<td>(0.95)(1.75)</td>
<td>884.6</td>
</tr>
</tbody>
</table>

**Shear Capacity of an Exterior Panel:**

\[ V_n = C V_p \]  \hspace{1cm} (S.6.10.7.3.3c-1)

where:

\[ V_p = 0.58 F_{yw} D t_w \]  \hspace{1cm} (S.6.10.7.3.3c-2)

for which:

\( F_{yw} = 345 \text{ MPa} \)
\( D = 1425 \text{ mm} \)
\( t_w = 12 \text{ mm} \)

Therefore, the web yield strength is:

\[ V_p = 0.58(345)(12)(1 \text{ kN/1000 N}) \]
\[ = 3422 \text{ kN} \]

Per Article S.6.10.7.3.3c, the maximum stiffener spacing for an exterior panel is:

\[ 1.5D = 1.5(1425) = 2138 \text{ mm} \]

Assume a stiffener spacing of 2100 mm

Calculate \( C \):

\[
\frac{k}{5} \left( \frac{d_o}{D} \right)^2 \cdot 5 \% \left( \frac{5}{2100} \right)^2 \cdot 7.30
\]
Check \( \frac{D}{t_w} # 1.10 \sqrt{\frac{E_k}{F_{yw}}} \)

\[
\frac{D}{t_w} = \frac{1425}{12} = 119
\]

\[
1.10 \sqrt{\frac{E_k}{F_{yw}}} = 1.10 \sqrt{\frac{200000(7.30)}{345}} = 71.6 < 119
\]

Therefore, check \( \frac{D}{t_w} # 1.38 \sqrt{\frac{E_k}{F_{yw}}} \)

\[
1.38 \sqrt{\frac{E_k}{F_{yw}}} = 1.38 \sqrt{\frac{200000(7.30)}{345}} = 90 < 119
\]

therefore,

\[
C \cdot \left( \frac{D}{t_w} \right)^2 \left( \frac{E_k}{F_{yw}} \right) = 1.52 \left( \frac{200000(7.30)}{345} \right) = 0.45
\]

Finally,

\[
V_n = CV_p = 0.45(3422 \text{ kN}) = 1540 \text{ kN}
\]

and,

\[
V_r = \varphi V_n = 1.0(1540) = 1540 \text{ kN} > 1316 \text{ kN} \quad \text{(OK)}
\]

**Compute the Factored Shear Resistance at the Pier**

\((x = 36 \text{ m}):\)

Table 12.7.10.10.2-2 - Factored Shear Load (Strength I; values in kN):

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>( \eta \gamma )</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_{DL1} )</td>
<td>-490</td>
<td>(0.95)(1.25)</td>
<td>-582</td>
</tr>
<tr>
<td>( V_{DL2P} )</td>
<td>-88</td>
<td>(0.95)(1.25)</td>
<td>-105</td>
</tr>
<tr>
<td>( V_{DL2W} )</td>
<td>-86</td>
<td>(0.95)(1.50)</td>
<td>-123</td>
</tr>
<tr>
<td>( V_{LL} )</td>
<td>-626</td>
<td>(0.95)(1.75)</td>
<td>-1041</td>
</tr>
</tbody>
</table>

\[ -1851 \text{ kN} \]
Shear Capacity of an Interior Panel of a Non-Compact Section (S6.10.7.3.3b):

Because \( f_u > 0.75 \phi_f F_y \) (340 MPa > 259 MPa):

\[
V_n' = R V_p \left[ C \% \frac{0.87(1 + C)}{1 \% \left( \frac{d_o}{D} \right)^2} \right] CV_p
\]

(S6.10.7.3.3b-2)

where:

\[
R' = 0.6 \% 0.4 \left( \frac{F_r \& f_u}{F_r \& 0.75 \phi_f F_y} \right) \% 1.0
\]

(S6.10.7.3.3b-3)

\[
V_p = 0.58 F_{yw} D t_w
\]

(S6.10.7.3.3a-4)

Calculate \( R \):

From previous calculations for flexure:

\[
F_r = 344 \text{ MPa}
\]

\[
F_u = 340 \text{ MPa}
\]

\[
F_y = 345 \text{ MPa}
\]

\[
\phi_f = 1.0
\]

Therefore,

\[
R' = 0.6 \% 0.4 \left( \frac{344 \& 340}{344 \& 0.75(1.0)345} \right) \% 1.0 = 0.619 \% 1.0
\]

Calculate \( V_p \):

\[
V_p = 0.58 F_{yw} D t_w
\]

= 0.58(345)(1425)(10)(1 kN/1000 N)

= 2851 kN

Compute the maximum stiffener spacing, \( d_o \), as the lesser of:

\[
C \frac{d_{o_{max}}}{D} = \left[ \frac{260}{(D/t_w)} \right] \%
\]

(S6.10.7.3.2-2)

\[
C = 3.0 D \text{ (for interior panels without longitudinal stiffeners)}
\]

So,

\[
\frac{d_{o_{max}}}{3} = 1425 \left[ \frac{260}{(1425/10)} \right] \%
\]

Try a spacing of 4200 mm:
Compute $C$:

\[ k' = 5\% \frac{5}{\left(\frac{d_o}{D}\right)^2} \frac{5}{\left(\frac{4200}{1425}\right)^2} = 5.58 \]

Check $\frac{D}{t_w} \# 1.38 \sqrt{\frac{E_k}{F_{yw}}}$

\[ \frac{D}{t_w} = \frac{1425}{10} = 143 \]

\[ 1.38 \sqrt{\frac{E_k}{F_{yw}}} = 1.38 \sqrt{\frac{200000(5.58)}{345}} = 78.5 < 143 \]

Therefore,

\[ C = \frac{1.52}{\left(\frac{D}{t_w}\right)^2} \frac{E_k}{F_{yw}} = \frac{1.52}{\left(\frac{1425}{10}\right)^2} \frac{200000(5.58)}{345} = 0.242 \]

\[ V_n = RV_p \left[ C \% \frac{0.87(1 + C)}{\sqrt{1 \% \left(\frac{d_o}{D}\right)^2}} \right] C V_p \]

\[ V_n = (0.619)(2851) \left[ 0.242 \% \frac{0.87(1 + 0.242)}{\sqrt{1 \% \left(\frac{4200}{1425}\right)^2}} \right] \]

\[ = (0.619)(2851)(0.453) \]

\[ = 801 \text{kN} > 0.242(2851) = 690 \text{kN} \text{ (OK)} \]

\[ V_r = \phi V_n = (1.0)(801) = 801 \text{kN} \]

But, $801 \text{kN} < V_u = 1851 \text{kN}$ so the section fails in shear.
Try decreasing the stiffener spacing to 1000 mm:

\[ k' = \frac{5}{5} \frac{5}{\left(\frac{d_o}{D}\right)^2} \frac{5}{\left(\frac{1000}{1425}\right)^2} = 15.15 \]

Check \( \frac{D}{t_w} \) # 1.38 \( \sqrt{\frac{E_k}{F_{yw}}} \)

\[ \frac{D}{t_w} = \frac{1425}{10} = 143 \]

\[ 1.38 \sqrt{\frac{E_k}{F_{yw}}} = 1.38 \sqrt{\frac{200000(15.15)}{345}} = 129 < 143 \]

Therefore,

\[ C = \frac{1.52}{\left(\frac{D}{t_w}\right)^2} \left(\frac{E_k}{F_{yw}}\right) = \frac{1.52}{\left(\frac{1425}{10}\right)^2} \left(\frac{200000(15.15)}{345}\right) = 0.657 \]

\[ V_n = RV_p \left[ C \frac{0.87(1 + C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right] \]

\[ V_n = (0.619)(2851) \left[ 0.242 \frac{0.87(1 + 0.657)}{\sqrt{1 + \left(\frac{1000}{1425}\right)^2}} \right] = (0.619)(2851)(0.901) = 1590 \text{ kN} < 0.657(2851) = 1873 \text{ kN} \]

\( (1873 \text{ kN controls}) \)

\[ V_r = \phi V_n = (1.0)(1873) = 1873 \text{ kN} > 1851 \text{ kN} \text{ (OK)} \]
12.7.10.10.3 Specification Check: Fatigue

For a plate girder being examined in this example, fatigue is a concern for the following types of details:

**Category B Details:**

Article S6.6.1.2.3 of the LRFD Specification states that for typical design problems, only details of Category C or worse need to be checked. Experience has shown that detail Categories A through B rarely govern other than, perhaps, for an unusual design problem.

- Base and weld metal of full penetration groove welded splices with transitions in width or thickness ground to provide slopes no greater than 1.0 to 2.5 (for base metal other than Grades 690/690W would be category B*). See Figure S6.6.1.2.3-1(11).

- Base and weld metal in components without attachments connected by continuous fillet welds parallel to the direction of the applied stress (flange to web fillet welds) See Figure S6.6.1.2.3-1(4).

**Category C and C' Details:**

- Base metal adjacent to stud-type shear connectors. See Figure S6.6.1.2.3-1(18).

- Base metal at toe of transverse stiffener-to-flange and transverse stiffener-to-web welds (this is a Category C* detail which has a higher value of the constant amplitude fatigue threshold). See Figure S6.6.1.2.3-1(6).

Fatigue stresses in the tension flange in negative moment regions will be controlled by Category “C” detail of the stud connectors. The allowable fatigue stress range may be further limited by the allowable fatigue stress range in the longitudinal reinforcing in the slab as given in Article S5.5.3.1.

In positive moment regions, the fatigue stresses at the toe of the transverse stiffener or cross-frame connection plate welds are limited to those appropriate for Category “C*”. Locations of the flange transitions are considered for Category “B” where the flange can undergo reversal or is in tension.

The allowable fatigue stress is a function of not only the detail Category, but also the number of cycles of loading which is now calculated based on the ADTT rather than assumed for a particular class of highway.
For an actual design, the fatigue should be checked at the critical locations for all of the appropriate detail categories. In this particular example, however, the fatigue checks will only be shown for the following selected details:

C The toe of the transverse stiffener/cross-frame connection plate at the point of maximum positive moment (14 400 mm from the centerline of bearing at the abutment).

C The shear connectors at the point of maximum negative moment over the interior support.

C The fatigue stress range in the slab reinforcing at the point of maximum negative moment.

Fatigue Load - Resistance: Basic Equation

Because $\phi$ and $\eta$ are taken as 1.0 at the fatigue limit state the base equation that must be satisfied for fatigue is as follows:

$$\gamma (\Delta f)^{\phi} (\Delta F)^{\eta} \quad (S6.6.1.2.2-1)$$

Check fatigue stress for a transverse stiffener/cross-frame connection plate located at the point of maximum positive moment ($x = 14.4$ m)

Typically, the stiffeners can be terminated at a distance 4 to 6 times the flange thickness from the inside face of the bottom flange to facilitate painting and to reduce the possibility of a fatigue crack forming in the weld during shipping and handling. Stiffeners which also act as cross-frame connection plates, however, should be welded to both flanges. The neutral axis for the live load section (i.e., the "n-section") is located at a distance of 1264 mm from the bottom of the beam. Because the connection is located on the inside face of the flange, this value is reduced by the flange thickness although some may wish to take the slightly more conservative approach of computing stresses at the bottom of the beam. Therefore:

$$c = 1264 - t_b = 1234 \text{ mm}$$

$$I_{LL} = 2.784 \times 10^10 \text{ mm}^4$$

The range of unfactored fatigue moments is:

$$\Delta M_{LL} = 820.5 - (-216.3) = 1037 \text{ kN}\cdot\text{m}$$

$$\gamma \Delta M_{LL} = (0.75)(1037) = 778 \text{ kN}\cdot\text{m}$$

$$\gamma \Delta f = \frac{778(1E6)(1234)}{2.784E10} \approx 34.5 \text{ MPa}$$
Note that the "positive moment" section properties were used to compute the stress due to the range of fatigue moments. Although the lower moment is negative, it was determined that the stress in the slab due to the composite dead load moment and the negative fatigue live load moment is compressive. As a result, the net effect is that the section is in positive flexure for the entire range of fatigue moments. In some locations along the girder, it would be appropriate to use the positive section properties for the larger fatigue moment and the negative section properties for the smaller fatigue moment.

Compute the fatigue resistance (Category C):

**Fatigue Constants (Tables S6.6.1.2.5-1 and 3):**

\[ A = 14.4 \times 10^{11} \text{ MPa}^3 \]

\[ \Delta F_{TH} = 82.7 \text{ MPa} \]

Determine the number of cycles per truck passage, \( n \) (Table S6.6.1.2.5-2):

For a continuous girder with a span length greater than 12 000 mm, at a location greater than ten percent of the span length away from an interior support, the "equivalent" number of cycles per truck passage, \( n = 1.0 \).

Compute the number of cycles in the design life of the bridge (75 years):

\[ ADTT_{SL} = p(ADTT) \quad (S3.6.1.4.2-1) \]

where:

\[ p = \text{percentage of trucks in a single design lane} = 0.85 \quad (\text{Table S3.6.1.4.2-1}) \]

\[ ADTT = 500 \text{ (assumed)} \]

Therefore,

\[ ADTT_{SL} = (0.85)(500) = 425 \]

So,

\[ N = 365(75)(n)(ADTT_{SL}) \quad (S6.6.1.2.5-2) \]

\[ = 365(75)(1)(425) = 11.6 \text{ million cycles} \]
Compute the resistance:

\[
(\Delta F_n) ' \left( \frac{A}{N} \right)^{\frac{1}{3}} \cdot \left( \frac{14.4 \times 10^{11}}{11.6 \times 10^{6}} \right)^{\frac{1}{3}} \cdot 49.9 \text{ MPa (controls)}
\]

\[
\frac{1}{2}(\Delta F)_{TH} ' = 41.4 \text{ MPa}
\]

49.9 MPa > 34.5 MPa (OK)

Check a shear connector for fatigue at the interior support

(Category C):

The live load section modulus for the tension flange, \( S_{LL} \), is 4.341E7 mm\(^3\).

Calculate the factored fatigue range of moments:

\[
\gamma \Delta M_{LL} = (0.75) (540.7) = 406 \text{ kN}\cdot\text{m}
\]

So,

\[
\gamma \Delta f ' = \frac{406 \times 10^6}{4.341 \times 10^7} = 9.4 \text{ MPa}
\]

Fatigue Constants for Category C:

\[
A = 4.4 \times 10^{11} \text{ MPa}^3
\]

\[
\Delta F_{TH} = 69.0 \text{ MPa}
\]

Determine the number of cycles per truck passage, \( n \) (S6.6.1.2.5):

For a continuous girder with a span length greater than 12 000 mm, at a location greater within ten percent of the span length away from an interior support, the “equivalent” number of cycles per truck passage, \( n = 1.5 \).
Compute the number of cycles in the design life of the bridge (75 years):

\[ ADTT_{sl} = 425 \]

So,

\[ N = 365(75)(n)(ADTT_{sl}) = 365(75)(1.5)(425) = 17.5 \text{ million cycles} \]

Compute the resistance:

\[
\left( \frac{A}{N} \right)^{\frac{1}{3}} \left( \frac{14.4E11}{17.5E6} \right)^{\frac{1}{3}} \frac{1}{2}(\Delta F)_{TH} = 43.5 \text{ MPa (controls)}
\]

\[
\frac{1}{2}(\Delta F)_{TH} = 34.5 \text{ MPa}
\]

43.5 MPa > 9.4 MPa (OK)

**Check fatigue stresses in the slab reinforcement at the pier**

(X = 36 m):

Because the slab steel is always in tension at this location, a fatigue check is required. The maximum stress in the slab steel is given by the following equation:

\[
f_f' = 145 \& 0.33f_{\text{min}} \% 55\left( \frac{f}{h} \right)
\]

(S5.5.3.2-1)

The unfactored moments at the pier are as follows:

\[
M_{DL2P} = -696.0
\]
\[
M_{DL2W} = -679.6
\]
\[
M_{LLf} = -540.7
\]

(570.7 is the range of fatigue moments because there is no positive fatigue moment at the support)

Thus,

\[
M_{DL2} = (-696) + (-680) = -1376 \text{ kN@}
\]

The section modulus for the slab steel is 3.565E7 mm³ (for both dead and live load).
Compute the stress due to the unfactored permanent load:

\[ f_{\text{perm}} = \frac{1376(1 \times 10^6)}{3.565 \times 10^7} = 0.386 \text{ MPa} \quad \text{(tension)} \]

Compute the stress due to the factored fatigue live load:

\[ f_f = \frac{(0.75)(540.7)(1 \times 10^6)}{3.565 \times 10^7} = 0.0114 \text{ MPa} \quad \text{(tension)} \]

Compute \( f_{\text{min}} \), which is the stress due to the unfactored permanent plus the factored fatigue moment which causes the most compression. Because the range of factored fatigue moments for fatigue is 0 to 406 kN\(\cdot\)m, the zero value is selected because it induces the least tension.

Therefore,

\[ f_{\text{min}} = 0.386 + 0 \quad \text{(positive because tensile)} \]

As a result, the allowable fatigue stress range in the slab steel can be computed as:

\[ f_f' = 145 \times 0.33(38.6) = 149 \text{ MPa} > 11.4 \text{ MPa} \quad (\text{OK}) \]

12.7.10.10.4 Specification Check: Shear Connectors

This girder being designed in this example is composite for the entire length of the bridge. Thus, shear connectors are required in both the positive and negative flexural regions. Figure 1 below shows the regions for shear connector placement for the strength limit state as required by Article S6.10.7.4.4.

![Shear Connector Regions](image)

(Note: Location of \( M_{\text{max}} \) can be based on the live load)

Figure 12.7.10.10.4-1 - Shear Connector Regions
For this problem, the point of maximum live load moment is at X = 14.4 meters and the point of dead load contraflexure for composite dead load is around X = 25.8 meters.

Determine number of shear connectors required for the strength limit state:

At the strength limit state, the number of shear connectors required in each region is determined by the following equations:

\[ Q_r = \phi_{sc} Q_n \quad (S6.10.7.4.4a-1) \]

and,

\[ n' = \frac{V_h}{Q_r} \quad (S6.10.7.4.4a-2) \]

where the nominal horizontal shear force, \( V_h \), is taken as:

**Positive Moment Regions:** \( V_h \) is the least of:

\[ V_h = 0.85 f_c^* b_t s \quad (S6.10.7.4.4b-1) \]

or

\[ V_h = F_w D_t + F_{yt} t_t + F_{yc} b_{tt} \quad (S6.10.7.4.4b-2) \]

**Negative Moment Regions:**

\[ V_h = A_r F_{yr} \quad (S6.10.7.4.4b-3) \]

Region 1 (Sections A, B):

\[
0.85 f_c^* b_t s \left[ \frac{0.85(28)(2550)(200)}{1000} \right] = 12138 \text{ kN}
\]

\[ F_{yw} D_t + F_{yt} t_t + F_{yc} b_{tt} \quad (\text{Use Section B & most critical}) \]

\[ \left[ \frac{(345)(1425)(12)(345)(300)(30)(345)(300)(13)}{1000} \right] = 10350 \text{ kN} \]

\[ V_{h1} = 10350 \text{ kN} \quad \text{(controls)} \]

Region 2 (Sections B, C):

Same as for region 1, therefore, \( V_{h2} = 10350 \text{ kN} \)

Region 3 (Sections D, E, F):

\[ A_r F_{yr} = \frac{(4700 + 2400)(400)}{1000} = 2840 \text{ kN} \]
Determine the nominal capacity of a single shear connector:

\[ Q_n = 0.5A_{sc}\sqrt{f_{c}E_c} \cdot A_{sc}F_u \]  
(S6.10.7.4.4c-1)

where:

\[ d_{sc} = 22 \]
\[ A_{sc} = \frac{\pi(22)^2}{4} \cdot 380 \text{ mm}^2 \]
\[ f_{c}^* = 28 \text{ MPa} \]
\[ E_c = 25,400 \text{ MPa} \]
\[ F_u = 400 \text{ MPa (see S6.4.4)} \]

So,

\[ 0.5A_{sc}\sqrt{f_{c}E_c} \cdot 0.5(380)\sqrt{(28)(25400)}/1000 \cdot 160.2 \text{ kN} \]
\[ A_{sc}F_u \cdot (380)(400)/1000 \cdot 152 \text{ kN (controls)} \]

Then,

\[ Q_r = \phi_{sc}Q_n = (0.85)(152) \]
\[ = 129.2 \text{ kN per connector} \]

Compute the number of shear connectors in each region:

\[ n_1 \cdot n_2 \cdot \frac{V_h}{Q_r} \cdot \frac{10350}{129.2} \cdot 80.1 \text{ (round to 81)} \]

\[ n_3 \cdot \frac{V_{h3}}{Q_r} \cdot \frac{2840}{129.2} \cdot 22 \text{ (round to 24 for even rows of 3)} \]

Check the transverse spacing assuming three shear connectors per row (Article S6.10.7.4.1c). Figure 2 shows the minimum clearances for a cross-section with three rows of shear connectors.
Figure 12.7.10.4-2 - Minimum Clearances for Shear Connectors

\[ b_{r_{\text{min}}} = 2(25) \% 2 \left( \frac{22}{2} \right) \% 2(4(22)) \left( 248 > 300 \text{ (OK)} \right) \]

Compute the required pitch for the strength limit state:

Region 1 (x = 0 to 14.4 m)

\[ p' = \frac{3(14400)}{81} = 533 \]

Region 2 (x = 14.4 to 25.8 m)

\[ p' = \frac{3(11400)}{81} = 422 \]

Region 3 (x = 25.8 to 36.0 m)

\[ p' = \frac{3(10200)}{24} = 1275 \]

But the limits on the pitch are given in Article S6.10.7.4.1b as follows:

\[ 6d \# p \# 600 \text{ mm, So:} \]
\[ 6(22) \# p \# 600 \]
\[ 132 \# p \# 600 \]
Determine the maximum pitch for fatigue resistance as given by Equation S6.10.7.4.1b-1:

\[ p \geq \frac{n Z_r I}{V_{sr} Q} \]

where:

- \( n = 3 \) connectors per cross-section
- \( Z_r = \alpha d^2 + 19d^2 \) (S6.10.7.4.2-1)

for which:

- \( \alpha = 238 - 29.5 \log (N) \)
- \( N = 11.6 \) million cycles away from support
- \( N = 17.5 \) million cycles near the support
  (see fatigue calculations)

Compute \( \alpha \) for sections away from the support:

- \( \alpha = 238 - 29.5 \log(11.6E6) = 29.6 \)
- \( \alpha d^2 = 14,326 \text{ N} \) (Sections A-D)

Compute \( \alpha \) for sections near the support:

- \( \alpha = 238 - 29.5 \log (17.5 \text{ E6}) = 24.3 \)
- \( \alpha d^2 = 11,776 \text{ N} \) (Sections E, F)

Check the fatigue threshold value for \( Z_r \):

- \( Z_r = 19d^2 \)
  = \( 19(22)^2 = 9196 \text{ N} \) (does not control)

Compute the maximum pitch for Section “A”:

---

\[ b_e/n \]

**Effective Slab Area, Transformed by Modular Ratio, \( n \)**

\[ t_e/2 \]

\[ u \]

**N.A. for Live Load**

---

Figure 12.7.10.10.4-3 - Section for Computing First Moment of Transformed Area, Q
Section properties for the short-term section:

\[ I = 2.192 \times 10^{10} \]

\[ Q = \frac{b_e n}{t_s} \]

\[ A = \frac{2}{2} \% \frac{c_{if}}{S_{LLf}} \times \frac{I}{100} \% \frac{2.192 \times 10^{10}}{1.425 \times 10^{8}} \times 254 \]

\[ A = \frac{b_e n}{t_s} = \frac{(2550/8)(200)}{63750} \]

So,

\[ Q = (63750)(254) = 1.62 \times 10^7 \text{ mm}^3 \]

The maximum shear force range, \( V_{sr} \), within Section A (at \( X = 0 \)) is:

\[ V_{sr} = (0.75)(196.9 - (-23.8)) = 166 \text{ kN} \]

So,

\[ p \# \frac{nZ_I}{V_{sr} Q} = 350 \text{ mm} \]

At Section “B”:

Section properties for the short-term section:

\[ I = 2.784 \times 10^{10} \]

\[ Q = \frac{b_e n}{t_s} \]

\[ A = \frac{2}{2} \% \frac{c_{if}}{S_{LLf}} \times \frac{I}{100} \% \frac{2.784 \times 10^{10}}{1.364 \times 10^{8}} \times 304 \]

\[ A = \frac{b_e n}{t_s} = \frac{(2550/8)(200)}{63750} \]

So,

\[ Q = (63750)(304) = 1.94 \times 10^7 \text{ mm}^3 \]

The maximum shear force range, \( V_{sr} \), within Section B (at \( X = 23.4 \)) is:

\[ V_{sr} = (0.75)(177.4) = 133 \text{ kN} \]

So,

\[ p \# \frac{nZ_I}{V_{sr} Q} = 463 \text{ mm} \]

At Section “C”:

Section properties for the short-term section:
\[ I = 2.343 \times 10^{10} \]
\[ Q = \frac{A}{u} \]
\[ u = \frac{t_s}{2} \% c_{fr} \]
\[ A = b_e/n (t_s) = (2550/8)(200) = 63750 \]

So,
\[ Q = (63750)(267) = 1.70 \times 10^7 \text{ mm}^3 \]

The maximum shear force range, \( V_{sr} \), within Section C (at \( X = 25.8 \)) is:
\[ V_{sr} = (0.75)(184.2) = 138 \text{ kN} \]

So,
\[ \rho \# \frac{nZ/I}{V_{sr}Q} = \frac{3(14326)(2.343 \times 10^{10})}{138000(1.70 \times 10^7)} = 429 \text{ mm} \]

At Section “D”:

Section properties for the short-term section:

\[ I = 1.652 \times 10^{10} \]
\[ Q = \frac{A}{u} \]
\[ c = 786 \text{ mm} \] (live load N.A. to the top layer of steel)
\[ u = 786 \]
\[ A = A_r = (4700+2400) = 7100 \text{ mm}^2 \]

So,
\[ Q = (7100)(786) = 5.58 \times 10^6 \text{ mm}^3 \]

The maximum shear force range, \( V_{sr} \), within Section D (at \( X = 30.6 \)) is:
\[ V_{sr} = (0.75)(201.1) = 151 \text{ kN} \]

So,
\[ \rho \# \frac{nZ/I}{V_{sr}Q} = \frac{3(14326)(1.652 \times 10^{10})}{151000(5.58 \times 10^6)} = 842 \text{ mm} \]

At Section “E”:

Section properties for the short-term section:

\[ I = 2.086 \times 10^{10} \]
\[ Q = \frac{A}{u} \]
\[ c = 802 \text{ mm} \] (live load N.A. to the top layer of steel)
\[ u = 802 \]
\[ A_r = (4700 + 2400) = 7100 \text{ mm}^2 \]

So,

\[ Q = (7100)(802) = 5.69 \times 10^6 \text{ mm}^3 \]

The maximum shear force range, \( V_{sr} \), within Section E (at \( X = 33.0 \)) is:

\[ V_{sr} = (0.75)(209.3) = 157 \text{ kN} \]

So,

\[ \rho \geq \frac{nZ_I}{V_{sr}Q} \cdot \frac{3(11776)(2.992 \times 10^9)}{(157000)(5.69 \times 10^6)} \cdot 825 \text{ mm} \]

At Section “F”:

Section properties for the short-term section:

\[ I = 2.992 \times 10^9 \]

\[ Q = Au \]

\[ c = 839 \text{ mm} \quad \text{(live load N.A. to the top layer of steel)} \]

\[ u = 839 \]

\[ A_r = (4700 + 2400) = 7100 \text{ mm}^2 \]

So,

\[ Q = (7100)(839) = 5.96 \times 10^6 \text{ mm}^3 \]

The maximum shear force range, \( V_{sr} \), within Section F (at \( X = 36.0 \)) is:

\[ V_{sr} = (0.75)(219.1) = 164 \text{ kN} \]

So,

\[ \rho \geq \frac{nZ_I}{V_{sr}Q} \cdot \frac{3(11776)(2.992 \times 10^9)}{(164000)(5.96 \times 10^6)} \cdot 1081 \text{ mm} \]

Table 12.7.10.4-1 - Summary of Maximum Pitch Values (based on three shear connectors per cross-section):

<table>
<thead>
<tr>
<th>Section</th>
<th>Maximum Pitch</th>
<th>Limited By</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>350</td>
<td>Fatigue</td>
</tr>
<tr>
<td>( B_{start} ) to ( M_{max} )</td>
<td>460</td>
<td>Fatigue</td>
</tr>
<tr>
<td>( M_{max} ) to ( D_{start} )</td>
<td>422</td>
<td>Strength</td>
</tr>
<tr>
<td>D</td>
<td>600</td>
<td>Maximum Allowed</td>
</tr>
<tr>
<td>E</td>
<td>600</td>
<td>Maximum Allowed</td>
</tr>
<tr>
<td>F</td>
<td>600</td>
<td>Maximum Allowed</td>
</tr>
</tbody>
</table>
12.7.10.10.5 Specification Check: Transverse Stiffener Details

Assumed dimensions:

![Assumed Transverse Stiffener Dimensions](image_url)

Check the projecting width:

\[ b_t \geq 50 \% \frac{d}{30} \]  
\( (S6.10.8.1.2-1) \)

and,

\[ 16.0t_p \leq b_t \leq 0.25b_f \]  
\( (S6.10.8.1.2-2) \)

\[ d_{\text{max}} = 1425 + 42 + 48 = 1515 \quad \text{(at } X = 36 \text{ m}) \]

\[ 50 \% \frac{d}{30.0}, \quad 50 \% \frac{1515}{30}, \quad 101 < 125 \quad \text{(OK)} \]

\[ 16.0t_p = 16(15) = 240 > 125 \quad \text{(OK)} \]

\[ 0.25b_f = 0.25(460) = 115 < 125 \quad \text{(OK)} \]

Check the required moment of inertia:

\[ I_t \geq d_o t_w^3 J \]  
\( (S6.10.8.1.3-1) \)
where:

\[ J' = 2.5 \left( \frac{D_p}{d_o} \right)^2 & 2.0 \leq 0.5 \]  

(S6.10.8.1.3-2)

\[ l_t = \text{the moment of inertia about the edge of web in contact with the stiffener for single stiffeners, and about the mid-thickness of the web for stiffener pairs} \]

\[ D_p = \text{web depth (for girders without longitudinal stiffeners)} \]

Assume a single stiffener, therefore, the actual \( l_t \) is:

\[ l_t' = \frac{15(125)^3}{12} \% \left( \frac{125}{2} \right)^2 9.77 \times 10^6 \text{ mm}^4 \]

The maximum stiffener spacing is 4200 mm (approximately 3D) and the maximum web thickness is 12 mm. The required moment of inertia will first be checked against the "required" value.

\[ J' = 2.5 \left( \frac{D_p}{d_o} \right)^2 & 2.0' = 2.5 \left( \frac{1425}{4200} \right)^2 & 2.0' \leq 1.71 \]

(0.5 controls)

\[ d_o t_w' = 4200(12)^3(0.5)' = 3.63 \times 10^6 \text{ mm}^4 \]

\[ 9.77 \times 10^6 \geq 3.63 \times 10^6 \quad \text{(OK)} \]

**Check Transverse Stiffener Area:**

The shear resistance formulas used in this example were based on the assumption that the transverse stiffeners would have sufficient area to resist the vertical component of the tension field that exists in the web. Transversely stiffened webs designed under this assumption must satisfy the following minimum area requirement:

\[ A_s \geq \left[ 0.15B \frac{D}{t_w(10 - C)} \frac{V_u}{V_r} - 18 \left( \frac{F_{yw}}{F_{cr}} \right) t_w^2 \right] \]

(S6.10.8.1.4-1)

\[ F_{cr} = \frac{0.311 E}{\left( \frac{b_l}{t_p} \right)^2} \leq F_{ys} \]
Although this check should be performed at all analysis points along
the girder, only a single check of this requirement will be shown herein
for brevity. The minimum stiffener area requirement will be checked
at the pier where the following data was calculated during the shear
design:

\[
d_{0, \text{reqd.}} = 1000 \text{ mm} \\
C = 0.657 \\
V_{u} = 1851 \text{ kN} \\
V_{r} = 1873 \text{ kN} \\
t_{w} = 10 \text{ mm} \\
F_{cr} = \frac{0.311 E}{t_p} \leq F_{ys} \\
0.311 E \left(\frac{b_{t}}{t_p}\right)^2 = 895 \text{ MPa} \therefore F_{cr} = 250 \text{ MPa} \\
\begin{align*}
A_{s} & \geq \left[ 0.15 B \frac{D}{t_w} (1.0 & C) \frac{V_{u}}{V_{r}} & 18 \left( \frac{F_{yw} t_w^2}{F_{cr}} \right) \\
& \left[ 0.15(2.4)(1425) & (10 & 0.657) \frac{1851}{1873} & 18 \left( \frac{345}{250} \right)(10)^2 \\
& \geq 648 \text{ mm}^2 \right] \\
A_{s, \text{prov}} & = (125)(15) = 1875 \text{ mm}^2 \text{ (OK)}
\end{align*}
\]

Because the required area was calculated as a negative value, the
stiffener need only satisfy the projecting width and stiffness
requirements given in Articles S6.10.8.1.2 and S6.10.8.1.3,
respectively. The negative value was obtained because the web
alone is sufficient to resist the vertical component of the tension field.
**Bearing Stiffener Design (S6.10.8.2)**

Design the stiffeners at the abutment (X = 0)

Table 12.7.10.10.5-1 - Factored Reactions (Strength I; values in kN):

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>( \eta \gamma )</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R_{DL1} )</td>
<td>256</td>
<td>(0.95)(1.25)</td>
<td>304</td>
</tr>
<tr>
<td>( R_{DL2P} )</td>
<td>49</td>
<td>(0.95)(1.25)</td>
<td>58</td>
</tr>
<tr>
<td>( R_{DL2W} )</td>
<td>48</td>
<td>(0.95)(1.50)</td>
<td>68</td>
</tr>
<tr>
<td>( R_{LL} )</td>
<td>532</td>
<td>(0.95)(1.75)</td>
<td>884</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1314 kN</td>
</tr>
</tbody>
</table>

Try a single pair of 17 mm thick bearing stiffeners (one on each side of the web). The yield stress of the bearing stiffener is assumed to be 345 MPa (Note that for the transverse stiffeners, the yield stress was assumed to be 250 MPa). Compute the maximum width:

\[
b_t \geq 0.48 t_p \sqrt{\frac{E}{F_{ys}}} \geq 0.48(17) \sqrt{\frac{200000}{345}} = 196
\]  
(S6.10.8.2.2-1)

Try a bearing stiffener width of 140 mm. The assumed clip width is 13 mm and the bearing stiffener width should be reduced by this amount when checking bearing resistance. This reduction is not required for the axial resistance check, however.

![Bearing Stiffener Geometry](image)

Compute the net area:

\[A_{pn} = (140 - 13)(17) = 2154 \text{ mm}^2\]

Compute the bearing resistance:

\[B_r = \varphi_b A_{pn} F_{ys}\]  
(S6.10.8.2.3-1)

\[= 1.0(2159)(345)(2 \text{ stiffeners})/1000 = 1490 \text{ kN} > 1316 \text{ kN (OK)}\]
Determine the axial resistance (S6.10.8.2.4):

Effective Length:

\[
kl = 0.75 \, D \\
= 0.75(1425) = 1069 \, \text{mm}
\]  

(S6.10.8.2.4a)

Determine effective section:

![Effective Section Diagram]

Figure 12.7.10.10.5-3 - Effective Section for a Single Pair of Bearing Stiffeners

\[
A_s ' = 2(17)(140) \% 10[18(10) \% 17]' = 6730 \, \text{mm}^2
\]

\[
l_s ' = \frac{17(2(140) \% 10)^3}{12} ' = 3.455E7 \, \text{mm}^4
\]

\[
r ' = \sqrt{\frac{l_s}{A_s}} = \sqrt{\frac{3.455E7}{6730}} = 71.7 \, \text{mm}
\]

Axial Resistance (Article S6.9.4.1):

Compute slenderness factor, \( \lambda \):

\[
\lambda ' = \left( \frac{kl}{r_s' \pi} \right)^2 \frac{F_y}{E} \left( \frac{1069}{(71.7)(3.14)} \right)^2 \frac{345}{200000} = 0.039
\]

Because \( \lambda \neq 2.25, \)

\[
P_n ' = 0.66\lambda F_y A_s
\]

\[
0.66^{0.039}(345)(6730)/1000
\]

\[
339(6730)/1000
\]

\[
2285 \, \text{kN}
\]

\[
\varphi_c P_n = 0.90(2285) = 2057
\]
2057 > 1316 (OK)

12.7.10.10.6 Specification Check: Wind Loads

Although the effects of wind loads on steel girder is only applicable to exterior girders, these effects will be investigated herein to illustrate the procedure.

Table 12.7.10.10.6-1 - Load Combinations and Load Factors for Wind

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>γ_{LL}</th>
<th>γ_{WS}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength III</td>
<td>0.0</td>
<td>1.40</td>
</tr>
<tr>
<td>Strength V</td>
<td>1.35</td>
<td>0.40</td>
</tr>
</tbody>
</table>

The load factors for permanent loads are the same as those used for the Strength I Load Combination.

Determine the Wind Load:

Assumptions:

Height of superstructure = 5500 mm
Basic wind speed, \( V_B = 160 \) km/hr (see 3.8.1.1)

Compute the exposed height of superstructure:

\[
H_{exp} = H_{parapet} + t_{lab} + t_{haunch} + d_{beam}
\]

\[
H_{exp} = 1070 + 210 + 0 + 1468 = 2748 \text{ mm}
\]

Compute the design wind load, \( V_{DZ} \) (3.8.1.1):

Because the superstructure height is less than 10 000 mm, the design wind speed is taken as the basic wind speed \( V_B \).

Determine the design wind pressure, \( P_B \):

\[
\begin{align*}
P_D & = P_B \left( \frac{V_{DZ}}{V_B} \right)^2 \frac{V_{DZ}^2}{25600} \\
\text{(S3.8.1.2-1)}
\end{align*}
\]

where,

\[
P_B = 0.0024 \text{ MPa} \quad \text{(Table S3.8.1.2-1)}
\]

So,

\[
P_B \frac{V_{DZ}^2}{25600} \cdot 0.0024 \left( \frac{160^2}{25600} \right) = 0.0024 \text{ MPa}
\]

Compute the load per lineal mm:

\[
w_{\text{wind}} = P_D H_{exp} = 0.0024(2748) = 6.60 \text{ N/mm}
\]
And the minimum for a girder component is 4.4 N/mm (OK).

Calculate the wind moment, $M_w$ (SC4.6.2.7.1):

Compute the pressure to be carried by the bottom flange:

$$W' = \frac{\gamma P_d d}{2}$$

( SC4.6.2.7.1-1)

Strength III:

$$W' = \frac{\gamma P_d d}{2} \cdot (1.4)(0.0024)(1468) \cdot 2.47 \text{ N/mm}$$

Strength V:

$$W' = \frac{\gamma P_d d}{2} \cdot (0.4)(0.0024)(1468) \cdot 0.70 \text{ N/mm}$$

Compute $M_w$ assuming the wind is transmitted by frame action of the cross-frames transmitting forces to the deck (second load path in Article S4.6.2.7.1).

$$M_w = \frac{wL_b^2}{10}$$

( S4.6.2.7.1-2)

Strength III:

$$M_w = \frac{wL_b^2}{10} \cdot \frac{2.47(7200)^2}{10} \cdot 12.8 \text{ kN@}$$

Strength V:

$$M_w = \frac{wL_b^2}{10} \cdot \frac{0.70(7200)^2}{10} \cdot 3.6 \text{ kN@}$$

Determine the wind effects at point of maximum moment

(X = 14.4 m; See Article S6.10.5.7.1):

Because the section was determined to be compact, Equation S6.10.3.5.1-1 applies:
The reduction of the flange width at each edge of the bottom flange is computed by the following:

\[
 b_{w}' = b_f \sqrt{\frac{b_f^2 \cdot \frac{4M_w}{t_f F_y b_f}}{2} + \frac{b_f}{2}}
\]  

(S6.10.3.5.1-1)

Strength III:

\[
 b_{w}' = b_f \sqrt{\frac{b_f^2 \cdot \frac{4M_w}{t_f F_y b_f}}{2} + \frac{b_f}{2}} \quad 300 \sqrt{\frac{300^2 \cdot \frac{4\left(12.8 \times 10^6\right)}{30 \times 345}}{2}} = 4.2 \text{ mm}
\]

Strength V:

\[
 b_{w}' = b_f \sqrt{\frac{b_f^2 \cdot \frac{4M_w}{t_f F_y b_f}}{2} + \frac{b_f}{2}} \quad 300 \sqrt{\frac{300^2 \cdot \frac{4\left(3.6 \times 10^6\right)}{30 \times 345}}{2}} = 1.16 \text{ mm}
\]

Compute the capacity of the compact section using method “A”:

\[
 M_{n}' = 1.3 R_h M_y \# M_p
\]  

(S6.10.4.2.2a-3)

Compute the factored loads for Strength III and Strength V:

**Table 12.7.10.10.6-2 - Factored Moment (values in kN@m; Strength III):**

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>( \eta \gamma )</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{DL1}</td>
<td>1582.7</td>
<td>(0.95)(1.25)</td>
<td>1879</td>
</tr>
<tr>
<td>M_{DL2P}</td>
<td>314.1</td>
<td>(0.95)(1.25)</td>
<td>373</td>
</tr>
<tr>
<td>M_{DL2W}</td>
<td>306.7</td>
<td>(0.95)(1.50)</td>
<td>437</td>
</tr>
<tr>
<td>M_{LL}</td>
<td>2770.5</td>
<td>(0.95)(0)</td>
<td>0</td>
</tr>
</tbody>
</table>

**Table 12.7.10.10.6-3 - Factored Moment (values in kN@m; Strength V):**

<table>
<thead>
<tr>
<th>Force Effect</th>
<th>Unfactored Values</th>
<th>( \eta \gamma )</th>
<th>Factored Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{DL1}</td>
<td>1582.7</td>
<td>(0.95)(1.25)</td>
<td>1879</td>
</tr>
<tr>
<td>M_{DL2P}</td>
<td>314.1</td>
<td>(0.95)(1.25)</td>
<td>373</td>
</tr>
<tr>
<td>M_{DL2W}</td>
<td>306.7</td>
<td>(0.95)(1.50)</td>
<td>437</td>
</tr>
<tr>
<td>M_{LL}</td>
<td>2770.5</td>
<td>(0.95)(1.35)</td>
<td>3553</td>
</tr>
</tbody>
</table>
Reduced bottom flange widths:

Strength III:

\[ b_f^* = b_r - 2(b_w) \]
\[ b_f^* = 300 - 2(4.4) = 291.2 \text{ mm} \]

Strength V:

\[ b_f^* = 300 - 2(1.16) = 297.7 \text{ mm} \]

Compute the new section properties and yield moment, \( M_y \), based on the reduced section:

Table 12.7.10.6-4 - Section Moduli for Wind Load Case (units of mm\(^4\)):

<table>
<thead>
<tr>
<th>Section Properties</th>
<th>DL1</th>
<th>DL2</th>
<th>LL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength III</td>
<td>Strength V</td>
<td>Strength III</td>
</tr>
<tr>
<td>( S_f )</td>
<td>1.078E7</td>
<td>1.082E7</td>
<td>4.488E7</td>
</tr>
<tr>
<td>( S_n )</td>
<td>1.483E7</td>
<td>1.483E7</td>
<td>1.991E7</td>
</tr>
</tbody>
</table>

Compute \( M_y \), \( M_p \), \( M_u \) for Strength III:

\[ M'_y = M_{DL1} \% M_{DL2} \% S_{LL} \left( F_y f \& \frac{M_{DL1}}{S_{DL1}} \& \frac{M_{DL2}}{S_{DL2}} \right) \]  
(See SA6.2)

\[ M_y' = 1879 \% 810 \% 3804 \' 6494 \text{ kN@} \]

\( M_p = 9853 \text{ kN@} \) (See SA6.1)

\( M_u = 2689 \text{ kN@} \)

Compute \( M_y \), \( M_p \), \( M_u \) for Strength V:

\[ M_y' = 1879 \% 810 \% 3906 \' 6595 \text{ kN@} \]

\( M_p = 9953 \text{ kN@} \)

\( M_u = 6242 \text{ kN@} \)

Compute the factored resistance, \( M_r \):

Strength III:

\[ M_r = 1.3 R_h M_y = 1.3(1.0)(6494) \]
\[ = 8442 \text{ kN@} # M_p \text{ (OK)} \]

\[ \varphi M_r = (1.0)(8442) \]
\[ = 8442 \text{ kN@} > 2689 \text{ kN@} \text{ (OK)} \]
Strength V:

\[ M_n = 1.3 R_n M_r = 1.3(1.0)(6595) \]
\[ = 8574 \text{ kN@} M_n \text{ (OK)} \]

\[ M_r = \phi M_n = (1.0)(8574) \]
\[ = 8574 \text{ kN@} > 6242 \text{ kN@} \text{ (OK)} \]

Therefore, this section is sufficient to resist the wind load cases of Strength III and Strength V.
REFERENCES


Guide Specifications for Fatigue Design of Steel Bridges (1989), American Association of State Highway and Transportation Officials, Washington, DC

Standard Specifications for Highway Bridges (1992), American Association of State Highway and Transportation Officials, Washington, DC
Quiz: Steel Girder Fatigue

Given:

- 40 m long, two-lane simple span plate girder bridge
- estimated ADTT over design life = 3000 trucks per day one way
- plate girder cross-section
  - top flange = 305 mm x 19 mm
  - web = 1829 mm x 13 mm
  - bottom flange = 356 mm x 35 mm
  - girder spacing = 3600 mm
  - slab thickness = 200 mm
  - modular ratio = 7

Find:

1. Check fatigue at the toe of a transverse stiffener-to-flange fillet weld at mid-span of an interior girder.

2. Check fatigue at the end of a long gusset plate (< 25 mm thick) groove-welded to flange with
no transition radius at mid-span of an interior girder.
Part 1

The stress due to permanent loads in the bottom flange at mid-span is tensile, therefore, the fatigue provisions of Article 6.6.1.2 must be applied.

Fatigue Load

Estimate the live load moment per lane due to the fatigue truck.

Influence line for mid-span moment of simple span beam

Influence line ordinate at front axle = 1/2 (20-4.3) = 7.85 m

Influence line ordinate at drive axle = 1/2 (20) = 10.0 m

Influence line at rear axle = 1/2 (20-9.0) = 5.50 m
Live load moment per lane:

\[ M_{LL} = 35 \text{kN (7.85)} + 145 \text{kN (10.0)} + 145 \text{kN (5.50)} \]

\[ = 2522 \text{kN-m} \]

Live load plus impact moment per lane:

\[ M_{LL+I} = (1+0.15) (2522 \text{kN-m}) \]

\[ = 2900 \text{kN-m} \]

Estimate the live load plus impact moment per girder using the distribution factor for moment to interior girders for single-lane loading.

\[ g = 0.06 \% \left( \frac{S}{4300} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{k_g}{L e_g^3} \right)^{0.1} \]

\[ S = \text{girder spacing} = 3600 \text{ mm} \]

\[ L = \text{span length} = 40000 \text{ mm} \]

\[ k_g = n (I + A e_g^2) \]

\[ n = \text{modular ratio} = 7 \]

\[ I = \text{girder moment of inertia} = \]

\[ Q2-4 \]
2.13 \times 10^{10} \text{ mm}^4

A = \text{girder area} = 4.22 \times 10^4 \text{ mm}^2

e_g = \text{eccentricity of girder}

\begin{align*}
e_g &= \bar{y}_{\text{top}} \frac{t_s}{2} \\
&= 1,083 \% \frac{200}{2}, 1,183 \text{ mm}
\end{align*}

k_g = 7 (2.13 \times 10^{10} + (4.22 \times 10^4) (1183)^2)

= 5.63 \times 10^{11}

g = 0.06 \% \left( \frac{3660}{4300} \right)^{0.4} \left( \frac{3660}{40,000} \right)^{0.3} \left( \frac{5.63 \times 10^{11}}{(40,000)(200)^3} \right)

= 0.06 + (0.851)^{0.4} (0.092)^{0.3} (1.759)^{0.1}

g = 0.06 + 0.938 (0.489) (1.058) = 0.545

For fatigue design, the multiple presence factor must be removed.

Q2-5
Live load plus impact moment per girder

\[ M_{LL+i} = 0.454 \times 2900 \text{ kN-m} \]

\[ = 1317 \text{ kN-m} \]

Live load plus impact stress in bottom flange

\[ \sigma = \frac{M_{LL+i} \cdot S_b}{\beta_b} \]

\[ = \frac{1317 \text{ kN}\cdot\text{m}}{3.80 \times 10^7 \text{ mm}^3} \]

\[ = 34,660 \frac{kN}{m^2} \]

\[ = 3.47 \times 10^7 \frac{N}{m^2} \text{ or Pa} \]

\[ = 34.7 \text{ MPa} \]

In a simple span, the maximum live load plus impact stress is the stress range.

\[ (\Delta f) = 34.7 \text{ MPa} \]

\[ \gamma (\Delta f) = 0.75 \times 34.7 \text{ MPa} = 26.0 \text{ MPa} \]
γ = load factor specified in Table 3.4.1-1 for fatigue

**Fatigue Resistance**

Toe of transverse stiffener-to-flange fillet weld is Category C' as per Table 6.6.1.2.3-1

Nominal fatigue resistance

\[(ΔF)^n \left(\frac{A}{N}\right)^{1/3} \frac{1}{2} (ΔF)_{TH}\]

First, always check the lower bound of nominal fatigue resistance.

\[(ΔF)^n \frac{1}{2} (ΔF)_{TH} \frac{1}{2} (82.7 \text{ MPa})' 41.4 \text{ MPa}\]

**Constant amplitude threshold from Table 6.6.1.2.5-3**

Since the lower bound of fatigue resistance is greater than the fatigue load, the LRFD equation for fatigue, Equation 6.6.1.2.2-1, is satisfied.

\[γ (Δf) #(ΔF)_n\]

Q2-7
26.0 MPa = γ (∆f) #(∆F)_n $ \leq 41.4$ MPa

Theoretically, the Category C' detail will experience infinite life.

**Part 2**

End of long gusset plate, less than 25 mm thick, groove-welded to flange with no transition radius is Category E as per Table 6.6.1.2.3-1.

$$(ΔF)_n \left( \frac{A}{N} \right)^{1/3} \leq \frac{1}{2} (ΔF)_TH$$

For Category E,

$$\frac{1}{2} (ΔF)_TH \leq \frac{1}{2} (31.0 \text{ MPa}) \leq 15.5 \text{ MPa}$$

(from Table 6.6.1.2.5-3)

Detail will not exhibit infinite life! Check finite life.

$N = (365) (75) \ n \ (ADTT)_{SL}$ Equation 6.6.1.2.5-2

$$(ADTT)_{SL} = P (ADTT)$$ Equation 3.6.1.4.2-1

$= 0.85 \ (3000) = 2550$

(from Table 3.6.1.4.2-1)

Q2-8
n = 1.0 (from Table 6.6.1.2.5-2)

\[ N = (365) (75) (1.0) (2550) \]
\[ = 70 \times 10^6 \text{ cycles} \]

\[ (\Delta F)_n' \left( \frac{A}{N} \right)^{1/3} \left( \frac{3.61 \times 10^{11}}{70 \times 10^6} \right) \]

(from Table 6.6.1.2.5-1)

= 17.3 MPa

26.0 MPa = γ (Δf) #(ΔF)_n = 17.3 MPa

'' The detail is not acceptable!

Try the same detail, but with a 150 mm transition radius with end welds ground smooth.

Now detail is Category C, just as the detail in Part 1, and, therefore, it is acceptable.
LECTURE 13 - BOLTED CONNECTIONS

13.1 GENERAL

Provisions regarding bolts and bolted connections have been updated and consolidated in Article S6.13.2.

In bolted slip-critical connections subject to shear, the load is transferred between the connected parts by friction up to a certain level of force which is dependent upon the total clamping force on the faying surfaces and the coefficient of friction of the faying surfaces. The connectors are not subject to shear, nor is the connected material subject to bearing stress. As loading is increased to a level in excess of the frictional resistance between the faying surfaces, slip occurs, but failure in the sense of rupture does not occur. As a result, slip-critical connections are able to resist even greater loads by shear and bearing against the connected material. The strength of the connection is not related to the slip load. These Specifications require that the slip resistance, and the shear and bending resistance be computed separately. Because the combined effect of frictional resistance with shear or bearing has not been systematically studied and is uncertain, any potential greater resistance due to combined effect is ignored.

Joints subject to stress reversal, heavy impact loads, severe vibration or where stress and strain due to joint slippage would be detrimental to the serviceability of the structure are to be designated as slip-critical. They include:

• joints subject to fatigue loading;

• joints in shear with bolts installed in oversized holes;

• joints in shear with bolts installed in short- and long-slotted holes where the force on the joint is in a direction other than perpendicular to the axis of the slot, except where the Engineer intends otherwise and so indicates in the contract documents,

• joints subject to significant load reversal;

• joints in which welds and bolts share in transmitting load at a common faying surface;

• joints in axial tension or combined axial tension and shear;

• joints in axial compression only, with standard or slotted holes in only one ply of the connection with the direction of the load perpendicular to the direction of the slot, except for connections designed according to the provisions specified in Article S6.13.6.1.3;
• joints in which, in the judgment of the Engineer, any slip would be critical to the performance of the joint or the structure and which are so designated in the contract documents.

Slip-critical connections are proportioned to prevent slip under Load Combination Service II, as specified in Table S3.4.1-1, and to provide bearing, shear and tensile resistance at the applicable strength limit state load combinations.

Bearing-type connections shall be permitted only for joints subjected to axial compression, or for joints on bracing members, and shall satisfy the factored resistance, \( R_r \), at the strength limit state.

In bolted bearing-type connections, the load is resisted by shear in the fastener and bearing upon the connected material, plus some uncertain amount of friction between the faying surfaces. The final failure will be by shear failure of the connectors, or by tear out of the connected material, or by unacceptable ovalization of the holes. Final failure load is independent of the clamping force provided by the bolts.

13.2 FACTORED RESISTANCE

13.2.1 General

For slip-critical connections, the factored resistance, \( R_r \), of a bolt at the Service II Load Combination is taken as:

\[
R_r = R_n \quad (13.2.1-1)
\]

where:

\( R_n \) = nominal resistance as specified in Article S6.13.2.8 and summarized in Article 13.2.3

Equation 13.2.1-1 applies to a service limit state for which the resistance factor is 1.0, and, hence, is not shown in the equation.

The factored resistance, \( R_r \) or \( T_r \), of a bolted connection at the strength limit state is taken as either:

\[
R_r = \varphi R_n \quad (13.2.1-2)
\]

\[
T_r = \varphi T_n \quad (13.2.1-3)
\]

where:

\( R_n \) = nominal resistance of the bolt, connection or connected material as follows:

• for bolts in shear, \( R_n \) is taken as specified in Article 13.2.3
for the connected material in bearing joints, \( R_n \) is taken as specified in Article 13.2.4

for connected material in tension or shear, \( R_n \) is taken as specified in Article S6.13.5

\[ T_n = \text{nominal resistance of a bolt as follows:} \]

- for bolts in axial tension, \( T_n \) is taken as summarized in Article 13.2.5
- for bolts in combined axial tension and shear, \( T_n \) is taken as summarized in Article 13.2.6

\[ \varphi = \text{resistance factor for bolts specified in Article S6.5.4.2} \]
and given in Article 12.2.3, taken as:

- \( \varphi_s \) for bolts in shear,
- \( \varphi_t \) for bolts in tension,
- \( \varphi_{bb} \) for bolts bearing on material,
- \( \varphi_v \), or \( \varphi_u \) for connected material in tension, as appropriate, or
- \( \varphi_v \) for connected material in shear.

13.2.2 Slip Resistance

The current AASHTO LFD Specifications and the LRFD Specifications for slip-critical connections are identical insofar as the surface conditions of the faying surfaces and hole size factors are concerned.

In the LFD Specifications, the bolt tension, \( T_b \) (MPa), is constant for all bolt sizes in a given faying surface condition and hole type, whereas in the LRFD Specifications, the bolt tension, \( P_t \) (Kn), is given for each bolt size and is independent of the surface condition and hole type.

For A325 bolts up to 25 mm, the LFD and LRFD Specifications provide approximately the same slip resistance, however, in the larger bolt sizes, the LRFD Specifications have about 10% less resistance than the LFD Specifications.

For A490 bolts in all bolt sizes, the LRFD Specifications have 10-15% greater slip resistance than the LFD Specifications.

The nominal slip resistance of a bolt in a slip-critical connection is taken as:

\[ R_n = K_h K_s N_s P_t \]  \hspace{1cm} (13.2.2-1)
where:

\[ N_s = \text{number of slip planes per bolt} \]

\[ P_t = \text{minimum required bolt tension specified in Table 13.2.2-1 (N)} \]

\[ K_h = \text{hole size factor specified in Table 13.2.2-2} \]

\[ K_s = \text{surface condition factor specified in Table 13.2.2-3} \]

Extensive data developed through research has been statistically analyzed to provide improved information on slip probability of connections in which the bolts have been preloaded to the requirements of Table 13.2.2-1. Two principal variables, coefficient of friction, i.e., the surface condition factor of the faying surfaces, and bolt pretension were found to have the greatest effect on the slip resistance of connections.

Hole size factors less than 1.0 are provided for bolts in oversize and slotted holes because of their effects on the induced tension in bolts using any of the specified installation methods. In the case of bolts in long slotted holes, even though the slip load is the same for bolts loaded transverse or parallel to the axis of the slot, the values for bolts loaded parallel to the axis have been further reduced, based upon judgment, because of the greater consequences of slip.

The criteria for slip resistance are for the case of connections subject to a coaxial load. For cases in which the load tends to rotate the connection in the plane of the faying surface, a modified formula accounting for the placement of bolts relative to the center of rotation should be used.
Table 13.2.2-1 - Minimum Required Bolt Tension

<table>
<thead>
<tr>
<th>Bolt Diameter, mm</th>
<th>Required Tension $P_t$ (x $10^3$ N)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M164 (A325M)</td>
</tr>
<tr>
<td>16</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td>M253 (A490M)</td>
</tr>
<tr>
<td>20</td>
<td>142</td>
</tr>
<tr>
<td>22</td>
<td>176</td>
</tr>
<tr>
<td>24</td>
<td>205</td>
</tr>
<tr>
<td>27</td>
<td>267</td>
</tr>
<tr>
<td>30</td>
<td>326</td>
</tr>
<tr>
<td>36</td>
<td>475</td>
</tr>
</tbody>
</table>

Table 13.2.2-2 - Values of $K_h$

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>for standard holes</td>
<td>1.0</td>
</tr>
<tr>
<td>for oversize and short-slotted holes</td>
<td>0.85</td>
</tr>
<tr>
<td>for long-slotted holes with the slot perpendicular to the direction of the force</td>
<td>0.70</td>
</tr>
<tr>
<td>for long-slotted holes with the slot parallel to the direction of the force</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Table 13.2.2-3 - Values of $K_s$

<table>
<thead>
<tr>
<th>Surface Conditions</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>for Class A surface conditions</td>
<td>0.33</td>
</tr>
<tr>
<td>for Class B surface conditions</td>
<td>0.50</td>
</tr>
<tr>
<td>for Class C surface conditions</td>
<td>0.33</td>
</tr>
</tbody>
</table>

The following descriptions of surface condition shall apply to Table 3:

- Class A Surface: unpainted clean mill scale, and blast cleaned surfaces with Class A coatings,
• Class B Surface: unpainted blast cleaned surfaces, and blast cleaned surfaces with Class B coatings, and

• Class C Surface: hot-dip galvanized roughened by wire brushing after galvanizing.

The effect of ordinary paint coatings on limited portions of the contact area within joints and the effect of overspray over the total contact area have been investigated experimentally. The tests demonstrated that the effective area for transfer of shear by friction between contact surfaces was concentrated in an annular ring around and close to the bolts. Paint on the contact surfaces approximately 25 mm, but not less than the bolt diameter, away from the edge of the hole did not reduce the slip resistance. On the other hand, bolt pretension might not be adequate to completely flatten and pull thick material into tight contact around every bolt. Therefore, these Specifications require that all areas between bolts also be free of paint.

On clean mill scale, this research found that even the smallest amount of overspray of ordinary paint, i.e., a coating not qualified as Class A, within the specified paint-free area, reduced the slip resistance significantly. On blast-cleaned surfaces, the presence of a small amount of overspray was not as detrimental. For simplicity, these Specifications prohibit any overspray from areas required to be free of paint in slip-critical joints, regardless of whether the surface is clean mill scale or blast cleaned.

The contract documents shall specify that joints having painted faying surfaces be blast cleaned and coated with a paint which has been qualified by test as a Class A or Class B coating.

The mean value of slip coefficients from many tests on clean mill scale, blast cleaned steel surfaces and galvanized and roughened surfaces were taken as the basis for the three classes of surfaces. As a result of research, a test method was developed and entitled "Test Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints", AISC (1988). The method includes long-term creep test requirements to ensure reliable performance for qualified paint coatings. The method, which requires requalification if an essential variable is changed, is the sole basis for qualification of any coating to be used under these Specifications. Further, normally only two categories of surface conditions for paints to be used in slip-critical joints are recognized: Class A for coatings which do not reduce the slip coefficient below that provided by clean mill scale, and Class B for paints which do not reduce the slip coefficient below that of blast cleaned steel surfaces.

Tests on surfaces that were wire brushed after galvanizing have indicated an average value of the slip coefficient equal to 0.35. Untreated surfaces with normal zinc have much smaller slip coefficients. Even though the slip coefficient for Class C surfaces is the same as for Class A surfaces, a separate class is retained to avoid
potential confusion. The higher value of the slip coefficient equal to 0.40 in previous specifications assumes that the surface has been blast cleaned after galvanizing, which is not the typical practice.

**13.2.3 Shear Resistance**

In the LFD Specifications, the values of shear strength ($\varphi R$) are 25% lower than the LRFD Specifications. The reductions in shear strength for the larger bolt diameters and for length between extreme fasteners are the same in either Specification.

The nominal shear resistance of a high-strength bolt or an ASTM A307 bolt at the strength limit state in joints whose length between extreme fasteners measured parallel to the line of action of the force is less than 1270 mm is taken as:

- where threads are excluded from the shear plane:
  \[ R_n = 0.48 A_b F_{ub} N_s \]  
  (13.2.3-1)

- where threads are included in the shear plane
  \[ R_n = 0.38 A_b F_{ub} N_s \]  
  (13.2.3-2)

where:

- $A_b$ = area of the bolt corresponding to the nominal diameter (mm$^2$)
- $F_{ub}$ = specified minimum tensile strength of the bolt specified in Article S6.4.3 (MPa)
- $N_s$ = number of shear planes per bolt

The nominal shear resistance of a bolt in connections greater than 1270 mm in length shall be taken as 0.80 times the value given by Equations 13.2.3-1 or 13.2.3-2.

In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, the thread length of the bolt shall be determined as two thread pitch lengths greater than the specified thread length.

If the threads of a bolt are included in the shear plane in the joint, the shear resistance of the bolt in all shear planes of the joint shall be the value for threads included in the shear plane.

The nominal resistance in shear is based upon the observation that the shear strength of a single high-strength bolt is about 0.6 times the tensile strength of that bolt. However, in shear connections with more than two bolts in the line of force, deformation of the connected material causes nonuniform bolt shear force distribution so that the strength of the connection in terms of the average bolt strength
decreases as the joint length increases. Rather than provide a function that reflects this decrease in average fastener strength with joint length, a single reduction factor of about 0.80 was applied to the 0.6 multiplier. Studies have shown that the allowable stress factor of safety against shear failure ranges from 3.3 for compact, i.e., short, joints to approximately 2.0 for joints with an overall length in excess of 1270 mm. It is of interest to note that the longest, and often the most important, joints had the lowest factor, indicating that a factor of safety of 2.0 has proven satisfactory in service.

The average value of the nominal resistance for bolts with threads in the shear plane has been determined by a series of tests to be 0.833 $F_{ub}$ with a standard deviation of 0.03. A value of 0.80 times the nominal resistance, when threads are excluded, was selected for the specification formula based upon the area corresponding to the nominal body area of the bolt.

The shear strength of bolts is not affected by pretension in the fasteners provided the connected material is in contact at the faying surfaces.

The factored resistance equals the nominal shear resistance multiplied by a resistance factor less than that used to determine the factored resistance of a component. This ensures that the maximum strength of the bridge is limited by the strength of the main members rather than by the connections.

### 13.2.4 Bearing Resistance

The effective bearing area of a bolt shall be taken as its diameter multiplied by the thickness of the connected material on which it bears. The effective thickness of connected material with countersunk holes shall be taken as the thickness of the connected material, minus one-half the depth of the countersink.

Bearing stress produced by a high-strength bolt pressing against the side of the hole in a connected part is important only as an index to behavior of the connected part. Thus, the same bearing resistance applies regardless of bolt shear strength or the presence or absence of threads in the bearing area. The critical value can be derived from the case of a single bolt at the end of a tension member.

It has been shown using finger-tight bolts, that a connected plate will not fail by tearing through the free edge of the material if the distance $L$, measured parallel to the line of applied force from a single bolt to the free edge of the member toward which the force is directed, is not less than the diameter of the bolt multiplied by the ratio of the bearing stress to the tensile strength of the connected part.

The criterion for nominal bearing strength is
\[ \frac{L}{d} \frac{r_n}{F_u} \]  

(13.2.4-1)

where:

\( r_n \) = nominal bearing pressure (MPa)

\( F_u \) = specified minimum tensile strength of the connected part (MPa)

In these Specifications, the nominal bearing resistance of an interior hole is based on the clear distance between the hole and the adjacent hole in the direction of the bearing force. The nominal bearing resistance of an end hole is based on the clear distance between the hole and the end of the member. The nominal bearing resistance of the connected member may be taken as the sum of the resistances of the individual bolts. The clear distance is used to simplify the computations for oversize and slotted holes.

For standard holes, oversize holes, short-slotted holes loaded in any direction, and long-slotted holes parallel to the applied bearing force, the nominal bearing resistance of interior and end bolt holes at the strength limit state, \( R_n \), are taken as:

- with bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

  \[ R_n = 2.4dtF_u \]  
  (13.2.4-2)

- if either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

  \[ R_n = 1.2LctF_u \]  
  (13.2.4-3)

For long-slotted holes perpendicular to the applied bearing force:

- with bolts spaced at a clear distance between holes not less than 2.0d and with a clear end distance not less than 2.0d:

  \[ R_n = 2.0dtF_u \]  
  (13.2.4-4)

- if either the clear distance between holes is less than 2.0d, or the clear end distance is less than 2.0d:

  \[ R_n = LctF_u \]  
  (13.2.4-5)

where:

\( d \) = nominal diameter of the bolt (mm)

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\[
t = \text{thickness of the connected material (mm)}
\]

\[
F_{u} = \text{tensile strength of the connected material specified in Table S6.4.1-1 (MPa)}
\]

\[
L_c = \text{clear distance between holes or between the hole and the end of the member (mm)}
\]

The spacing limits for center-to-center of bolts and edge distance in the LFD Specifications are 3d and 1-1/2d respectively, whereas these limits are 2d for either situation in the LRFD Specifications.

The LRFD Specifications also recognize oversize holes, whereas the LFD Specifications do not.

The bearing resistance in the LRFD Specifications are 33% higher than the LFD Specifications.

**13.2.5 Tensile Resistance**

The nominal tensile resistance of a bolt, \(T_n\), independent of any initial tightening force shall be taken as:

\[
T_n = 0.76 A_b F_{ub} \quad (13.2.5-1)
\]

where:

\[
A_b = \text{area of bolt corresponding to the nominal diameter (mm}^2\)
\]

\[
F_{ub} = \text{specified minimum tensile strength of the bolt specified in Article S6.4.3 (MPa)}
\]

The recommended design strength is approximately equal to the initial tightening force; thus, when loaded to the service load, high-strength bolts will experience little, if any, actual change in stress. For this reason, bolts in connections, in which the applied loads subject the bolts to axial tension, are required to be fully tensioned.

The lower value specified for A307 bolts in tension reflects the greater variability of the test data, since tensile loading of mild steel bolts may often result in the thread stripping before development of the tensile strength.

Where high-strength bolts in axial tension are subject to fatigue, the stress range, \(\Delta \sigma\), in the bolt, due to the fatigue design live load, plus the dynamic load allowance for fatigue loading specified in Article S3.6.1.4, plus the prying force resulting from cyclic application of the fatigue load, shall satisfy Equation S6.6.1.2.2-1.
The nominal diameter of the bolt shall be used in calculating the bolt stress range. In no case shall the calculated prying force exceed 60% of the externally applied load.

Properly tightened A325M and A490M bolts are not adversely affected by repeated application of the recommended service load tensile stress, provided that the fitting material is sufficiently stiff such that the prying force is a relatively small part of the applied tension. The provisions covering bolt tensile fatigue are based upon study of test reports of bolts that were subjected to repeated tensile load to failure.

Low carbon ASTM A307 bolts shall not be used in connections subjected to fatigue.

The tensile force due to prying action shall be taken as:

\[
Q_u = \left[ \frac{3b}{8a} \times \frac{t^3}{328000} \right] P_u
\]

where:

- \(Q_u\) = prying tension per bolt due to the factored loadings taken as 0 when negative (N)
- \(P_u\) = direct tension per bolt due to the factored loadings (N)
- \(a\) = distance from center of bolt to edge of plate (mm)
- \(b\) = distance from center of bolt to the toe of fillet of connected part (mm)
- \(t\) = thickness of thinnest connected part (mm)

13.2.6 Resistance to Combined Shear and Tension

The nominal tensile resistance of bolts subject to combined axial tension and shear is provided by elliptical interaction curves, which account for the connection length effect on bolts loaded in shear, the ratio of shear strength to tension strength of threaded bolts, and the ratios of root area to nominal body area and tensile stress area to nominal body area.

The nominal tensile resistance of a bolt subjected to combined shear and axial tension, \(T_n\), shall be taken as:

If \(\frac{P_u}{R_n} \leq 0.33\), then

\[
T_n = 0.76A_b F_{ub}
\]
Otherwise:

\[ T_n' = 0.76 A_b F_{ub} \left( 1 - \frac{P}{\phi_s R_n} \right)^2 \]  

(13.2.6-1)

where:

- \( A_b \) = area of the bolt corresponding to the nominal diameter (mm²)
- \( F_{ub} \) = specified minimum tensile strength of the bolt specified in Article S6.4.3 (MPa)
- \( P_u \) = shear force on the bolt due to the factored loads (N)
- \( R_n \) = nominal shear resistance of a bolt specified in Article 13.2.3 (N)

Equation 13.2.6-1 is a conservative simplification of the set of elliptical curves, and represents the case for A325M bolts where threads are not excluded from the shear plane. Curves for other cases may be found in AISC (1988). No reduction in the nominal shear resistance is required when applied tensile stress is equal to or less than the nominal tensile resistance.

The nominal resistance of a bolt in slip-critical connections under Load Combination Service II, specified in Table S3.4.1-1, subjected to combined shear and axial tension, shall not exceed the nominal slip resistance specified in Article 13.2.2 multiplied by:

\[ 1 & \frac{T_u}{P_t} \]  

(13.2.6-2)

where:

- \( T_u \) = tensile force due to the factored loads under Load Combination Service II (N)
- \( P_t \) = minimum required bolt tension specified in Table 13.2.2-1 (N)

**13.3 BOLTED SPLICE DESIGN EXAMPLE**

The design of a bolted splice is presented in this example. The section dimensions of the composite plate girder and the section forces at the splice location were obtained from a preliminary design of a continuous girder bridge with two spans 55 000 mm each. The splice was located about 15 000 mm from the intermediate support.
The splice plates were designed to satisfy the requirements of the strength and fatigue limit states, and the bolts were designed to satisfy the strength and service limit states.

Assumptions

- All bolts are high-strength, slip-critical in standard holes.
- For all limit states, assume combined factor for ductility, redundancy and importance, \( \eta = 1.0 \) (S1.3.2.1).
- Only the forces produced by the notional design loads were considered in this example. For a complete design, the forces produced by the permit vehicles need to be checked using the same procedures.
- The longitudinal steel in the slab was ignored in the analysis of both the positive and negative moment sections. However, the designer may chose to consider such reinforcement as a part of the section resisting negative moments (S6.10.3.1.1c).
- When the combined effect of the moments and shears is applied to a component, the maximum moment and the maximum shear at the section are applied simultaneously. The designer may chose to check the component under the effect of the maximum moment and the concurrent shear or the maximum shear and the concurrent moment.
• When calculating the bolt edge distance in the girded web and flanges, a 6 mm gap is assumed to exist between the two segments of the spliced girder.

**Steel Plate Dimensions**

The design is controlled by the smaller of the two girder sections connected by the splice (S6.13.6.1.1). All given plate dimensions, cross-section properties and load resistance are calculated for this section.

\[
\begin{align*}
D_w &= 1950 \text{ mm}, \text{ depth of girder web} \\
T_w &= 20 \text{ mm}, \text{ thickness of girder web} \\
W_{tf} &= 420 \text{ mm}, \text{ width of girder top flange} \\
T_{tf} &= 32 \text{ mm}, \text{ thickness of girder top flange} \\
W_{bf} &= 420 \text{ mm}, \text{ width of girder bottom flange} \\
T_{bf} &= 32 \text{ mm}, \text{ thickness of girder bottom flange}
\end{align*}
\]

**Composite Concrete Slab**

For the purpose of this example, no deck slab haunch was considered in the analysis. See Figure 13.3-1 for the dimensions of the effective flange width of the composite girder.

\[
\begin{align*}
f'_c &= 28 \text{ MPa}, 28\text{-day compressive strength of slab concrete} \\
n &= 8, \text{ modular ratio (S6.10.3.1.1b)}
\end{align*}
\]

**Unfactored Loads at the Strength and Service Limit States**

\[
\begin{align*}
M_{DL1} &= -6.274E08 \text{ N@m, moment due to noncomposite dead load} \\
V_{DL1} &= -4.496E05 \text{ N, shear due to noncomposite dead load} \\
M_{DL2a} &= -2.29E07 \text{ N@m, moment due to weight of parapet} \\
V_{DL2a} &= -5.29E04 \text{ N, shear due to weight of parapet} \\
M_{DL2b} &= -2.90E07 \text{ N@m, moment due to weight of future wearing surface} \\
V_{DL2b} &= -6.69E04 \text{ N, shear due to weight of future wearing surface} \\
M_{LL,pos} &= 2.916E09 \text{ N@m, maximum positive live load moment} \\
M_{LL,neg} &= -2.677E09 \text{ N@m, minimum negative live load moment}
\end{align*}
\]
\[ V_{\text{LL,pos}} = 6.45 \times 10^4 \text{ N}, \text{ maximum positive live load shear} \]
\[ V_{\text{LL,neg}} = -4.912 \times 10^5 \text{ N}, \text{ minimum negative live load shear} \]

**Unfactored Moments for the Fatigue Limit States**

\[ M_{\text{pos}} = 1.233 \times 10^9 \text{ N}\cdot\text{m} \]
\[ M_{\text{neg}} = -8.867 \times 10^8 \text{ N}\cdot\text{m} \]

**Steel Plate Materials**

M270 web and web splice plates:

\[ F_y = 250 \text{ MPa} \]
\[ F_u = 400 \text{ MPa} \]

M270 flange and flange splice plates:

\[ F_y = 345 \text{ MPa} \]
\[ F_u = 450 \text{ MPa} \]

**Allowable Fatigue Stress Range (All Steel Plate Components)**

Category B Fatigue

\[ F_{r,fat} \leq 110/2 = 55.0 \text{ MPa} \] (S6.6.1.2.5)

**Bolts**

For the purpose of this example, all bolts are assumed to be AASHTO M164 bolts with no threads included in the shear planes (S6.13.2.7). It is worth noting that having several, e.g., 3-5, threads in the grip (not necessarily in the shear plane) improves the ductility of the connection.

\[ D_{\text{bolt}} = 24 \text{ mm} \] (S6.13.2.4.2-1)
\[ D_{\text{hole}} = 27 \text{ mm} \] (S6.13.2.4.2-1) [**Notice that the hole diameter was changed to 26 mm instead of the 27 mm used in this example, in the 1996 Interim**]
\[ F_{u,bolt} = 830 \text{ MPa} \] (S6.4.3)

The hole size used in this example conforms to the provisions of the 1994 AASHTO-LRFD Specifications. However, due to the continuous revisions of the SI conversions, future refinements to the hole sizes in the SI edition of the Specifications are possible. The user of this example is advised to refer to any applicable addendums to the Specifications.
See Figure 13.3-2 for splice plate dimensions and for bolt pattern, pitch, gage spacing and end and edge distances. These distances satisfy the spacing limits of Article S6.13.2.6.

![Diagram of splice plates and bolt patterns](image)

**Figure 13.3-2 - Dimensions of Splice Plates and Configuration of Bolt Patterns**

### Section Properties

The gross section properties are required to be used in determining the stresses at the fatigue limit state and at Service Limit State (see Article S6.13.6.1.4a and commentary). To facilitate the
In checking the stresses in the flange splice plates at the strength limit state, both the yield on the gross section and the fracture on the net section are required to be investigated. In determining the stresses in the web splice plates, gross and net section properties of the web splice plates were used to determine the stresses at the fatigue and strength limit states, respectively.

The great majority of splices are located near the point of dead load coutraflexure points. In this case, live loads produce most of the moments at the splice location. Therefore, it is customary to use the following approximate procedure to calculate the applied stresses and bolt forces under the effect of live loads or the combined effect of live loads plus dead loads:

- In case of positive total factored moments, the short-term composite section properties are used.
- In case of negative total factored moments, the noncomposite section properties are used.
- The fatigue check requires the calculation of the stresses under the effect of permanent loads. In this case, the noncomposite section properties were considered.

In general, following this procedure reduces the amount of calculations and does not cause a significant error. However, in some unique situations, the designer is forced to locate field splices at a significant distance from the dead load coutraflexure points. An example of these situations is the case of long-span bridges where the location of the field splices may be controlled by the maximum available plate length and the maximum transportable segment length. In such situations, a more accurate approach to calculate the stresses in the splice plates and forces on the bolts may be required. One such approach is to apply each of the moment components to the appropriate section properties, i.e., DL1, DL2 and live load moments are applied to the noncomposite section, long-term composite section and short-term composite section, respectively.

Since the splice in this example is located near the point of dead load coutraflexure point, it was appropriate to use the approximate procedure.

Noncomposite section properties:

\[ I_{nc} = 3.876 \times 10^{10} \text{ mm}^4 \]

Distance to Section N.A., from bottom of bottom flange:
\[ Y_{nc} = 1007.0 \text{ mm} \]

Composite short-term section properties:

\[ I_{c-n} = 7.983 \times 10^{10} \text{ mm}^4 \]

Distance to Section N.A., from bottom of bottom flange:

\[ Y_{c-n} = 1563.0 \text{ mm} \]
Figure 13.3-3 - Location of Neutral Axis for Composite and Noncomposite Sections
Section Resistance

Using the provisions of Articles S6.10.5 and S6.10.6, the resistance of the smaller of the two girder sections at the splice location was determined as:

Positive Flexure:

\[ M_{r,pos} = 2.043 \times 10^{10} \text{ N@m (Compact, Composite, Plastic)} \]

Negative Flexure:

\[ M_{r,neg} = -1.394 \times 10^{10} \text{ N@m (Compact, Plastic)} \]

For shear:

\[ V_r = 3.733 \times 10^{6} \text{ N (Unstiffened Web)} \]

Maximum Factored Loads (S3.4)

Apply the maximum load factor to the components of the load effect that has the same sign as the force effect being maximized. For the load effect components that have the opposite sign, apply the minimum load factor to the effect of noncomposite dead loads and the weight of the parapet and ignore the effects of the future wearing surface and the live loads.

Factored Loads, Strength I Limit State

Maximum factored positive moments:

\[ M_{u,pos} = 0.9M_{DL1} + 0.9M_{DL2a} + 1.75M_{LL,pos} \]

\[ = -(0.90)(6.274 \times 10^8) - (0.90)(2.29 \times 10^7) + (1.75)(2.916 \times 10^9) \]

\[ = 4.518 \times 10^9 \text{ N@m} \]

Minimum factored negative moments:

\[ M_{u,neg} = 1.25M_{DL1} + 1.25M_{DL2a} + 1.5M_{DL2b} + 1.75M_{LL,neg} \]

\[ = -(1.25)(6.274 \times 10^8) - (1.25)(-2.29 \times 10^7) - (1.50)(2.90 \times 10^7) - (1.75)(2.677 \times 10^9) \]

\[ = -5.541 \times 10^9 \text{ N@m} \]

Maximum factored positive shear:

\[ V_{u,pos} = 0.9V_{DL1} + 0.9V_{DL2a} + 1.75V_{LL,pos} \]

\[ = -(0.90)(4.496 \times 10^5) - (0.90)(5.29 \times 10^4) + (1.75)(6.45 \times 10^4) \]
Minimum factored negative shear:

\[ V_{u,neg} = 1.25V_{DL1} + 1.25V_{DL2a} + 1.5V_{DL2b} + 1.75V_{LL,neg} \]

\[ = -(1.25)(4.496E05) - (1.25)(5.29E04) - (1.50)(6.69E04) - (1.75)(4.912E05) \]

\[ = -1.588E06 \text{ N} \]

The minimum negative shear has a larger absolute value and, hence, controls the design.

**Maximum Factored Loads, Service II Limit State**

Maximum factored positive moments:

\[ M_{u,pos} = M_{DL1} + M_{DL2a} + 1.3M_{LL,pos} \]

\[ = -6.274E08 - 2.29E07 + (1.3)(2.916E09) \]

\[ = 3.141E09 \text{ N@m} \]

Minimum factored negative moments:

\[ M_{u,neg} = M_{DL1} + M_{DL2a} + M_{DL2b} + 1.3M_{LL,neg} \]

\[ = -6.274E08 - 2.29E07 - 2.90E07 - (1.3)(2.677E09) \]

\[ = -4.159E09 \text{ N@m} \]

Maximum factored positive shear:

\[ V_{u,pos} = V_{DL1} + V_{DL2a} + 1.3V_{LL,pos} \]

\[ = -4.496E05 - 5.29E04 + (1.3)(6.45E04) \]

\[ = -58.64 \text{ N} \]

Minimum factored negative shear:

\[ V_{u,neg} = V_{DL1} + V_{DL2a} + V_{DL2b} + 1.3V_{LL,neg} \]

\[ = -4.496E05 - 5.29E04 - 6.69E04 - (1.3)(4.912E05) \]

\[ = -1.21E06 \text{ N} \]

The minimum negative shear has a larger absolute value and, hence, controls the design.
Determination of Splice Design Loads (S6.13)

Positive Flexure:

Average of section moment resistance and the applied factored moment: (S6.13.1 of the first edition)

\[ M_{\text{ave}} = \frac{(M_u, \text{pos} + M_r, \text{pos})}{2} \]
\[ = \frac{(4.518 \times 10^9 + 2.043 \times 10^{10})}{2} = 1.247 \times 10^{10} \text{ N@m} \]

(1)

75% of section moment resistance: (S6.13.1)

\[ M_{r,75\%} = 0.75M_{r,\text{pos}} = 0.75 \times 2.043 \times 10^{10} = 1.532 \times 10^{10} \text{ N@m} \]

(2)

Splice design positive moment = Larger of (1) and (2) = 1.532E10 N@m

Negative Flexure:

Average of section moment resistance and the applied factored moment (S6.13.1, first edition):

\[ M_{\text{ave}} = \frac{(M_u, \text{neg} + M_r, \text{neg})}{2} \]
\[ = \frac{(-5.541 \times 10^9 - 1.394 \times 10^{10})}{2} = -9.74 \times 10^9 \text{ N@m} \]

(3)

75% of section moment resistance: (S6.13.1)

\[ M_{r,75\%} = 0.75M_{r,\text{neg}} = -0.75 \times 1.394 \times 10^{10} = -1.046 \times 10^{10} \text{ N@m} \]

(4)

Splice design negative moment = Larger of (3) and (4) = 1.046E10 N@m

Shear:

Average of applied factored shear and shear resistance of the section:

\[ V_{\text{ave}} = \frac{|V_u, \text{neg}| + V_r}{2} \]
\[ = \frac{|-1.588 \times 10^6| + 3.733 \times 10^6}{2} = 2.660 \times 10^6 \text{ N} \]

(5)

75% of the shear resistance of the section: (S6.13.1, first edition)

\[ V_{r,75\%} = 0.75V_r = 0.75 \times 3.733 \times 10^6 = 2.800 \times 10^6 \text{ N} \]

(6)
Notional Shear = the applied factored shear multiplied by the ratio of the design moment divided by the applied factored moment

Case of positive moment: Notional shear= \(|-1.588\times 10^6| \times 1.532\times 10^{10} / 4.518\times 10^9 = 5.385\times 10^6 \) N \hspace{1cm} (7)

Case of negative moment: Notional shear= \(|-1.588\times 10^6| \times 1.046\times 10^{10} / 5.541\times 10^9 = 2.998\times 10^6 \) N \hspace{1cm} (8)

For the design of web splice plates and their bolts, the design shear = Largest of (5) through (8) = 5.385E06 N

Notice that the value of the notional shear exceeds the calculated shear resistance of the web plate. The designer may choose to use an upper bound to the notional shear value equal to the calculated shear resistance of the web plate, i.e., 3.733E06 N. However, the post-buckling shear resistance of the web plate is actually higher than that calculated above. In addition, for this example, the web shear resistance was determined assuming unstiffened web. The presence of the splice plates will provide some stiffening effect to the web and the actual shear resistance may exceed the calculated value. Using the notional shear for design allows the bridge owner to upgrade the bridge load capacity without disturbing the field splice. Therefore, the notional shear was used in the design without any adjustment to account for the smaller web capacity.

In addition to the above design loads, the web splice plates and their bolts are designed to resist an additional moment due to the eccentricity of the notional shear relative to the center of the bolt group.

Notional shear = 5.385E06 N

Eccentricity = distance between the center of the bolt group at one side of the splice to the center of the splice = 90.5 mm

Moment of eccentricity of notional shear = 5.385E06 x 90.5 = 4.875E08 N@m

The entire moment due to the eccentricity of the notional shear is resisted by the web splice plates and their bolts.

**Design of Web Splice Plates**

The portion of the design moment resisted by the web plate at the splice location cannot be determined accurately. The most common method to determine the portion of the design moment resisted by the web plate is to multiply the splice design moment by the ratio of the moment of inertia of the web to the moment of inertia.
of the girder section. The following are some of the sources of the inaccuracy of this assumption:

- The ratio of the moment of inertia of the web to the moment of inertia of the section at a certain location differs if the properties of the net or gross section were used. In addition, the ratio changes based on if the section was in positive or negative flexure due to the change from composite to noncomposite section. Even if the sign of the moment does not change, more than one value of this ratio may be obtained for the same section. For sections in positive flexure, the ratio of the moment of inertia of the web to the moment of inertia of the section differs for the noncomposite section, long-term composite section and short-term composite section. For sections in negative flexure, if the longitudinal reinforcement of the slab were considered active, the ratio would differ before and after the slab hardens. In addition, a separate set of ratios may be obtained for each of the two sections of the girder connected at the splice location.

- The design moment for the splice may exceed the moment that initiates yield in the webs of hybrid sections. In this case, the portion of the moment resisted by the web is not proportionate to the ratio of the moment of inertia of the web to the moment of inertia of the section.

For the purpose of this example, in case of positive total factored moments, the properties of the gross short-term composite section were used to determine the portion of the total moment resisted by the web. Similarly, in case of negative total factored moments, the portion of the total moment resisted by the web was calculated using the properties of the gross noncomposite steel section.

[Notice that new provisions were introduced in the 1999 Interim as Article S6.10.3.6. These provisions are to be used in calculating section properties of flexural members with holes.]

**Web Plate Section Properties**

Gross area of the web plate:

\[ A_w = D_wT_w = (1950)(20) = 3.90E04 \text{ mm}^2 \]

Gross moment of inertia of the web plate about its horizontal centroidal axis:

\[ I_w = T_w(D_w)^3/12 = (20)(1950)^3/12 = 1.236E10 \text{ mm}^4 \]

Gross moment of inertia of the web plate, about the neutral axis of the gross noncomposite section (see Figure 13.3.3-a):

\[ I_{w,nc} = I_w + A_w x (0.0)^2 = 1.236E10 \text{ mm}^4 \]
Ratio of web stiffness to section stiffness, \( R_{nc} = \frac{1.236 \times 10^10}{3.876 \times 10^10} = 0.319 \)

Gross moment of inertia of the web plate, about the neutral axis of the gross short-term composite section (see Figure 13.3.3-b):

\[
I_{w,c-n} = I_w + A_w \times (556)^2 \\
= 1.236 \times 10^10 + (3.90 \times 10^4)(556)^2 \\
= 2.442 \times 10^10 \text{ mm}^4
\]

Ratio of web stiffness to section stiffness, \( R_c = \frac{2.442 \times 10^10}{7.983 \times 10^10} = 0.306 \)

**Design of Web Splice Plates**

Gross section properties:

The gross section properties of the web splice plates are required to be used in the calculations of fatigue stresses (S6.13.6.1.4a).

Gross area of both web splice plates:

\( A_{wspl} = 2(1825)(18) = 6.57 \times 10^4 \text{ mm}^2 \)

Moment of inertia of both web splice plates about their horizontal centroidal axis:

\( I_{wspl} = 2(18)(1825)^3/12 = 1.824 \times 10^10 \text{ mm}^4 \)

Moment of inertia of both web splice plates about the neutral axis of different sections:

For noncomposite section (see Figure 13.3.3-a):

\[
I_{nc} = I_{wspl} + A_{wspl} \times (0.0)^2 \\
= 1.824 \times 10^10 \text{ mm}^4
\]

\[
S_{t,nc} = \frac{1.824 \times 10^10}{912.5} = 1.998 \times 10^7 \text{ mm}^3
\]

\[
S_{b,nc} = \frac{1.824 \times 10^10}{912.5} = 1.998 \times 10^7 \text{ mm}^3
\]

For short-term composite section (see Figure 13.3.3-b):

\[
I_{c-n} = I_{wspl} + A_{wspl} \times (556)^2 \\
= 1.824 \times 10^10 + 6.57 \times 10^4 \times (556)^2 \\
= 3.855 \times 10^10 \text{ mm}^4
\]
\[
S_{c,n} = \frac{3.855 \times 10^{10}}{356.5} = 1.081 \times 10^8 \text{ mm}^3
\]
\[
S_{b,c-n} = \frac{3.855 \times 10^{10}}{1468.5} = 2.625 \times 10^7 \text{ mm}^3
\]

**Net Section Properties**

The net section properties of the web splice plates are required to be used in the calculations of stresses at the strength and service limit states.

Diameter of standard holes for net section calculations = 27.2 mm (S6.8.3)  
*Notice that this value was changed from bolt diameter +3.2 mm = 24.0 + 3.2 = 27.2 mm to hole diameter + 2.0 mm = 26.0 + 2.0 = 28.0 mm in 1998*

Area of the holes = 2 x 24 x 27.2 x 18 = 23 500 mm²

Considering that the web splice plates are primarily flexural members, only the area of holes in excess of 15% of the gross area needs to be deducted (S6.10.1.4, first edition).  
*Notice that different rules to calculate the effective section were introduced as Article S6.10.3.6 in the 1999 Interim*

Actual percentage of section loss = 23 500 / 6.57E04 = 35.8%

Percentage loss to be considered in net section calculations = 35.8 - 15 = 20.8%

Ratio between net section to gross section = 1.00 - 0.208 = 0.792

Considering that the holes are uniformly distributed along the height of the web plate, it is reasonable to assume that reducing both the area and the moment of inertia of the gross section by 20.8% will produce the net section properties.

Net area of both web splice plates:

\[
A_{wsp} = 6.57E04 \times 0.792 = 5.203E04 \text{ mm}^2
\]

Moment of inertia of both web splice plates about their horizontal centroidal axis:

\[
I_{wsp} = 1.824E10 \times 0.792 = 1.445E10 \text{ mm}^4
\]

Moment of inertia of both web splice plates about the neutral axis of different sections:

For noncomposite section (see Figure 13.3.3-a):

\[
I_{nc} = 1.824E10 \times 0.792 = 1.445E10 \text{ mm}^4
\]
\[
S_{t,nc} = 1.998E07 \times 0.792 = 1.582E07 \text{ mm}^3
\]
$$S_{b,nc} = 1.998 \times 10^7 \times 0.792 = 1.582 \times 10^7 \text{ mm}^3$$

For short-term composite section (see Figure 13.3.3-b):

$$I_{c,n} = 3.855 \times 10^9 \times 0.792 = 3.053 \times 10^9 \text{ mm}^4$$

$$S_{t,c,n} = 1.081 \times 10^8 \times 0.792 = 8.562 \times 10^7 \text{ mm}^3$$

$$S_{b,c,n} = 2.625 \times 10^7 \times 0.792 = 2.079 \times 10^7 \text{ mm}^3$$

### Check of Stresses on the Net Section of the Web Splice Plates (S6.13.6.1.4b)

Allowable Flexural Stress at Strength I Limit State = $F_y = 250$ MPa (S6.13.6.1.4b)

Maximum applied stress in web splice plates = (ratio of web stiffness to section stiffness x $M + M_e$) / $S$

where:

- $M$ = total factored moment at the splice location
- $M_e$ = moment due to the eccentricity of the notional shear
- $S$ = the appropriate section modulus corresponding to the sign of the moment

**Positive Flexure:**

$$\sigma_t = \frac{-0.306 \times 1.532 \times 10^9 + 4.875 \times 10^8}{8.562 \times 10^7} = -60.4 \text{ MPa} \ < \ 250 \text{ MPa} \ \text{OK}$$

$$\sigma_b = \frac{0.306 \times 1.532 \times 10^9 + 4.875 \times 10^8}{2.079 \times 10^7} = 248.9 \text{ MPa} \ < \ 250 \text{ MPa} \ \text{OK}$$

**Negative Flexure:**

$$\sigma_t = \frac{0.319 \times 1.046 \times 10^9 + 4.875 \times 10^8}{1.582 \times 10^7} = 241.7 \text{ MPa} \ < \ 250 \text{ MPa} \ \text{OK}$$

$$\sigma_b = \frac{-0.319 \times 1.046 \times 10^9 + 4.875 \times 10^8}{1.582 \times 10^7} = -241.7 \text{ MPa} \ < \ 250 \text{ MPa} \ \text{OK}$$

### Shear (S6.13.5.3)

Nominal shear capacity of two web splice plates:

$$V_n = 0.58A_g F_y = (0.58)(6.57 \times 10^4)(250) = 9.527 \times 10^6 \text{ N}$$

Shear resistance, $V_r = \phi_r V_n = (1.00)(9.527 \times 10^6)$
Design shear = 5.385E06 N < \( V_r = 9.527E06 \) N \quad \text{OK}

**Check Fatigue in Web Splice Plates**

Gross section properties are required to be used in the stress calculations for the Fatigue Limit State (S6.13.6.1.4a). Fatigue needs to be checked only if the compressive stress due to permanent loads is less than twice the tensile stress due to the fatigue load combination (S6.6.1.2.1). Assuming new construction, the effect of the future wearing surface will be considered only if it reduces the compressive stresses due to permanent dead loads.

Unfactored stresses in splice plates due to permanent loads:

\[
\sigma_t = -0.319 \times (-6.274E08 - 2.29E07 - 2.90E07) / 1.998E07 \\
= 10.85\text{MPa} > 0.0
\]

Therefore, fatigue needs to be checked at the top of the web splice plates.

\[
\sigma_b = 0.319 \times (-6.274E08 - 2.29E07) / 1.998E07 \\
= -10.38 \text{MPa} < 0.0
\]

\( \sigma_b \) is compression, therefore, check maximum stresses due to fatigue load combination to determine if fatigue needs to be checked at bottom of web splice plates.

Stress in splice plates due to fatigue load combination:

Positive flexure acting on the short-term composite section:

\[
\sigma_t = -0.306 \times (0.75 \times 1.233E09) / 1.081E08 \\
= -2.61 \text{MPa}
\]

\[
\sigma_b = 0.306 \times (0.75 \times 1.233E09) / 2.625E07 \\
= 10.78 \text{MPa}
\]

Negative flexure acting on the noncomposite section:

\[
\sigma_t = 0.319 \times (0.75 \times 8.867E08) / 1.998E07 \\
= 10.62 \text{MPa}
\]

\[
\sigma_b = -0.319 \times (0.75 \times 8.867E08) / 1.998E07 \\
= -10.62 \text{MPa}
\]
Permanent load compressive stress at the bottom < twice the maximum tensile stress from the fatigue load combination.

Therefore, fatigue at the bottom of the web splice plates needs to be checked.

Allowable Fatigue Stress Range:

Bolted connections are considered Fatigue Category B (Table S6.6.1.2.3-1). For this category, the allowable fatigue stress range, \( F_{r,fat} \), is at least 55 MPa (S6.6.1.2.5).

Fatigue stress range, top of web splice plate
\[
= \left| (-2.61) - (10.62) \right|
\]
\[
= 13.23 \text{ MPa} < 55.0 \text{ MPa} \quad \text{OK}
\]

[Notice that fatigue of bolted splice plates is usually less critical than fatigue of the connected members. Therefore, fatigue of splice plate is rarely checked.]

Fatigue stress range, bottom of web splice plate
\[
= \left| (10.78) - (-10.62) \right|
\]
\[
= 21.4 \text{ MPa} < 55.0 \text{ MPa} \quad \text{OK}
\]

**Design of Web Splice Bolts**

Assuming:

\[ Y_i = \text{ the Y coordinate of a bolt} \]
\[ X_i = \text{ the X coordinate of a bolt} \]

Both \( X \) and \( Y \) are measured relative to the centroidal axes of the bolt group at one side of the splice.

Moment of inertia of the web splice bolts at one side of the splice about the horizontal axis at the center of gravity of the bolt group = \( G Y_i^2 = 1.294E07 \text{ mm}^2 \).

Moment of inertia of the web splice bolts at one side of the splice about the vertical axis at the center of gravity of the bolt group = \( G X_i^2 = 6.75E04 \text{ mm}^2 \).

All coordinates are measured relative to the center of gravity of the bolt group.

Moment of inertia of the bolt group about the neutral axis of the noncomposite section:

\[ I_{x,nc} = 1.294E07 \text{ mm}^2 \]
\[ \begin{align*}
I_{y,nc} &= 6.75 \times 10^4 \text{ mm}^2 \\
I_{p,nc} &= 1.294 \times 10^7 + 6.75 \times 10^4 = 1.301 \times 10^7 \text{ mm}^2 \\
\text{Moment of inertia of the bolt group about the neutral axis of the short-term composite section:} \\
I_{x,c-n} &= I_{x,nc} + \text{Number of bolts } \times (\text{the Y distance between the center of the bolt group to the center of the section})^2 \\
&= 1.294 \times 10^7 + 48 \times (556)^2 = 2.778 \times 10^7 \text{ mm}^2 \\
I_{y,c-n} &= 6.75 \times 10^4 \text{ mm}^2 \\
I_{p,c-n} &= 2.778 \times 10^7 + 6.75 \times 10^4 = 2.785 \times 10^7 \text{ mm}^2 \\
\end{align*} \]

**Maximum Factored Shearing Force Applied to the Most Stressed Bolt**

**Effect of Moments:**

The same loads that were used to design the splice plates are used to design the bolts.

Vertical component of bolt shear due to the portion of moment resisted by the web plus the moment of eccentricity of the notional shear = \( \frac{M \times X}{I_p} \).

Horizontal component of bolt shear due to the portion of the moment resisted by the web plus the moment of eccentricity of the notional shear = \( \frac{M \times Y}{I_p} \).

where:

\( M \) = total applied factored moment

\( I_p \) = polar moment of inertia of the appropriate section with respect to the applied moment

\( X \) and \( Y \) = coordinates of the bolt with respect to the neutral axis of the section

See Figure 13.3-4 for location of the bolts relative to the neutral axis of the section. See Figure 13.3.4 for the location of the most stressed bolts in case of positive and negative flexure.
Figure 13.3-4 - Location of the Most Stressed Bolts

Case of Positive Flexure:

Vertical force on Bolt A = 
\[
(0.306 \times 1.532 \times 10^8 + 4.875 \times 10^8) \times \frac{37.5}{2.785 \times 10^7} = 6970 \text{ N}
\]

Horizontal force on Bolt A = 
\[
(0.306 \times 1.532 \times 10^8 + 4.875 \times 10^8) \times \frac{306.5}{2.785 \times 10^7} = 56960 \text{ N}
\]

Vertical force on Bolt B = 6970 N

Horizontal force on Bolt B = 
\[
(0.306 \times 1.532 \times 10^8 + 4.875 \times 10^8) \times \frac{1418.5}{2.785 \times 10^7} = 263600 \text{ N}
\]

Case of Negative Flexure:

Vertical force on Bolt C = 
\[
(0.319 \times 1.046 \times 10^8 + 4.875 \times 10^8) \times \frac{37.5}{1.301 \times 10^7} = 11020 \text{ N}
\]

Horizontal force on Bolt C = 
\[
(0.319 \times 1.046 \times 10^8 + 4.875 \times 10^8) \times \frac{862.5}{1.301 \times 10^7} = 263600 \text{ N}
\]
(1.301E07) = 253 530 N

Vertical force on Bolt D = 11 020 N

Horizontal force on Bolt D = 253 530 N

Effect of Notional Shear:

Bolt vertical shear due to notional shear = notional shear / number of bolts = 5.385E06/48 = 112 190 N/bolt

Total Shear Force on Bolts

By inspection, the force on bolt B controls the design
Total shear force in bolt B, \( P_u \) = \[ (\text{total horizontal shear})^2 + (\text{total vertical shear})^2 \]^{0.5} = \[ (263 600)^2 + (6970 + 112 190)^2 \]^{0.5} = 289 300 N/bolt

Shear Resistance of Bolts (S6.13.2.7)

Number of slip planes per bolt, \( N_s = 2 \)

Assume:

\[ D_{\text{bolt}} = 24 \text{ mm} \]

\[ A_{\text{bolt}} = \pi (24)^2/4 = 452.4 \text{ mm}^2 \]

Hole diameter for net area calculations = 24 + 3.2 = 27.2 mm (S6.8.3) [Notice that hole diameter has changed in 1998 to hole diameter + 2 mm = 28.0 mm.]

Assume threads are excluded from shear plane.

Bolt Shear Resistance at the Strength Limit State (S6.13.2.7)

\[ P_n = 0.48 N_s A_{\text{bolt}} F_{u,bolt} \]

\[ = 0.48(2)(452.4)(830) = 360 472 \text{ N} \]

\[ P_r = \phi_s P_n = (0.80)(360 472) = 288 380 \text{ N} \]

\[ P_u = 289 300 \text{ N} \mu P_r = 288 380 \text{ N} \quad \text{OK} \]

Bolt Bearing Resistance at the Strength Limit State (S6.13.2.9)

By inspection, thickness of the web is less than the sum of thicknesses of the two splice plates. Therefore, bearing on the web controls.

Clear distance between holes = 75.0 - 27.0 = 48.0 mm [Notice that hole diameter was changed to 26.0 mm in 1996.]
Clear distance between holes and edge of splice plate = 
50.0 - (27.0/2) = 36.5 mm ---controls

\[ L_{c,min} = 36.5 \text{ mm} < 2.0D_{bolt} = 2.0(24) = 48.0 \text{ mm} \]

\[ P_n = 1.2L_{c,min}T_wF_{u,web} \]
\[ = 1.2(36.5)(20)(400) = 350 400 \text{ N} \]

\[ P_r = \phi_{bb}P_n = (0.80)(350 400) = 280 300 \text{ N} \]

\[ P_u = 289 300 \text{ N} > P_r = 280 300 \text{ N} \]

The applied load 3.2% larger than the bearing resistance of the bolt. In situations where the bolts are slightly overstressed, most designers exercise some judgment to determine if the over stress value is within the practical limits considering the ratio between the design loads and the applied loads and considering the approximations in the design procedures. For the purpose of this example, the number of the bolts was not increased.

**Bolt Slip Resistance at the Service II Limit State (S6.13.2.8)**

**Applied Forces at the Service II Limit State:**

Bolt vertical shear due to applied shear force 
= shear force / number of bolts

\[ = (1.21E06)/(48) = 25 210 \text{ N/bolt} \]

Moment of eccentricity of applied shear = 1.21E06 x 90.5
\[ = 1.095E08 \text{ N} \]

**Case of Positive Flexure:**

By inspection, the force on Bolt B is higher than that on Bolt A.

Vertical Force in Bolt B = \((0.306 \times 3.141E09 + 1.095E08)(37.5)/(2.785E07) = 1440 \text{ N}\)

Horizontal Force in Bolt B = \((0.306 \times 3.141E09 + 1.095E08)(1418.5)/(2.785E07) = 54 530 \text{ N}\)

**Case of Negative Flexure:**

Vertical Force in Bolt C = \((0.319 \times 4.159E09 + 1.095E08)(37.5)/(1.301E07) = 4140 \text{ N}\)

Horizontal Force in Bolt C = \((0.319 \times 4.159E09 + 1.095E08)(862.5)/\)
Vertical Force in Bolt D = 4140 N
Horizontal Force in Bolt D = 95 210 N

By inspection, the force in Bolts C and D are equal and control the design.

Total shear force in bolt C or D, \( P_u = \left[ \text{total horizontal shear} \right]^2 + \left( \text{total vertical shear} \right)^2 \right]^{0.5} = \left[ (95 210)^2 + (4140 + 25 210)^2 \right]^{0.5} = 99 630 \text{ N/bolt} \)

Bolt slip resistance, \( P_r = K_h K_s N_s P_t \)

where:

Number of shear planes, \( N_s = 2 \)

Minimum required bolt tension, \( P_t = 205 000 \text{ N} \) (Table S6.13.2.8-1)

\( K_h = 1.00 \) for standard hole (Table S6.13.2.8-2)

\( K_s = 0.33 \) for surface class A (Table S6.13.2.8-3)

\( P_r = (1.00)(0.33)(2)(205 000) = 135 300 \text{ N} \) (S6.13.2.8)

\( P_r > \text{Applied force} = 99 630 \text{ N} \) OK

**Design of Flange Splice Plates**

Gross and Net Areas:

Due to the equal size of the top and bottom flanges and the use of the same number of gage lines in both of them, the gross and net areas are the same for both flanges.

Flange gross area = 420 x 32 = 13 440 mm²

Section Modulii to Middle of Top and Bottom Flanges:

For noncomposite section (see Figure 13.3.3-a):

\( S_{t,nc} = \frac{3.876E10}{991.0} = 3.911E07 \text{ mm}^3 \)

\( S_{b,nc} = \frac{3.876E10}{991.0} = 3.911E07 \text{ mm}^3 \)

For short-term composite section (see Figure 13.3.3-b):

\( S_{t,c-n} = \frac{7.983E10}{435.0} = 1.835E08 \text{ mm}^3 \)

\( S_{b,c-n} = \frac{7.983E10}{1547.0} = 5.160E07 \text{ mm}^3 \)
**Design Force for Flange Splice Plates**

Assume Flange Force = Flange gross area x stress at center of flange based on gross section properties

[See S6.13.6.1.4b for new provisions for flange splice plates.]

Positive Flexure:

\[
P_{t,\text{pos}} = -13\,440 \times 1.532\times10^9 / 1.835\times10^8 = -1.122\times10^6 \text{ N}
\]

\[
P_{b,\text{pos}} = 13\,440 \times 1.532\times10^9 / 5.160\times10^7 = 3.99\times10^6 \text{ N}
\]

Negative Flexure:

\[
P_{t,\text{neg}} = 13\,440 \times 1.046\times10^9 / 3.911\times10^7 = 3.595\times10^6 \text{ N}
\]

\[
P_{b,\text{neg}} = -13\,440 \times 1.046\times10^9 / 3.911\times10^7 = -3.595\times10^6 \text{ N}
\]

**Allowable Stresses**

Compression: (S6.9)

\[
F_n = F_y = 345.0 \text{ MPa}
\]

\[
F_r = \phi_c F_n = (0.90)(345.0) = 310.5 \text{ MPa}
\]

Tension, yield of gross section: (S6.8.2.1)

\[
F_{ny} = F_y = 345.0 \text{ MPa}
\]

\[
F_{ry} = \phi_y F_{ny} = (0.95)(345.0) = 327.8 \text{ MPa}
\]

Tension, fracture of net section: (S6.8.2.1)

\[
F_{nu} = F_u = 450.0 \text{ MPa}
\]

\[
F_{ru} = \phi_u F_{nu} = (0.80)(450.0) = 360.0 \text{ MPa}
\]

**Design of Bottom Flange Splice Plates**

Case of Positive Flexure:

\[P_{b,\text{pos}} = 3.99\times10^6 \text{ N} > 0 \quad \text{Use tension allowables}\]

Total required flange splice plates gross area for yield in tension (S6.8.2.1-1):

\[
A_g = \frac{P_{b,\text{pos}}}{F_{ry}} = \frac{3.99\times10^6}{327.8} = 12\,172 \text{ mm}^2
\]
Total required flange splice plates net area for fracture in tension (S6.8.2.1-1):

\[ A_{\text{net}} = \frac{P_{\text{b, pos}}}{F_{\text{ru}}} \]

\[ = \frac{3.99E06}{360} = 11083 \text{ mm}^2 \]

Case of Negative Flexure:

\[ P_{\text{b, neg}} = -3.595E06 \text{ N} < 0 \quad \text{Use compression allowable} \]

Total required flange splice plates gross area for yield in compression:

\[ A_g = \frac{P_{\text{b, pos}}}{F_r} = \frac{-3.595E06}{310.5} = 11578 \text{ mm}^2 \]

Splice plates dimensions are chosen such that the areas of the plates above and below the flange are as close as practicable. This arrangement causes the center of the forces in the splice plates to be almost concentric with the center of the flange and, hence, minimizes the moment due to force eccentricity.

Assume:

420 x 20 mm wide outer splice plate, gross area = 8400 mm²

175 x 22 mm wide inner plates, gross area of two plates = 7700 mm²

Four gage lines:

Total provided gross area = 8400 + 7700 = 16 100 mm² > 12 172 mm² required OK

Net area of splice plates (S6.8.3):

Net section area used in the design is the smaller of the actual net section area or 85% of the gross section area (S6.13.5.2).

Diameter of standard holes for the purpose of calculating net area = diameter of bolt + 3.2 = 27.2 mm (S6.8.3) [Notice this changed in 1998 to hole diameter + 2.0 = 28 mm]

Assuming four gage lines, as shown in Figure 13.3.2:

The width lost from the outer splice plate due to the presence of the holes = (4)(27.2) = 108.8 mm²

Net width of outer flange splice plate = 420 - 108.8 = 311.2 mm

Ratio of net to gross area = 311.2 / 420 = 0.74 < 0.85 OK (6.13.5.2)

Net width of each of the inner flange splice plates = 175 - 2 x 27.2 = 120.6 mm
Ratio of net to gross area = 120.6 / 175 = 0.69 < 0.85 OK (6.13.5.2)

Provided net area = 311.2 x 20 + 2 x 120.6 x 22 = 11 530 mm² > 11 083 mm² required OK

Design of Top Flange Splice Plates

By inspection, top flange has less force than the bottom flange and, hence, requires smaller splice plates area. For the purpose of this example, top flange splice is kept the same as the bottom flange splice.

Check Fatigue in Flange Splice Plates

[Notice that fatigue of splice plates is typically less critical than the connected members. Therefore, fatigue of splice plates is rarely checked.]

Top Flange Splice Plates

Maximum stress under permanent loads = (6.274E08 + 2.29E07 + 2.90E07)/(3.911E07) = 17.36 MPa > 0  Fatigue needs to be checked (S6.6.1.2.1)

Maximum stress from fatigue load combination = 0.75 x 8.867E08 / 3.911E07 = 17.0 MPa

Minimum stress from fatigue load combination = -0.75 x 1.233E09 / 1.835E08 = -5.0 MPa

Applied fatigue stress range = 17.0 -(-5.0) = 22.0 MPa

Allowed stress range for bolted connections, $F_{r,fat}$, is at least 55.0 MPa (S6.6.1.2)

Allowable range > applied range  OK

Bottom Flange Splice Plates

All components of permanent load produce compression in the bottom flange. The most critical case is the case of minimum permanent load compressive stress. Therefore, the effect of the future wearing surface is not included.

Compressive stress due to permanent loads = (-6.274E08 - 2.29E07) / 3.911E07 = -16.6 MPa

Maximum stress from fatigue load combination = 0.75 x 1.233E09 / 5.160E07 = 17.9 MPa

$|\text{Permanent load compressive strength}| < \text{twice the tensile stress due to fatigue load combination}$, therefore, fatigue needs to be investigated (6.6.1.2).
Minimum stress from fatigue load combination = \(-0.75 \times 8.867E08 / 3.911E07 = -17.0\) MPa

Applied fatigue stress range = 17.9 -(-17.0) = 34.9 MPa

Allowable fatigue stress range for bolted splice = 55 MPa

Allowable range > Applied range = 34.9 MPa  OK

**Bottom Flange Splice Bolts**

For 24 mm diameter bolts in a standard hole assuming two shear planes and assuming that threads are excluded from the shear planes, single bolt resistance was calculated earlier as:

Bolt shear resistance at the Strength Limit State (S6.13.2.7):

\[
\begin{align*}
Pr &= \varphi_s P_n = 288 380 \text{ N} \quad (9) \\

\text{Bolt Bearing Resistance at the Strength Limit State (S6.13.2.9):}
\end{align*}
\]

By inspection, thickness of the flange is less than the sum of thicknesses of the two splice plates and the bolt edge distances in the flange is equal to those in the splice plates. Therefore, bearing on the flange controls.

Clear distance between holes = 75.0 - 27.0 = 48.0 mm [**Notice hole diameter changed to 26.0 mm in 1996**]

Distance from the center of last hole in the flange to the center of the splice = 50 mm

Clear distance from the holes to the edge of the flange = 50.0 - (27.0/2) = 36.5 mm controls

\[
L_{c,\min} = 36.5 \text{ mm} < 2.0 D_{\text{bolt}} = 2.0(24) = 48.0 \text{ mm}
\]

\[
\begin{align*}
P_n &= 1.2L_{c,\min}T_{\text{flange}}F_{u,\text{flange}} \\
&= 1.2(36.5)(32)(450) = 630 700 \text{ N} \\
P_r &= \varphi_{bb} P_n = (0.80)(630 700) = 504 600 \text{ N} \quad (10)
\end{align*}
\]

From (9) and (10), at the strength limit state, shear resistance of the bolt controls.

Required number of bolts

= Maximum force in flange / bolt resistance

= 3.99E06/288 380 = 13.84 bolts

Assume 14 bolts for both the top and bottom flanges
Bolt Slip Resistance at the Service II Limit State (6.13.22 and S6.13.2.8)

As calculated earlier, bolt resistance \( P_r = \varphi P_u = 135300 \text{ N} \) (S6.13.2.8)

Assume Flange Force = Flange gross area x stress at center of flange based on gross section properties

Positive Flexure:

\[
\begin{align*}
P_{t,\text{pos}} &= -13440 \times 3.141\times10^9 / 1.835\times10^8 = -2.301\times10^5 \text{ N} \\
P_{b,\text{pos}} &= 13440 \times 3.141\times10^9 / 5.160\times10^7 = 8.181\times10^5 \text{ N}
\end{align*}
\]

Negative Flexure:

\[
\begin{align*}
P_{t,\text{neg}} &= 13440 \times 4.159\times10^9 / 3.911\times10^7 = 1.429\times10^6 \text{ N} \\
P_{b,\text{neg}} &= -13440 \times 4.159\times10^9 / 3.911\times10^7 = -1.429\times10^6 \text{ N}
\end{align*}
\]

By inspection, the maximum forces are equal in both the top and bottom flanges.

Maximum force per bolt = \(1.429\times10^6 / 14 = 102071 \text{ N} \)

102071 N < 135300 allowed OK

Block Shear Rupture in Bottom Flange (S6.13.4)

[Notice that block shear failure does not control splices. It is checked here only to provide the user of this example with a reference on the application of these new provisions that may control the design of truss connections.]

Various possible modes of block shear rupture need to be checked. For the purpose of this example, only the three failure modes shown in Figure 13.3.5 were considered for the bottom flange. Since the flange dimensions, splice plates dimensions and bolt pattern are the same for both the top and bottom flange and the top flange forces are less than the bottom flange, there is no need to check the top flange for block shear failure.

For Failure Mode 1 (see Figure 13.3.5a)

The length of the planes of failure in the flange plate and in the splice plates is nearly equal. By inspection, Failure Mode 1 will occur in the flange before the splice plates due to its smaller thickness.

\[
\begin{align*}
A_{vg} &= 2 \times (200 + 275) \times 32 = 30400 \text{ mm}^2 \\
A_{vn} &= 30400 - 2 \times (2.5+3.5) \times 27.2 \times 32 = 19955 \text{ mm}^2
\end{align*}
\]
\[ \frac{s^2}{4g} = \frac{75^2}{4 \times 75} = 18.75 \]

\[ A_{\text{tg}} = 2 \times (75 + 18.75) \times 32 = 6000 \text{ mm}^2 \]

\[ A_{\text{tn}} = 600 - 27.2 \times 32 = 4259 \text{ mm}^2 \]

\[ A_{\text{tn}} / A_{\text{vn}} = 4259 / 19955 = 0.21 < 0.58, \text{ therefore:} \]

\[ R_r = \varphi_{bs} \left( 0.58 F_u A_{\text{vn}} + F_y A_{\text{tg}} \right) \]

\[ = 0.8 \times \left[ (0.58 \times 450 \times 19955) + (345 \times 6000) \right] = 5.822 \times 10^6 \text{ N} \]

Maximum applied factored flange force = 3.99\times 10^6 \text{ N} < R_r \text{ OK}

**For Failure Mode 2 (see Figure 13.3.5b)**

Similar to Failure Mode 1, Failure Mode 2 will occur in the flange before the splice plates.

\[ A_{\text{tg}} = 2 \times 275 \times 32 = 17600 \text{ mm}^2 \]

\[ A_{\text{tn}} = 17600 - 2 \times 3.5 \times 27.2 \times 32 = 11507 \text{ mm}^2 \]

\[ A_{\text{tn}} = 2 \times 125 \times 32 = 8000 \text{ mm}^2 \]

\[ A_{\text{tn}} = 8000 - 2 \times 0.5 \times 27.2 \times 32 = 7130 \text{ mm}^2 \]

\[ A_{\text{tn}} / A_{\text{vn}} = 7130 / 11507 = 0.62 > 0.58 \text{ therefore:} \]

\[ R_r = \varphi_{bs} \left( 0.58 F_y A_{\text{tg}} + F_u A_{\text{tn}} \right) \]

\[ = 0.8 \times \left[ (0.58 \times 345 \times 17600) + (450 \times 7130) \right] = 5.384 \times 10^6 \text{ N} \]

Maximum applied factored flange force = 3.99\times 10^6 \text{ N} < R_r \text{ OK}

**For Failure Mode 3 (see Figure 13.3.5c)**

\[ A_{\text{tg}} = 2 \times 220 \times (20 + 22) = 18480 \text{ mm}^2 \]

\[ A_{\text{tn}} = 18480 - 2 \times 2.5 \times 27.2 \times (20 + 22) = 12768 \text{ mm}^2 \]

\[ A_{\text{tg}} = 320 \times 20 + 2 \times 125 \times 22 = 11900 \text{ mm}^2 \]

\[ A_{\text{tn}} = 11900 - 3 \times 27.2 \times 20 - 2 \times 1.5 \times 27.2 \times 22 = 8473 \text{ mm}^2 \]

\[ A_{\text{tn}} / A_{\text{vn}} = 8473 / 12768 = 0.66 > 0.58 \]

therefore:
\[ R_r = \varphi_{bs} (0.58 F_y A_{vg} + F_u A_{in}) \]
\[ = 0.8 \times [(0.58 \times 345 \times 18480) + (450 \times 8473)] = 6.009 \times 10^6 \text{ N} \]

Maximum applied factored flange force = 3.99 \times 10^6 \text{ N} < R_r \text{ OK}

Therefore, block shear failure is unlikely.

Figure 13.3-5 - Three Possible Modes of Block Shear Rupture

(a) Failure Mode 1 in the Flange

(b) Failure Mode 2 in the Flange

(c) Failure Mode 3

Figure 13.3-5 - Three Possible Modes of Block Shear Rupture