OHIO DEPARTMENT OF TRANSPORTATION
CENTRAL OFFICE, 1980 W. BROAD ST., COLUMBUS, OHIO 43216-0899

July 18, 2008

To: Users of the Bridge Design Manual

From: Tim Keller, Administrator, Office of Structural Engineering

By: Sean Meddles, Bridge Standards Engineer

Re: 2008 Third Quarter Revisions

Revisions have been made to the ODOT Bridge Design Manual, January 2004. This package contains the revised pages. The revised pages have been designed to replace the corresponding pages in the book and are numbered accordingly. Revisions, additions, and deletions are marked in the revised pages by the use of one vertical line in the right margin. The header of the revised pages is dated accordingly.

To keep your Manual correct and up-to-date, please replace the appropriate pages in the book with the pages in this package.

To ensure proper printing, make sure your printer is set to print in the 2-sided mode.

The January 2004 edition of the Bridge Design Manual may be downloaded at no cost using the following link: http://www.dot.state.oh.us/se/BDM/BDM2004/bdm2004.htm

Attached is a brief description of each revision.

Summary of Revisions to the July 2004 ODOT BDM

BDM Section	Affected Pages	Revision Description
202.2.3.2.a	2-13	This change reflects the new terminology for rock classification (e.g. soft rock = weak rock, hard rock = strong rock) in the ODOT Specifications for Geotechnical Exploration, Jan. 2007
208	2-35.1 through 2-36.2	This change clarifies the design requirements for temporary support of existing structures.
302.2.3	3-14 through 3-15.2	This section has been expanded to include new plan elevation requirements. Screed Elevations have been redefined and are no longer required above all beam/girder lines. This was done to avoid confusion with the new Top of Haunch Elevation. The introduction of the Top of Haunch Elevation above each beam/girder simplifies the contractor's deck falsework calculations. The availability of Final Deck Elevations simplifies the troubleshooting process for deck placement issues should these arise.
302.2.7	3-16 through 3-17.14	This new section addresses the deck placement issues that were presented in our office's seminar, "The State of Practice for Highly Skewed Bridges".
302.4.1.14 & Fig. 334	3-26	With the retirement of BS-1-93, a common plan error occurring for bolted splice designs is the interference of splice plates and rolled beam fillets. This figure was created to illustrate the practical limits for the splice plates in order to avoid beam fillets.
302.4.1.14.a	3-27	This change addresses the proper bolt finishes required for partially painted weathering steel beams and girders. If both faying surfaces for bolts are not coated, then the contact between the dissimilar metal coatings (i.e. galvanizing vs. weathering steel) will create a galvanic cell causing early deterioration of the coating.
303.4.2.5	3-72	This change reflects a revision to pile restrike requirements in the C&MS 523.02 which specifies a restrike item to consist of performing dynamic testing on two piles with a CAPWAP analysis on one of the two piles tested. Previous editions of the C&MS did not clearly define these restrike requirements.
303.4.2.6	3-73	Refer to the BDM Section 303.4.2.5 revision description.
303.4.2.7	3-73 through 3-75	Refer to the BDM Section 303.4.2.5 revision description.
606.1	6-16.1 through 6-16.2	Changes to notes [29] allow the contractor to perform dynamic testing to determine refusal in lieu of the traditional 20 bpi criteria avoiding excessive wear and tear on the driving equipment.

BDM Section	Affected Pages	Revision Description
606.2	6-18 through 6-19	Refer to the BDM Section 303.4.2.5 revision description.
611.10	6-34 through 6-35	This new section addresses the deck placement issues that were presented in our office’s seminar, “The State of Practice for Highly Skewed Bridges”.
702.16	7-12 through 7-12.2	This section has been revised to comply with the revisions to BDM Section 302.2.3
Fig. 703 & 703M		These figures have been revised to comply with the revisions to BDM Section 302.2.3
Fig. 704 & 704M		These figures have been revised to comply with the revisions to BDM Section 302.2.3
901	9-1 through 9-1.2	This section has been revised to define a “Long-span bridge” for rating purposes.
911	9-17 through 9-20	This section has been added in order to capture live loading events consisting of more than one legal or permit load on a long span bridge at any one time.

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H Pile Size	Design Load	Ultimate Bearing Value
HP10X42	75 tons	150 tons
HP12X53	95 tons	190 tons
HP14X73	130 tons	260 tons

H Pile Size	Design Load	Ultimate Bearing Value
HP250X62	650 kN	1300 kN
HP310X79	850 kN	1700 kN
HP360X108	1150 kN	2300 kN

Ultimate Bearing load is equal to the actual unfactored design load multiplied by a safety factor of two (2). Design load values for H piles are based on a maximum service load stress of 12.5 ksi [86 MPa] for Grade 50 steel.

The actual value listed in the structure general notes should not be the Ultimate Bearing Value of the H pile size selected but the calculated Ultimate Bearing Value load of the substructure unit or units.

For the piers, other than capped pile piers, HP10X42 [HP250X62] should be used if the calculated design load is less than 75 tons [650 kN] per pile.

In order to protect the tips of the steel “H” piling, steel pile points shall be used when the piles are driven to refusal onto strong bedrock. When the depth of overburden is more than 50 feet [15 meters] and the soils are cohesive in nature, piles driven to strong bedrock generally should not have steel points. Steel points should not be used when the piles are driven to bear on shale.

For projects where steel points are to be used, include the plan note entitled “Item 507, Steel Points, As Per Plan” with the Structure General Notes (Section 600 of this Manual).

For capped pile piers with steel H piles, pile encasement is required.

202.2.3.2.b CAST-IN-PLACE REINFORCED CONCRETE PILES

For piles not driven to bear on the bedrock, cast-in-place reinforced concrete piles should be used. This type of pile achieves its design load resistance through a combination of side friction and end bearing. The commonly used pile sizes are:

Pipe Pile Diameter	Design Load	Ultimate Bearing Value
12 inch	50 tons	100 tons
14 inch	70 tons	140 tons
16 inch	90 tons	180 tons

Pipe Pile Diameter	Design Load	Ultimate Bearing Value
300 mm	450 kN	900 kN
350 mm	650 kN	1300 kN
400 mm	800 kN	1600 kN

Ultimate Bearing load is equal to the actual unfactored design load multiplied by a safety factor of two (2). The design values for pipe piles are based on a maximum allowable service load stress on the pile wall thickness of roughly 10 ksi [69 MPa] for ASTM A 252 Grade 2 steel, Fy = 35 ksi [Fy = 240 MPa].

The actual value listed in the structure general notes should not be the Ultimate Bearing Value of the Pipe pile size selected but the calculated Ultimate Bearing Value load of the substructure unit or units.

For capped-pile piers with cast-in-place piles, 16 inch [400 mm] diameter piles shall be used. 16 inch [400 mm] diameter piles with additional reinforcing steel are preferred because the need for pile encasement is eliminated. Additional reinforcing steel which consists of 8 - #6 [19M] epoxy coated reinforcing bars with #4 [13M] spiral at 12 inch [300 mm] pitch should be provided for 16 inch [400 mm] diameter piles. Reinforcing steel shall be detailed on the plans, included in the reinforcing steel list, and be paid for under Item 507, 16 Inch [400 mm] Cast-In-Place Piles Furnished, As Per Plan. The reinforcing steel cage should extend 15 feet [5 meters] below the flow line and into the pier cap. Pile encasement is not used when additional reinforcement is provided. Painting of the cast-in-place reinforced concrete pile is not required.

For capped-pile piers where the exposed length of the piles is more than 20 feet [6 meters], 18 inch [450 mm] diameter piles can be used. Consult the Office of Structural Engineering before recommending the use of 18 inch [450 mm] diameter piles.

202.2.3.2.c DOWN DRAG FORCES ON PILES

When a significant height of new embankment is constructed over a compressible layer of soil and long term settlement is anticipated, the possibility of down drag forces on the piles should be considered. The extra load that the pile receives due to the down drag force should be computed and accounted for by driving the piles to a higher design load capacity. For example, the total design load for the piles should be equal to Dead Load + Live Load + Down Drag Force. See Section 600 of this Manual for note.

For the purposes of determining vertical clearances, "Reconstructed" shall refer to an improvement of an existing structure involving the replacement of the entire superstructure.

207.2 BRIDGE SUPERSTRUCTURE

Bridge superstructure widths shall be established in accordance with ODOT's Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.3 LATERAL CLEARANCE

Divided highways having four or more lanes crossing under an intersecting highway shall be provided with a minimum lateral clearance of 30 feet [9000 mm] from the edge of traveled lane to the point where the 2:1 back slope intersects the radius at the toe of the 2:1 slope. Refer to ODOT's Location and Design Manual, Figure 307-2. To satisfy cost considerations or in order to maintain the typical roadway section (including roadway ditch) of the underpass through the structure, for four or more lane highways, wall abutments or the 2:1 slope of typical two-span grade separation structures may be located farther than 30 feet [9000 mm] from the near edge of traveled lane.

Lateral clearances for other roadway classifications shall be established in accordance with ODOT's Location and Design Manual, Section 302, unless specified in the scope of services or other contract criteria.

207.4 INTERFERENCE DUE TO EXISTING SUBSTRUCTURE

Where a new pier or abutment is placed at the location of an existing pier or abutment the usual "Removal" note (and also the text of CMS 202.03) calls for sufficient removal of the old pier or abutment to permit construction of the new. However, a new pier or abutment preferably should not be located at an existing pier or abutment where the existing masonry may extend appreciably below the bottom of the proposed footing, or appreciably below the ground in case of capped-pile construction. This applies particularly where piles are to be driven. It is desirable to avoid the difficulty and expense of removing deep underground portions of the existing substructure and to avoid the resultant disturbance of the ground.

Where existing substructure units are shown on the Site Plan, the accuracy of the locations and extent should be carefully drawn. The existing substructure configuration should be shown based on existing plans or field verified dimensions, otherwise just a vertical line showing the approximate face of the abutment or pier widths should be shown. Misrepresentation of the location of the existing substructure units has resulted in expensive change orders during construction. Existing dimensions should be labeled as (+/-) plus or minus.

207.5 BRIDGE STRUCTURE, SKEW, CURVATURE AND SUPERELEVATION

During the Assessment of Feasible Alternatives, the location of the proposed structure should be studied to attempt to eliminate the presence of excessive skew, curves or extreme superelevation transitions within the actual bridge limits.

208 TEMPORARY SHORING

208.1 SUPPORT OF EXCAVATIONS

Whenever shoring is required to support a roadway where traffic is being maintained and the height of the retained earth will be over eight feet [2.5 meters], the Design Agency shall be required to provide a temporary shoring design with details provided in the plans and feasibility studied during the Structure Type Study.

For projects involving Railroads, the requirements will be different as each railroad company has their own specific requirements. The Design Agency is responsible for contacting the responsible railroad and obtaining the specific requirements for design and construction.

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Following are some conceptual ideas for the design of temporary shoring:

A. A cantilever sheet pile wall should generally be used for excavation up to approximately 12 feet [3.5 meters] in height. Design computations are necessary.

B. For cuts greater than 12 feet [3.5 meters] in height, anchored or braced walls will generally be required.

C. For anchored walls, the use of deadmen is preferred. Braced walls using waler and struts can sometimes be braced against another rigid element on the excavated side.

The use of soil or rock anchors(tiebacks) is generally the last option considered in the design of anchored walls.

D. The use of steel “H” piles with lagging is also a practical solution for some sites. Please note that some railroad companies allow only interlocking steel sheet piling adjacent to their tracks.

E. Where sufficient embedment can not be attained by driving sheet piling because of the location of shallow bedrock, predrilled holes into the bedrock with soldier “H” piles and lagging should be considered.

   For cuts greater than 12-15 feet [3.5-4.5 meters], the “H” piles may need to be anchored.

F. The highway design live loading should be equal to two feet [600 mm] of equivalent soil height as a surcharge.

G. The following items at a minimum should be shown on the detail plans:

   1. Minimum section modulus

   2. Top and minimum bottom elevation of shoring

   3. Limits of shoring

   4. Sequence of installation and/or operations.

   5. Method of payment

   6. If bracing or tiebacks are required, all details, connections and member sizes shall be detailed.

   7. A general note in plans allowing a Contractor designed alternate for temporary shoring.

208.2 SUPPORT OF EXISTING STRUCTURE

Whenever temporary support is required for a portion of an existing structure used to maintain traffic, the Design Agency shall provide sufficient information in the plans to allow contractors to prepare bids and construct the project. The feasibility of temporary support of an existing structure should be considered and discussed during the Structure Type Study.

The design shown in the plans should include: permissible locations of temporary support; temporary support loads; construction sequences; construction limitations not otherwise provided in C&MS 501.05; and any remaining plan notes. As a minimum, the plan notes should address method of measurement and basis of payment for temporary support.

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Where S is the effective span length in feet [millimeters]. Tmin shall be rounded up to the nearest one-quarter inch [5 mm].

The one inch [25 mm] wearing thickness, Section 302.1.3.1, is included in the calculations for minimum concrete deck thickness but not in the calculations during actual structural design of the deck slab.

For transversely reinforced concrete deck slabs supported on steel stringers the effective span length "S" shall be considered equal to the distance center-to-center of stringers minus 6 inches [150 mm].

For concrete I-beam stringers the effective span length shall meet the requirements of AASHTO 3.24.1.2.

302.2.2 CONCRETE DECK DESIGN

The concrete deck design shall be in conformance with AASHTO, latest edition, and additional requirements in this Manual. The design live load shall be HS25 for decks on new superstructures and HS20 for decks on existing superstructures.

For continuous slabs on three or more supports a continuity factor of 0.80 shall be applied to the simple span bending moments for both live load and dead load.

See Figures 312 & 313 for an illustration of a method of design for a reinforced concrete deck slab. Design data tables for HS25 (Fig. 314A) and HS20-44 (Fig. 314B) live loads are also provided.

Upon completing the concrete deck design from the example shown in Figure 312 & 313, or similar method, the designer should assure any cantilevered deck overhang will not over stress the initial deck design due to the dead load and the greater live load of either the vehicle wheel loads or the railing live loads. See relevant AASHTO sections for live load application requirements. See example Figures 315 & 316.

Transverse spacing of the top and bottom reinforcing in a deck design shall meet section 302.2.4.2.

302.2.3 DECK ELEVATION REQUIREMENTS

302.2.3.1 SCREED ELEVATIONS

Screed elevations are control elevations for concrete deck finishing machines that account for dead load deflections to ensure that the bridge deck is completed to the correct elevation. To establish screed elevations, the final surface elevations are adjusted for non-composite deflections resulting from deck placement and composite deflections resulting from utility and railing loads. Screed elevations shall not include adjustment for deflections due to the future wearing surface loading. Calculated deflections caused by the weight of the deck concrete should assume a completed placement sequence. Use deflection data from girder lines closest to each screed line to determine elevations. Refer to Figure 302.2.3-1.

If the deflections are determined through a line girder analysis method, the deck load should be distributed evenly to all beams/girders loaded in each construction phase to establish screed elevations. If a refined analysis method is used, establish screed elevations using the individual beam/girder deflections.

The bridge plans shall include a screed elevations table. The locations of all screed elevations in the table should be identified on a transverse section and plan view. Elevations should be provided for all: curblines or deck edges; profile grade points; transverse grade-break lines; and phased construction lines for the full length of the bridge. Screed elevations are not required above beam/girder lines. Bearing points, quarter-span points, mid-span points and splice points shall be detailed as well as any additional points required to meet a maximum spacing between points of 25'-0" [7.5 m].

For bridges with a separate wearing course, the elevations given should be those at the top of the portland cement concrete deck. Provide a plan note stating at what surface the elevations are given in order to eliminate any confusion.

Screed elevations are not required for non-composite box beam bridges or slab bridges. Screed elevations for composite box beam bridges shall meet the same requirements as steel beam, girder and prestressed I-beam bridges.

302.2.3.2 TOP OF HAUNCH ELEVATIONS

Top of haunch elevations represent the theoretical bottom of deck elevation before the concrete deck is placed. Top of haunch elevations should be provided at the centerline of each girder at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25’-0” [7.5 m]. The top of haunch elevation locations should be identified in a plan view and on the transverse section. Top of haunch elevations are not required for composite box beam bridges. Provide a plan note for a definition and description of the purpose for the top of haunch elevations (see BDM Section 700). Refer to Figure 302.2.3-1.

302.2.3.3 FINAL DECK SURFACE ELEVATIONS

Final deck surface elevations represent the position of the deck after all dead loads except future wearing surface have been applied. Final deck surface elevations shall be provided at bearing points, quarter points, mid-span points, splice points and additional points to meet a maximum spacing between points of 25’-0” [7.5 m] for each: girder centerline; curbline or deck edge; transverse grade-break line; and phased construction line. The final deck surface elevation locations should be identified in a plan view. Refer to Figure 302.2.3-1.

302.2.4 REINFORCEMENT

302.2.4.1 LONGITUDINAL

Distribution reinforcement in the top-reinforcing layer of a reinforced concrete deck on steel or concrete stringers shall be approximately 1/3 of the main reinforcement, uniformly spaced.

Research has shown that secondary bars in the top mat of reinforced concrete bridge decks on stringers should be small bars at close spacing. Therefore the required secondary bar size shall be a #4 [#13M]. The only exception to this requirement is if the bar spacing becomes less than 3 inches [75 mm].

For stringer type bridges with reinforced concrete decks, the secondary bars shall be placed above the top of deck primary bars. This helps in reducing shrinkage cracking and adds additional cover over the primary bars.

For reinforced concrete deck slabs on stringer type bridges, where the main reinforcement is transverse to the stringers, additional top longitudinal reinforcement shall be provided in the negative moment region over the piers. This additional secondary reinforcement shall be equal to the distributional reinforcement (1/3 of the main reinforcement). This additional reinforcement shall be uniformly spaced and furnished in length equal to the larger of: 40 percent of the length of the longer adjacent stringer span or a length that meets the requirements of AASHTO 8.24.3.3.

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This reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 3 feet [1000 mm] longitudinally.

For composite designs, the total longitudinal reinforcement over a pier shall meet the requirements of AASHTO.

302.2.4.2 TRANSVERSE

To facilitate the placement of reinforcing steel and concrete in transversely reinforced deck slabs top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.

For steel beam or girder bridges with a skew of less than 15 degrees the transverse reinforcing may be shown placed parallel to the abutments. Bridges with a skew greater than 15 degrees or where the transverse reinforcing will interfere with the shear studs should have the transverse reinforcement placed perpendicular to the centerline of the bridge. Refer to the appropriate Standard Bridge Drawing for the requirements on slab bridges.

For prestressed I-beams, transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

For composite box beam decks, the transverse reinforcing steel may be placed parallel to the abutment.

For steel beam or girder bridges, the clearance of the bottom transverse bars over the top of any bolted beam splice plates or moment plates should be checked as reinforcing bars at a skew generally cannot be placed between bolt heads.

302.2.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam, girder or prestressed I-beam structures shall have a concrete haunch to prevent a thinning of the deck slab as a result of unforeseen variations in beam camber. At a minimum, the design haunch shall allow for 2 inches [50 mm] of excessive camber. For steel beam and girder structures, the haunch shall be tapered back to the original concrete deck thickness in a 9 inch [225 mm] length and the concrete haunch shall encase the edges of the top flange. See Figures 317 & 318.

302.2.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, shall not be used.

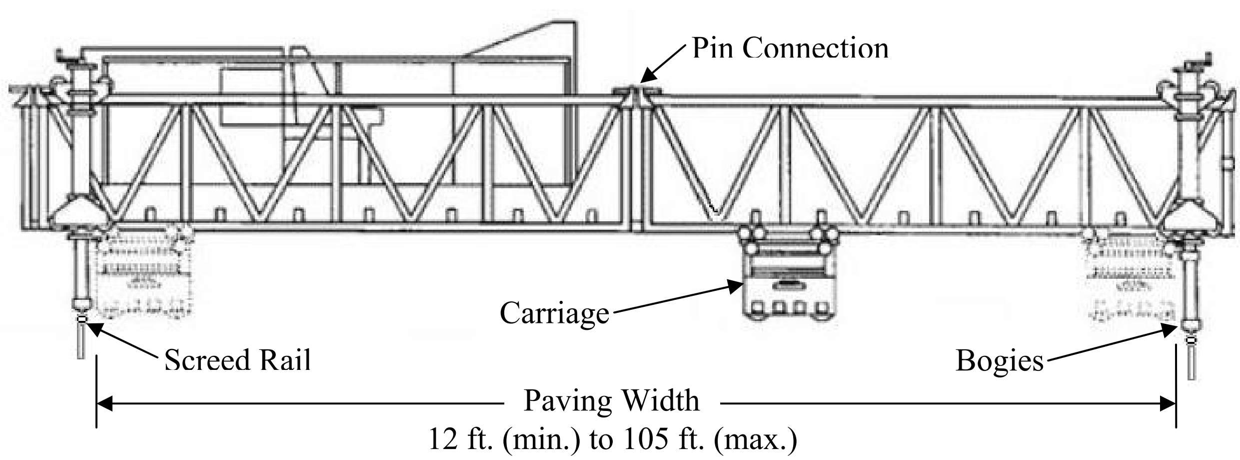
302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS

Mechanized finishing machines are preferred to hand finishing methods for both consistency of surface finish and economics. Designers should be aware of finishing machine limitations in order to avoid deck designs that require hand finishing methods.

The placement of deck concrete using mechanized finishing machines alone does not ensure a smooth riding surface. Achieving a smooth riding surface as well as ensuring the proper geometry of the concrete deck is further complicated by deflections of the concrete falsework and of the main structural support members during the placement operation. The Contractor is responsible for designing falsework and finishing machine support to minimize deflection during placement, but the Designer is responsible for deflections induced by deck placement on the superstructure. Many complications due to deflection during placement can be avoided with proper design considerations.

302.2.7.1 FINISHING MACHINES

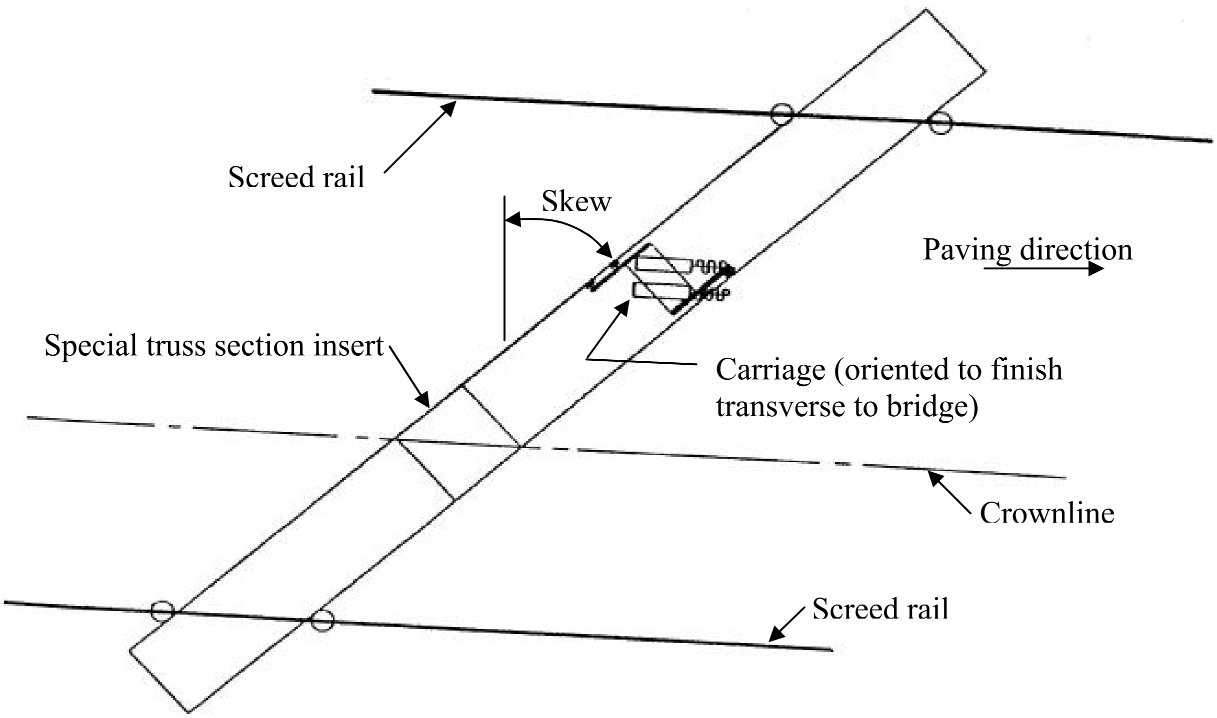
Mechanized finishing machines are comprised of fabricated truss sections pinned together to span the bridge deck width to be paved. The truss spans are supported at each end on a set of wheels, called “bogies,” which ride along the length of the bridge on screed rails. Suspended below the truss is a finishing head, called a “carriage,” which levels, compacts, vibrates and finishes the concrete.



Finishing machines can be placed such that the truss sections are skewed with respect to the screed rails. This orientation allows for concrete placement parallel to the substructure skew as required by the C&MS 511. For skew angles of 15º and greater, the finishing machine can be skewed to within 5 degrees of the plan specified skew angle.

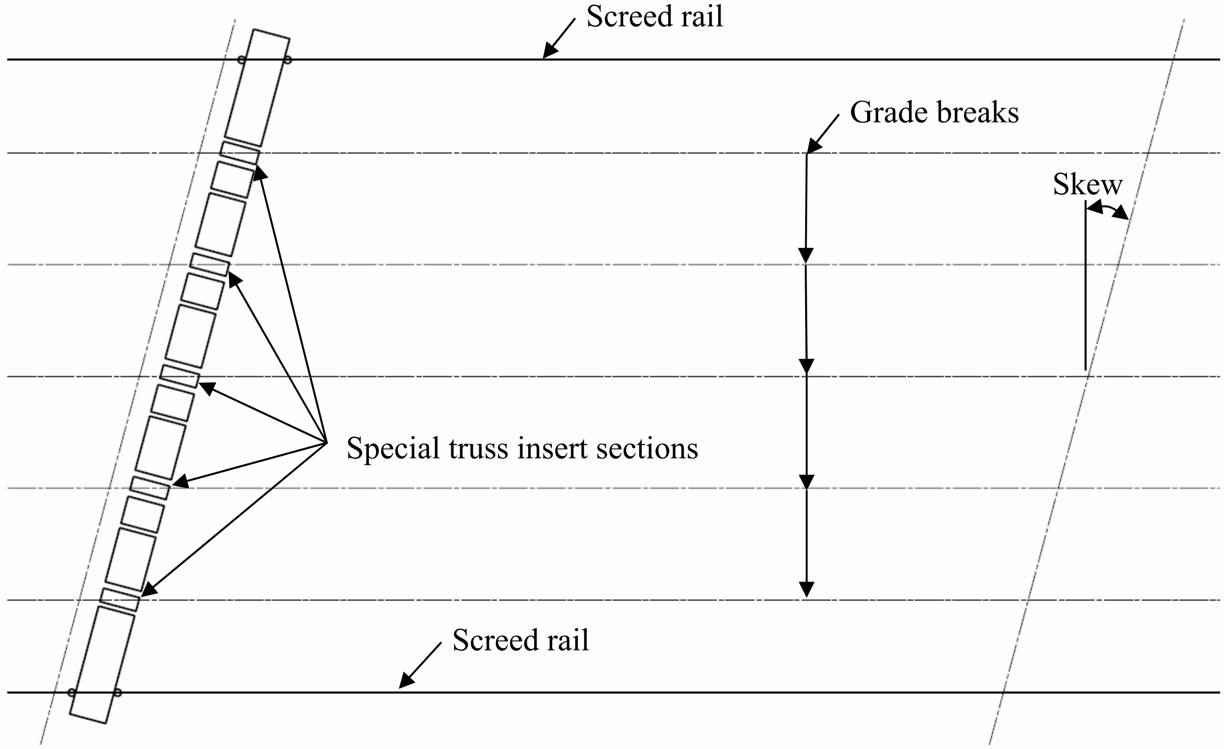
The carriage can also be skewed with respect to the truss sections. This feature allows the carriage to finish the concrete transverse to the bridge when the truss sections are placed at some other orientation (e.g. parallel to the substructure skew). In order to ensure a proper finish at transverse grade breaks (e.g. crown points), the carriage should always be oriented to finish the concrete transverse to the bridge. A special length truss section insert is required above the grade break locations such that the grade break line lies directly below opposite corners of the section.

For skewed bridges without transverse grade breaks, skewing the carriage with respect to the truss sections is not required.

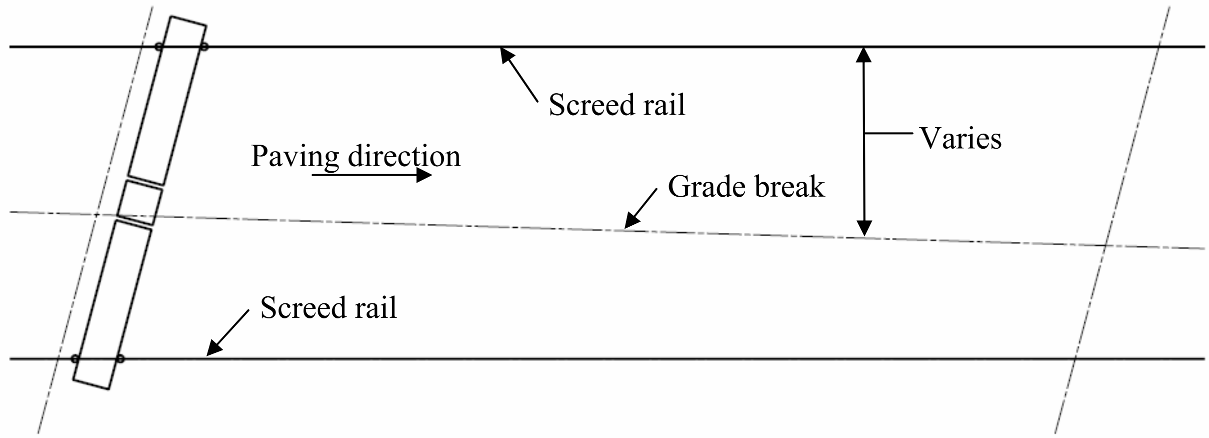


Most finishing machines do not easily accommodate non-parallel rails. The distance between the screed rails should be a fixed width. Designs that require tapered paving widths should be avoided.

The finishing machines can be hinged at the pin connections between truss sections in order to provide transverse grade breaks (e.g. crown points). In theory, multiple transverse grade breaks can be accommodated, but the grade breaks must remain at a fixed spacing in order to line up with a pin connection. The figure below illustrates the complexity of the machine set-up to accommodate multiple grade breaks in a transverse section placed on a skew. Note that the length of truss sections required between grade breaks must fit the standard truss section lengths.



Grade break locations that move laterally along the length of the bridge cannot be paved in a single operation using a mechanized finishing machine and should therefore be avoided. Note that as the machine progresses forward, the truss hinge locations and the grade break locations no longer coincide. See the figure below.



302.2.7.2 SOURCES OF GIRDER TWIST

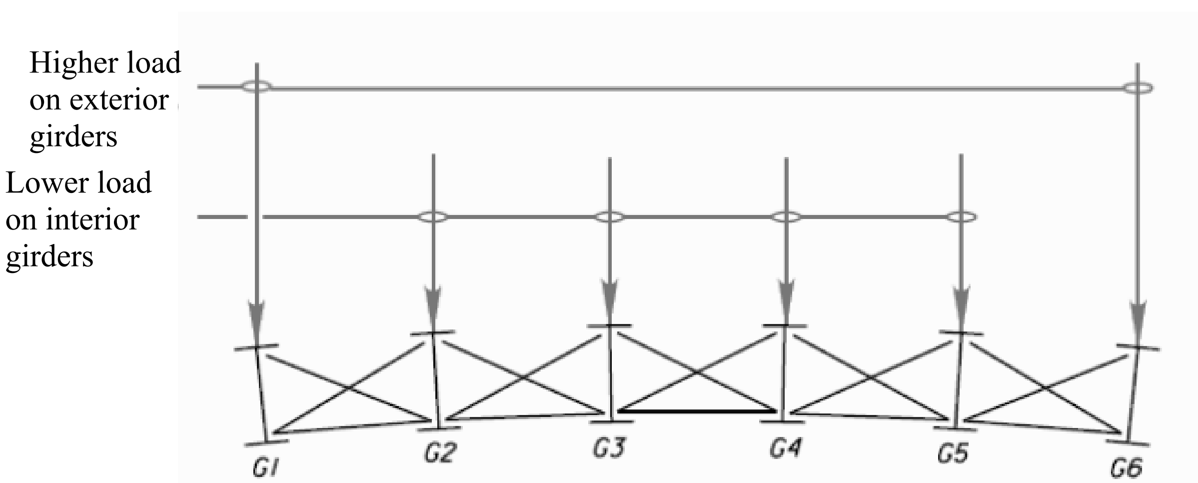
The interconnectivity between girders, intermediate crossframes/diaphragms and end crossframes/diaphragms is essential to a structure's stability throughout the construction process. Therefore, it is of utmost importance to ensure that all crossframes/diaphragms are fully installed prior to deck placement. Failure to do so may lead to construction disputes, expensive repairs and lengthy construction delays or even impact project safety. One major drawback to this interconnectivity is that the deflection caused by the placement of the concrete deck will result in girder twisting.

There are primarily three independent sources of girder twist resulting from deck placement. This manual will refer to these sources as: global superstructure distortion, oil-canning and girder warping.

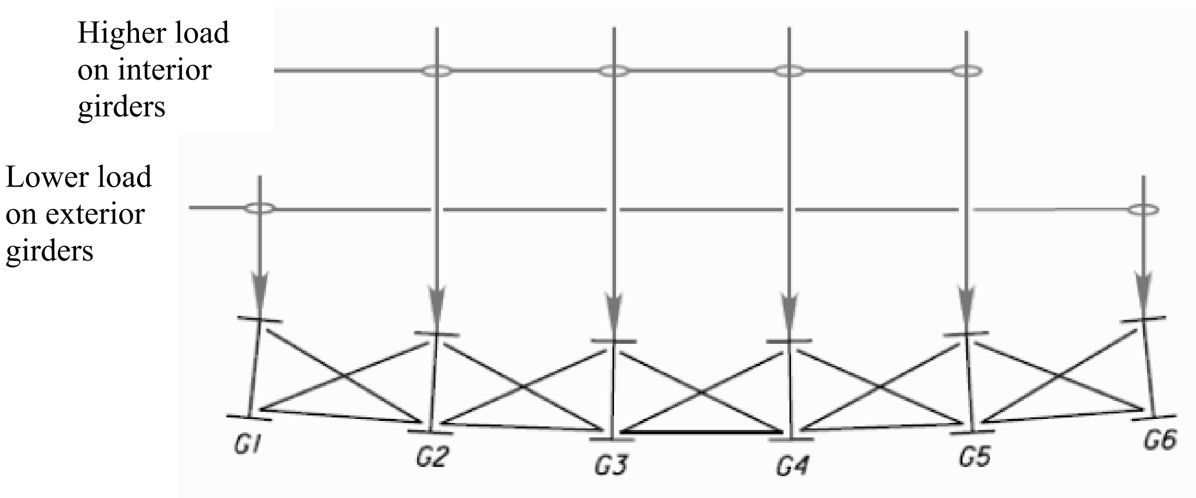
302.2.7.2.a GLOBAL SUPERSTRUCTURE DISTORTION

Global superstructure distortion is distortion of the bridge transverse section primarily caused by differential deflections between adjacent girders. As a girder deflects downward with respect to an adjacent girder, the rigidity of the cross framing between the two girders causes the deflecting girder to rotate as it deflects. This distortion may occur with both steel and prestressed concrete superstructures. The most common differential deflections occur between the exterior girders and adjacent interior girders for a given construction phase when the loaded tributary areas over the girders differ.

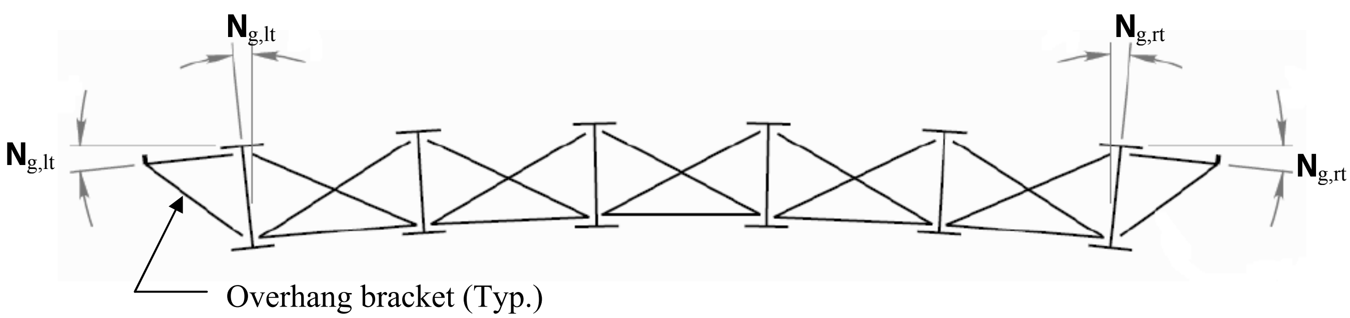
Transverse sections with more heavily loaded exterior girders distort in a convex shape.



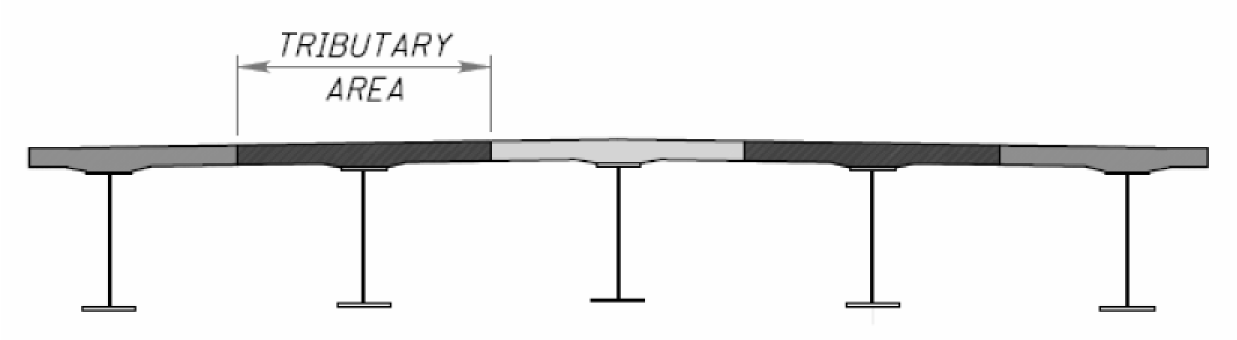
Transverse sections with more heavily loaded interior girders distort in a concave shape.



Twisting of the exterior girders can result in deck thickness and cover loss if the screed rails are supported on cantilevered falsework. The magnitude of girder twist (measured as **N**g) will vary over the length of the bridge and will be different for the left and right sides if loading or geometry is not symmetrical.



For bridges with tangent alignments and adjacent substructure skews that vary by no more than 15º, the magnitude of the girder twist can be reduced by utilizing transverse sections with balanced tributary deck loadings. For a new superstructure, the amount of girder twist due to global superstructure deformation can be neglected when the tributary deck load carried by the fascia girder does not exceed 110% of the average of the tributary deck load carried by the interior members for a given construction phase. For an existing superstructure, the amount of global deformation may be neglected when the tributary deck load carried by the fascia girder does not exceed 115% of the average of the tributary deck load carried by the interior members for a given construction phase.

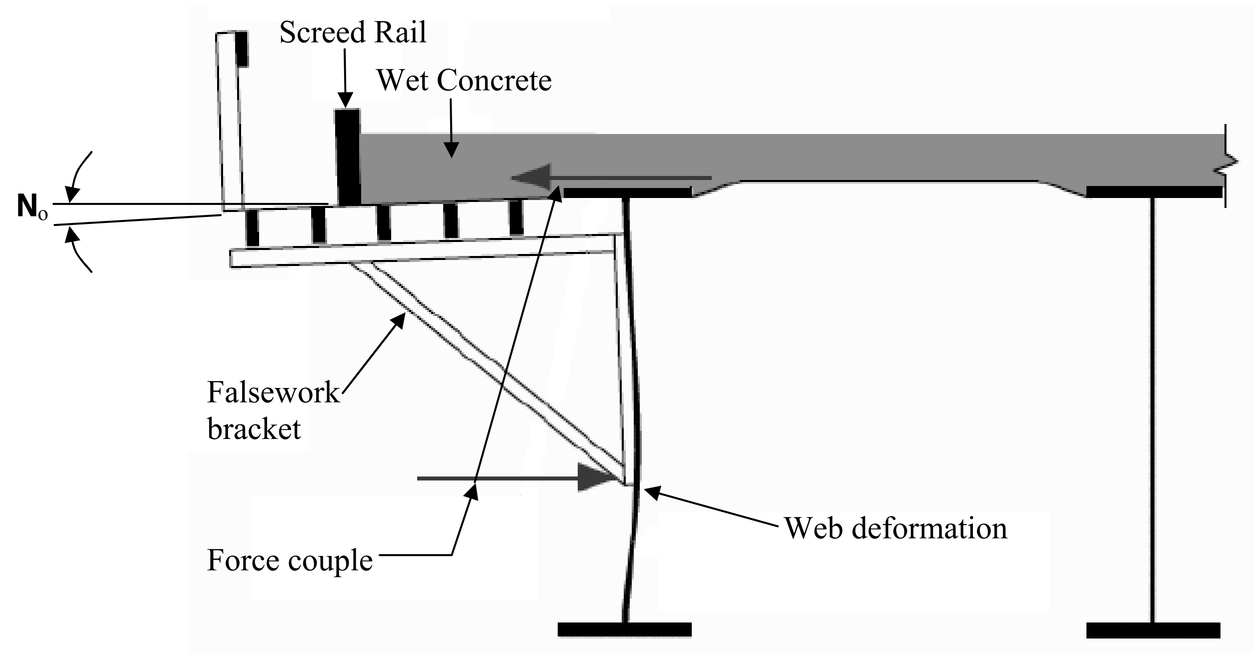


When the aforementioned tributary deck loading requirements of the fascia members cannot be met or, because of geometry, do not apply, the Designer shall perform a refined analysis of the superstructure system to determine the magnitude of fascia girder twist ($\mathbf{N}_g$) due to deck concrete placement. To properly calculate the effect of the twist angle on deck thickness, the analysis should be based on the deflection occurring due to the concrete present at the time that the finishing machine passes over the point under consideration. This degree of precision requires a separate refined analysis for each point of consideration. It is generally sufficient to calculate $\mathbf{N}_g$ based on the full wet concrete load placed over the entire structure. However, on complex structures with variable skews and/or curved girders, a higher degree of precision may be warranted to ensure proper deck thickness.

Additional measures to reduce global deformation include: adding or stiffening the crossframes/diaphragms; and increasing the stiffness of the girders. An increase in the crossframe stiffness results in better load distribution across the width of the structure and less distortion. An increase in the stiffness of the girders reduces the magnitude of vertical deflection resulting in less distortion of the transverse section.

302.2.7.2.b OIL-CANNING

Distortion due to oil-canning occurs when large lateral loads from the cantilevered deck slab falsework bracket deform the girder web.



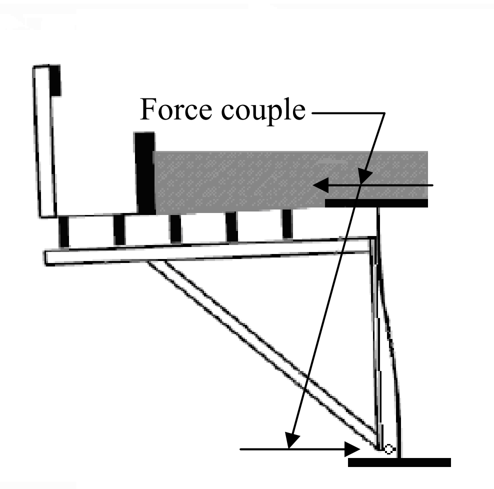
Locating the falsework bracket near the bottom flange will reduce the amount of web deformation. C&MS Item 508 requires the lower point of contact to be within 8" of the top of the bottom flange. Given this requirement and the geometric capabilities of the falsework brackets, the magnitude of girder twist (**N**o) resulting from oil-canning may be neglected for girder webs 84" deep or less.

For web depths greater than 84", designers shall provide the location of the falsework bracket in the plans. Provide a General Note that removes the lower point of contact requirement of C&MS Item 508 (see BDM Section 600 for an example). The pay item for deck concrete shall be "as per plan". Using the plan bracket location, designers shall determine **N**o. Designers may assume the lowest location of the falsework bracket to be 76" measured below the bottom of the top flange. The magnitude of twist can be predicted using finite element analysis of the web or by various approximate methods. If the magnitude results in excessive deck thickness loss, reducing the transverse stiffener spacing or adding temporary bracing on the inside of the web may be necessary. Any temporary bracing should be detailed in the plans.

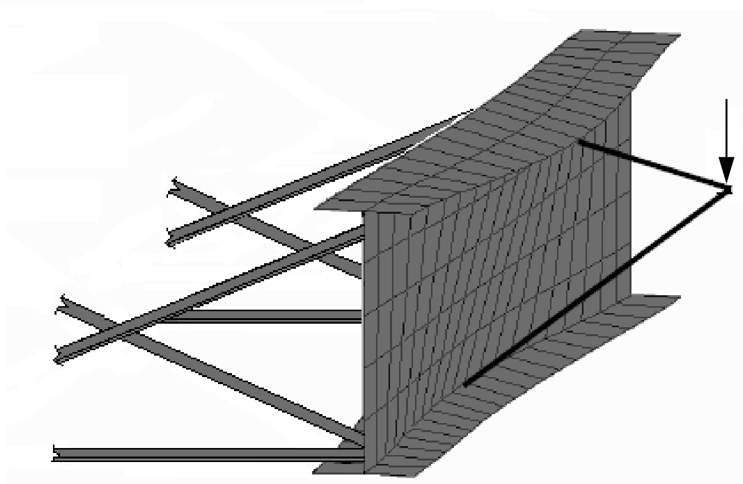
The magnitude of girder twist resulting from oil-canning may be neglected for prestressed I-beam superstructures.

302.2.7.2.c GIRDER WARPING

Distortion due to girder warping occurs as a result of deck slab overhang falsework loading on the fascia girder between points of lateral bracing (e.g. crossframes). The bracket loads produce twist between the crossframes due to a combination of girder warping and pure torsional distortion. The girder is restrained from warping at the crossframe locations. Due to the inherent torsional stiffness of prestressed I-beams, the distortion due to girder warping may be neglected. Other design considerations for I-beams due to the overhang bracket loadings are presented at the end of this section.



Girder warping



Torsional distortion

For steel superstructures, Designers should calculate the magnitude of twist ($\mathbf{N}_w$) due girder warping using the TAEG software developed by the Kansas Department of Transportation. TAEG ("Torsional Analysis of Exterior Girders") is available at no cost and can be downloaded at: http://www.ksdot.org/kart/.

Since most of the data input in TAEG is dependent upon the contractor's equipment and falsework design, designers should use conservative assumptions to accommodate most contractor resources. For design-build projects and value engineering change proposals (VECP's), data input for TAEG shall represent the actual falsework and equipment to be used by the contractor. Designers may use the following assumptions in lieu of actual contractor supplied information:

A. Girder Data:

   For bridges with constant web depths, designers may select the cross section with the least torsional resistance to represent the entire structure. For bridges with variable depth webs, designers may disregard the effect of girder warping in the web depth transition sections.

B. Bridge Lateral Data:

   Designers may select the largest crossframe spacing to represent the entire structure. For structures with variable beam spacings (i.e. flared girders) designers may select the largest spacing dimension to represent the entire structure. Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

C. Permanent Lateral Support Data:

The default crossframe type assumed by the TAEG software consists of a stiffener and diagonal x-bracing with top and bottom horizontal chords. In order to analyze the structure with a standard ODOT crossframe, designers should input stiffener dimensions and select the "Diaphragms (Inputted Ix)" option. For ODOT Type 1 crossframes, designers should assume a fictitious stiffener of dimensions: 5" x 3/8". Determine the diaphragm moment of inertia for all standard ODOT crossframes as follows:

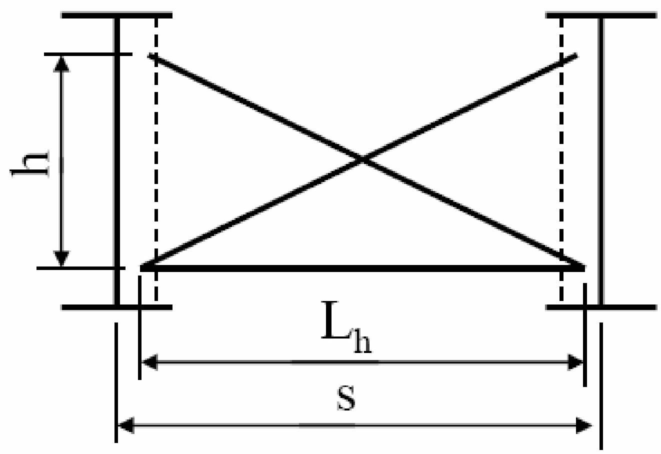
$$I_x = \frac{h^2 s}{4L_d^3\left(\frac{1}{A_d L_h^2} + \frac{L_h}{A_h L_d^3 + A_d L_h^3}\right)}$$

Where:

A_d = Area of the diagonal member (in^2)

A_h = Area of the horizontal member (in^2)

$L_d = \sqrt{L_h^2 + h^2}$



D. Temporary Lateral Support Data:

Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

E. Load Data:

1. Live Load on Walkway...50 lb/ft^2
2. Live Load on Slab...50 lb/ft^2
3. Dead Load of Formwork...10 lb/ft^2
4. Dead Load of Concrete .... $150/t_{avg}$ lb/ft^2

   (t_{avg} = Average thickness [ft.] of deck slab overhang)
5. Wheel Spacing [1-2-3]... 36" – 31" – 36"
6. Maximum Wheel Load:

   To estimate the total finishing machine length required for placement along the skew, add the rail-to-rail length and the extra end length from the following table using the plan specified skew rounded to the nearest 5 degrees. W is the rail-to-rail length as measured perpendicular to the centerline of the bridge.

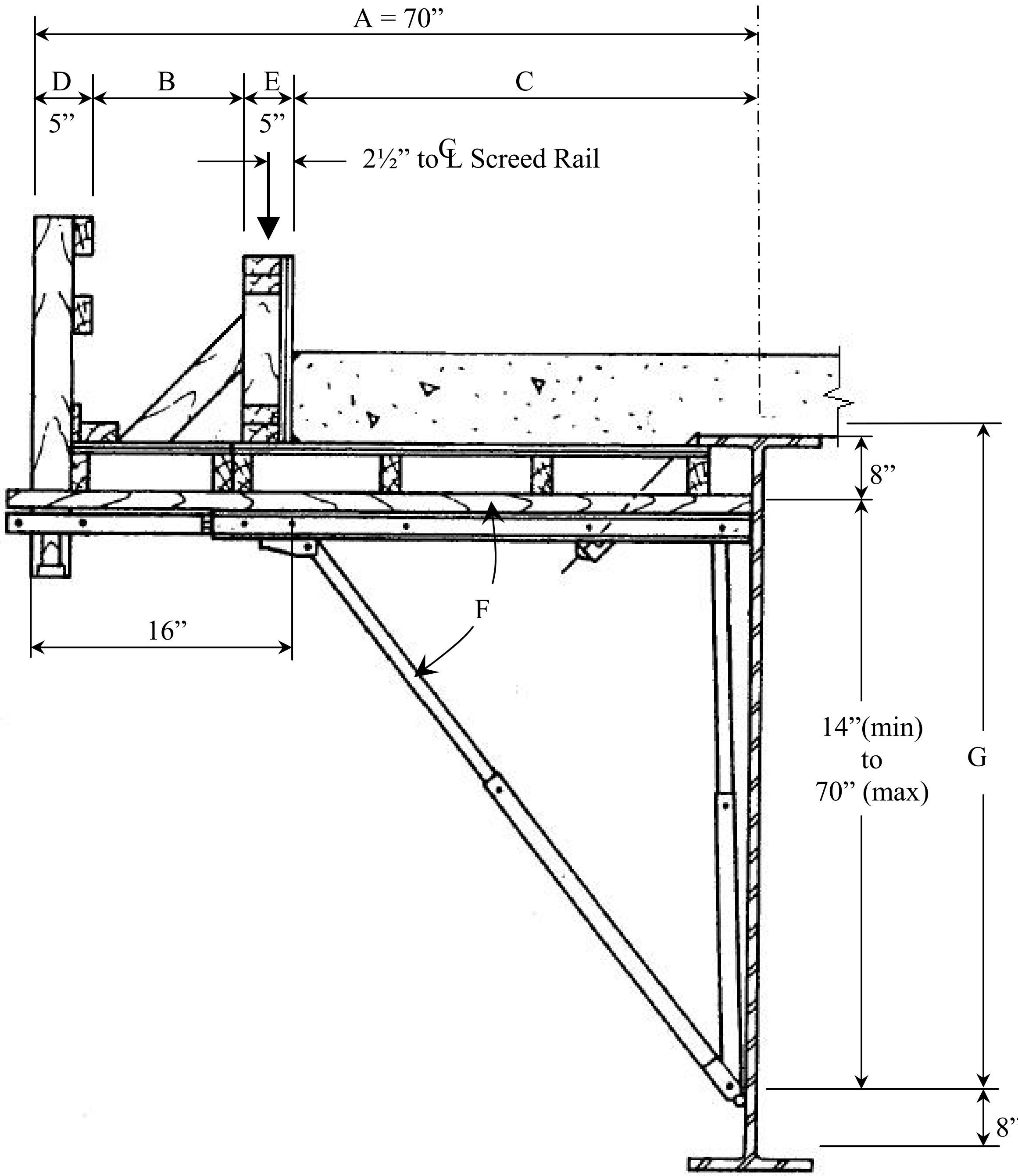
Skew Angle	Rail-to-Rail Length, ft.	Extra End Length, ft.
0	1.00 W	0.0
15	1.04 W	5.0
20	1.06 W	5.5
25	1.10 W	6.5
30	1.15 W	7.0
35	1.22 W	8.0
40	1.31 W	9.0
45	1.41 W	10.5
50	1.56 W	11.5
55	1.74 W	13.5

For total machine lengths of 36 ft. and less, assume a total machine weight of 7.6 kip. Add 0.09 kip for each additional foot of machine length required above 36 ft. The maximum total machine length shall not exceed 120 ft. If greater lengths are required, consult the Office of Structural Engineering for recommendations.

To determine the maximum wheel load, divide the total machine weight by 8.0.

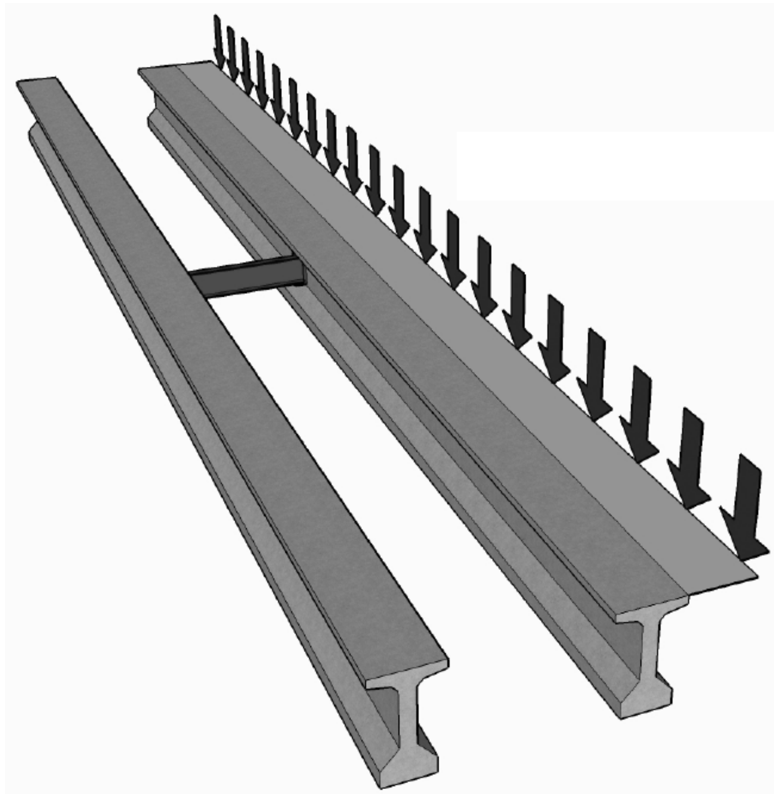
F. Bracket Data:

1. Refer to the following figure to determine TAEG dimensions A, B, C, D, E, F and G.
2. Designers may assume a center-to-center bracket spacing of 48.0 in.
3. Designers may assume a bracket weight of 50 lbs.



Assumptions for TAEG Bracket Data Input

For prestressed I-beam superstructures, Designers should verify that the intermediate crossframes/diaphragms in the exterior bay are capable of resisting the torsion caused by the cantilevered falsework.



302.2.7.3 DETERMINING EFFECT OF GIRDER TWIST

Once all sources of girder twist are quantified, Designers should determine the total effect that girder twist has on the finished deck surface. The primary effect of greatest concern is the loss of concrete cover over the top mat of deck reinforcing steel and the subsequent loss of deck thickness. The maximum loss due to twisting shall not exceed 0.5 in.
The total amount of girder twist at both the left and right screed rail should be determined as follows:

$\phi_{left} = (\phi_g + \phi_o + \phi_w)_{left}$ and $\phi_{right} = (\phi_g + \phi_o + \phi_w)_{right}$

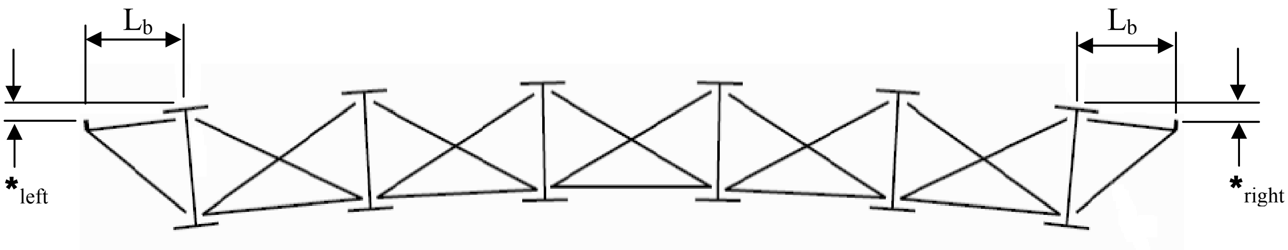
where:

N$_g$ = Girder twist due to global superstructure distortion (See BDM Section 302.2.7.2.a)

N$_o$ = Girder twist due to “oil-canning” (See BDM Section 302.2.7.2.b)

N$_w$ = Girder twist due to girder warping (See BDM Section 302.2.7.2.c)

The total amount of screed rail deflection at both the left and right screed rail should be determined as follows:



$\delta_{left} = \tan(\phi_{left}) \times L_b$ and $\delta_{right} = \tan(\phi_{right}) \times L_b$

where:

$*_{left}$ $*_{right}$ = Deflection of the screed rail due to total girder twist (in.). Upward deflection is positive and downward deflection is negative.

L_b = Lateral distance from center of screed rail to centerline of fascia girder (in.)

The total loss of deck thickness should be determined as follows:

$*_{Total} = (*_{left} + *_{right})/2$

302.2.8 SLAB DEPTH OF CURVED BRIDGES

For a curved deck on straight steel beams, steel girders or prestressed I-beams, the distance from the top of the slab to the top of the beams or girders will vary from end to end. The slab depth dimension shall show this variation by giving the maximum and minimum depth dimensions with their respective location, over the piers, center of span, etc.

An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

302.2.9 STAGED CONSTRUCTION

For all bridge types, except non-composite concrete box beams, where the differential dead load deflection between adjacent beams, girders or structural slabs is greater than ¼ inch [6 mm], a deck closure is required if the bridge is constructed in stages.

For requirements regarding closure pours on bridge widenings or on existing structures with new concrete decks see Section 400 of this Manual.

The closure pour between the stages shall be a minimum width of 30 inches [800 mm] but should be wide enough to accommodate the required reinforcing steel lap splices. In special cases, this distance may be reduced when mechanical reinforcing steel connectors are used (see

Section 200). The mechanical connector system used shall be able to develop 125 percent of the full yield strength of the reinforcing steel as a minimum.

Intermediate cross frames and diaphragms shall not be permanently attached in the closure pour location until the concrete pours on both sides of the closure pour location have been completed.

The two construction joints created by the concrete closure pour should be sealed with High Molecular Weight Methacrylate (HMWM), 705.15. The sealing width shown in the plans should be 2'-0" [600 mm], centered on the construction joints.

Placement of the staged construction joints above beam flanges is not recommended. The preferred location is the positive moment regions of the cast-in-place concrete deck slab.

The designer shall provide plan notes on the stage construction details sheet that detail the

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sequence of construction.

302.3 CONTINUOUS OR SINGLE SPAN CONCRETE SLAB BRIDGES

302.3.1 DESIGN REQUIRMENTS

Continuous reinforced concrete slab bridge design shall be in conformance with AASHTO, latest edition, and additional requirements in this Manual.

For simple span reinforced concrete slab bridges cast in place directly on concrete substructures, the effective span length shall be considered equal to the clear span plus 15" [380 mm].

The Designer shall include a final deck surface elevation table. Elevations shall be shown for all profile grade lines, curblines, crownlines, and phased construction lines for the full length of the bridge. Bearing points, quarter-span points and mid-span points shall be detailed as well as any additional points required to meet a maximum spacing between points of 30'-0" [10 m].

Details for simple span reinforced concrete slab bridge superstructures are provided in Standard Bridge Drawing SB-1-03.

Details for multi-span reinforced concrete slab bridge superstructures are provided in Standard Bridge Drawing CS-1-03.

302.4 STRUCTURAL STEEL

302.4.1 GENERAL

Structural steel shall be designed utilizing a composite section. A non-composite design may be used only if the design is the most economical.

Designs incorporating shear connectors in the negative moment region may be used.

All curved beams or girders shall be designed in accordance with AASHTO, this Manual and the latest AASHTO Guide Specifications for Horizontally Curved Highway Bridges including all interims.

The laterally unsupported length of top flanges of beam and girder members with a concrete deck encasing the top flange or compositely designed with studs shall be considered to be zero. In the absence of such fastening or direct contact of an individual beam or girder member, the unsupported length shall be considered as the distance between the diaphragms, struts, bridging, or other bracing.

For designs that assume the unbraced length of the top flange to be zero as mentioned above, the designer shall investigate the strength of the non-composite section during steel erection, deck

302.4.1.9.b STRESS CATEGORY

In order to allow for future rehabilitation involving welded attachments, steel member designs in the negative moment regions should be limited to an allowable fatigue stress range of Category C, even though shop or field welded attachments are avoided in the original design.

302.4.1.10 TOUGHNESS TESTS

On steel structures, main load carrying members, such as beams, moment plates, bolted joint splice plates (excluding fill plates) require Charpy V Notch Testing. These components shall be identified on the detail plans by placing "(CVN)" after the component's description.

Example: W36x150 (CVN) [W920 x 223 (CVN)]

The web and all flanges of plate girders shall be CVN material.

Cross frame members, cross frame connection stiffeners and any steel connecting these elements on horizontally curved beam or girder structures are considered main members and shall require and be identified on the detail plans as CVN.

302.4.1.11 STANDARD END CROSS FRAMES

End cross frames for needed support and reduction of deflection of expansion devices should be designed to provide support at intervals not exceeding 4'-0" [1200 mm]. Standard expansion joints have designs already established as part of the standard drawings. For suggested details of special conditions review existing expansion joint Standard Bridge Drawings.

302.4.1.12 BASELINE REQUIREMETNS FOR CURVED AND DOG-LEGGED STEEL STRUCTURES

CMS 513 requires the fabricator to include in the shop drawings an overall layout with dimensions showing the horizontal position of beam or girder segments with respect to a full-length base or workline. Offsets from this full-length base line are to be provided by the fabricator for each 10 feet [3000 mm] of length. The designer shall provide this baseline in the plans along with enough information for the fabricator to be able to readily calculate the required offsets. The requirement for this information is especially critical on structures located on a curve or spiral or having other complex geometry.

302.4.1.13 INTERMEDIATE EXPANSION DEVICES

Intermediate expansion devices for a structure, if required, shall be located over a pier and the structural members shall be designed to be discontinuous at that pier.

302.4.1.14 BOLTED SPLICES

Bolted splices for rolled beams are detailed in a Standard Bridge Drawing. The standard incorporates Load Factor designed beam splices for A709[M] grade 36, 50 and 50W steel materials. The designer is required to confirm that the capacity of the standard splice is greater than the actual loads for the designer's structure.

For galvanized structures the designer should not specify standard drawing splices. The bolt hole size requires a 1/16 inch [1.5 mm] increase over the standard drawing’s hole size to allow for the additional thickness of the zinc coating. This increase in hole size decreases the standard drawing’s splice capacity. The designer should either evaluate the standard drawing splice based on the decreased capacity or design a new splice.

Bolt allowable stresses for painted surfaces or unpainted weathering steel surfaces shall be based on AASHTO 's values for Class A, Contact Surface, Standard Hole Type.

Bolt allowable stressed for metallized surfaces shall be based on AASHTO’s values for Class C, Contact Surface, Standard Hole Type.

Bolt allowable stressed for galvanized surfaces shall be based on AASHTO’s values for Class C, Contact Surface, Oversized Hole Type.

Beams having bolted splices at bend points shall have additional details incorporated in the plans to completely detail the joint requirements. The minimum edge distances specified in AASHTO shall be provided at the edges of all main members and splice plates.

For splices at bend points the lines of holes in the beam or girder flanges should be parallel to the centerline of the web. If the bend angle is small enough use rectangular splice plates (splice plates should not overhang flange by more than ½ inch [13 mm] and inside splice plates should not have to be trimmed to clear web or web to flange radius). When the angle is too large to allow rectangular splice plates the plates should be trimmed to align with the flange edges. In either case minimum edge distances shall be met.

Bolted compression splices, such as in a column, while designed as a friction type connection, also require the ends of the spliced members to be in full bearing by milling of the ends. For compression splice members with milled ends the AASHTO requirements of Section 10.18.3.1 shall be met.

The designer should recognize that “FULL BEARING” of beams and girders is not defined by AASHTO. “FULL BEARING” has been generally defined by ODOT as 75 percent of the bearing surface in contact and the other 25 percent with no gap greater than 1/32 inch [0.8 mm]. The designer shall specify the required fit definition when designing in conformance to the AASHTO design requirements for bolted splices in compression members.

Refer to Figure 302.4.1.14-1 for additional bolted splice details.

302.4.1.14.a BOLTS

Field splices in beams and girders shall be bolted connections using high strength bolts, ASTM A325[M].

The designer shall specify the diameter of the bolts and check that the type (Type I for Galvanized or Type III for Weathering) of A325[M] bolts is described in the coating notes or bolt material specifications.

Coating systems that are zinc based, such as OZEU, IZEU, Galvanizing or Metallizing require galvanized Type I bolts.

Un-coated weathering steel structures shall have A325[M], Type III bolts. If the faying surfaces under both the head and nut of every bolt of a weathering steel member are coated, specify galvanized A325[M] Type I bolts. Otherwise, specify A325[M], Type III bolts.

Generally, bolted splices should be designed using 1 inch [25 mm] or 1⅛ inch [29 mm] diameter bolts. No metric bolts or studs are available in the small quantities required for bridges.

The use of A490[M] bolts is not permitted.

302.4.1.14.b EDGE DISTANCES

1" [25 mm] diameter bolts used in splice plates should be detailed to allow for 2" [50 mm] edge distances in lieu of the AASHTO requirements. 1⅛ inch [29 mm] diameter bolts used in splice plates should be detailed to allow for 2¼ inch [60 mm] edge distances in lieu of the AASHTO requirements.

This increase to AASHTO's edge distances is to help alleviate the problem fabricators have of drilling bolt holes in flange splice plates and maintaining required minimum edge distances, especially on the inside splice plates.

If larger diameter bolts are specified the designer shall add ¼ inch [6 mm] to the AASHTO minimum edge distance.

302.4.1.14.c LOCATION OF FIELD SPLICES

Generally bolted splices should be located at points of dead load contraflecture on a continuous structure. Splices may also be supplied to help meet shipping and handling limitations. Plans should show optional field splice locations.

302.4.1.15 SHEAR CONNECTORS

AASHTO Sections 10.38.2.3 and 10.38.2.4 on studs shall be followed.

Shear studs shall be automatic welded studs. The use of channel sections is not allowed. 7/8 inch [22 mm] diameter studs are recommended as a standard diameter. The length of stud specified should be checked with manufacturers as to availability.

The Department's policy of using a 2 inch [50 mm] deep haunch over the top flange will have an effect on the length of shear studs.

Shear studs shall be field installed. In the case of galvanized structures, the design plans shall allow shop installation of studs prior to galvanizing or field installation after removing the coating by grinding at each stud location. If the studs are shop installed, the Contractor will be responsible for meeting all applicable OSHA requirements. A Detail note is available in Section 700.

302.4.2 ROLLED BEAMS

Effective in January 2006, the producers of rolled beams implemented changes to the physical dimensions of the W36X16 group of shapes (i.e. beams with 16" and wider flanges). The traditional W36X16 series of shape sizes will no longer be available from the producers. Below is a complete list for the new W36X16 group of shapes.

Designation	Area, A (in^2)	Depth, d (in)	Flange		Web Thickness, t_w (in)
			Width, b_f (in)	Thickness, t_f (in)	
W36 X 800	236.4	42.55	17.990	4.290	2.380
W36 X 652	192.5	41.05	17.575	3.540	1.970
W36 X 529	156.1	39.79	17.220	2.910	1.610
W36 X 487	143.8	39.33	17.105	2.680	1.500
W36 X 441	130.2	38.85	16.965	2.440	1.360
W36 X 395	117.4	38.41	16.830	2.200	1.220
W36 X 361	106.5	37.99	16.730	2.010	1.120
W36 X 330	97.4	37.67	16.630	1.850	1.020
W36 X 302	89.3	37.33	16.655	1.680	0.945
W36 X 282	83.4	37.11	16.595	1.570	0.885
W36 X 262	77.4	36.85	16.550	1.440	0.840
W36 X 247	72.9	36.67	16.510	1.350	0.800
W36 X 231	68.5	36.49	16.470	1.260	0.760

302.4.2.1 GALVANIZED BEAM STRUCTURES

If a galvanized bridge structure is the selected structure type, the following problems should be recognized and dealt with by the designer.

Galvanizing tanks are shallow and normally not longer than 45 feet [13.7 meters] in length.

Include in the table of Estimated Quantities.

303.4.2.2 PILES, NUMBER & SPACING

The designer shall comply with the following maximum center-to-center spacing of piles:

A. In capped pile piers, 7.5 feet [2300 mm].

B. In capped pile abutments, 8 feet [2500 mm].

C. In stub abutments, front row, 8 feet [2500 mm].

D. In wall type abutments and retaining walls, front row, 7 feet [2100 mm].

E. Cap and column piers should have at least 4 piles per individual footing.

303.4.2.3 PILES BATTERED

The path of battered piles should be checked to see that the piles remain within the right-of-way and do not interfere with piles from adjacent and existing substructure units nor conflict with portions of staged construction.

In general, a batter of 1:4 is considered desirable, but in cases where sufficient resistance is not otherwise attainable, a batter of 1:3 may be specified.

Piles should be battered to resist the stream forces. Battered piles also should be provided where necessary to avoid settlement due to group action by increasing the periphery of the soil mass.

Abutment piles should be battered normal to the centerline of bearings.

Piles fewer than 15 feet [5 meters] in length should not be battered.

When friction battered piles are specified, include note **[30d]** from Section 600 in the plans.

303.4.2.4 PILES, DESIGN LOADS

The pile's Ultimate Bearing Value, based on calculation of dead and live load transferred to the piles shall be given in the structure General Notes.

Ultimate Bearing Value load is equal to the actual unfactored design load multiplied by a safety factor of two (2).

The largest of these calculated individual pile Ultimate Bearing Value loads for each substructure unit should be used as the Ultimate Bearing Value for that substructure unit. This value for each substructure shall be listed in the structure General Notes.

The following table for H-piles should be used for selecting the required pile size based on the calculated Ultimate Bearing Value load for each substructure unit.

H Pile Size	Maximum Design Load	Ultimate bearing Value
HP10x42 [HP250X62]	75 Tons [650 kN]	150 Tons [1300 kN]
HP12x53 [HP310X79]	95 Tons [850 kN]	190 Tons [1700 kN]
HP14x73 [HP360X108]	130 Tons [1150 kN]	260 Tons [2300 kN]

Design load values for H piles are based on a maximum service load stress of 12.5 ksi [86 MPa] for Grade 50 steel.

The following table for pipe piles should be used for selecting the required pile size based on the calculated Ultimate Bearing Value load for each substructure unit.

Pipe Pile Diameter	Maximum Design Load	Ultimate bearing Value
12" [300 mm]	50 Tons [450 kN]	100 Tons [900 kN]
14" [350 mm]	70 Tons [650 kN]	140 Tons [1300 kN]
16" [400 mm]	90 Tons [800 kN]	180 Tons [1600 kN]

The actual value listed in the structure general notes should not be the Ultimate Bearing Value of the pile size selected, whether H pile or Pipe pile, but the calculated Ultimate Bearing Value load of the substructure unit or units.

Maximum specified pile spacings and maximum allowable Ultimate Bearing loads should be utilized to minimize the number of piles.

303.4.2.5 PILES, STATIC LOAD TEST

The Designer shall specify a Static Load Test when the total estimated pile length for an individual structure exceeds 10,000 ft [3000 m] for piling of the same size and Ultimate Bearing Value. Static load testing is not required for piling driven to refusal on bedrock.

The Designer shall specify one subsequent static load test for each additional 10,000 ft [3000 m] increment of estimated pile length. Each static load test requires two dynamic testing items and two restrike items. Restrikes are a useful tool to determine if a driven pile gains or loses capacity over time.

The results of both the static and dynamic testing shall be forwarded to the Office of Structural Engineering to the attention of the Foundations Engineer. Refer to Section 600 for a General Note to include in the plans.

303.4.2.6 PILES, DYNAMIC LOAD TEST

The Department now requires dynamic load testing to establish the driving criteria (i.e. blow count) for all piling not driven to refusal on bedrock. The dynamic testing and resulting wave analysis has replaced the Engineering News Record Formula, used in previous issues of the CMS.

For an individual structure, the Designer shall specify one dynamic load testing item for each pile size. If multiple pile capacities are required for a given pile size, the Designer shall specify one testing item for each Ultimate Bearing Value. When static load tests are required, provide two dynamic load testing items and two restrike items for each static load test item.

The driving criteria for battered piles will be determined in the field as a function of a dynamically tested vertical pile of the same Ultimate Bearing Value. When battered piles are specified, refer to Section 600 for a General Note to include in the plans.

One dynamic load testing item consists of testing a minimum of 2 piles and performing a CAPWAP analysis on one of the two piles. One restrike item consists of performing dynamic testing on two piles and performing a CAPWAP analysis on one of the two piles.

303.4.2.7 PILE FOUNDATION – DESIGN EXAMPLE

The following example for a 6-span bridge shall be used as a guide for specifying pile testing and estimated quantities for pile foundations.

Rear Abutment ~
30 - 12" C.I.P. Reinforced Concrete Piles
20 piles installed vertical & 10 piles battered
Ultimate Bearing Value = 76 ton
Estimated Length = 65 ft
Order Length = 70 ft (Total Length = 2100 ft)

Requires 1 dynamic load-testing item.

Piers 1, 2, 3, & 4 ~
80 - 14" C.I.P. Reinforced Concrete Piles at each pier
56 piles installed vertical & 24 piles battered
Ultimate Bearing Value = 125 ton

Estimated Length = 70 ft
Order Length = 75 ft (Total Length = 24,000 ft)

The total length (24,000 ft) requires 1 static load test item and 1 subsequent static load test. Each static load test requires 2 dynamic load testing items and 2 restrike items.

Pier 5 ~
52 - 14" C.I.P. Reinforced Concrete Piles
36 piles installed vertical & 16 piles battered
Ultimate Bearing Value = 135 ton
Estimated Length = 85 ft
Order Length = 90 ft (Total Length = 4680 ft)

The difference in Ultimate Bearing Value between piers 1, 2, 3 & 4 and pier 5 requires 1 dynamic testing item.

Forward Abutment ~
30 - 12" C.I.P. Reinforced Concrete Piles
20 piles installed vertical & 10 piles battered
Ultimate Bearing Value = 76 ton
Estimated Length = 75 ft
Order Length = 80 ft (Total Length = 2400 ft)

No additional dynamic load testing items are required.

For this example, the Designer should include notes **[30]**, **[30c]** and **[30d]** from Section 606.2 in the General Notes. Note **[30]** should be modified as follows:

PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is 76 ton per pile for the rear and forward abutment piles. The Ultimate Bearing Value is 125 ton per pile for Pier 1, 2, 3, and 4 piles and 135 ton per pile for Pier 5 piles.

Abutment Piles:
30 piles 70 ft long, order length (Rear)
30 piles 80 ft long, order length (Forward)
1 dynamic load testing item

Pier 1, 2, 3, and 4 Piles:
320 piles 75 ft long, order length
1 static load test item
1 subsequent static load test item
4 dynamic load-testing items
4 restrike items

Pier 5 Piles:
52 piles 90 ft long, order length
1 dynamic load testing item

The Designer should provide the following items in the Estimated Quantities:

Item	Extension	Total	Unit	Description
506	11100	Lump	Sum	Static Load Test
506	12200	1	Each	Subsequent Static Load Test
507	00500	4200	ft	12" Cast-In-Place Reinforced Concrete Piles, Driven
507	00550	4500	ft	12" Cast-In-Place Reinforced Concrete Piles, Furnished
507	00600	26,820	ft	14" Cast-In-Place Reinforced Concrete Piles, Driven
507	00650	28,680	ft	14" Cast-In-Place Reinforced Concrete Piles, Furnished
523	20000	6	Each	Dynamic Load Testing
523	20500	4	Each	Restriking

303.4.3 DRILLED SHAFTS

3'-6" [1065 mm] diameter drilled shafts for piers and 3'-0" [915 mm] diameter shafts for abutments are normally used.

The diameter of bedrock sockets of a drilled shaft are generally 6 inches [150 mm] less in diameter than the diameter of the drilled shaft above the bedrock elevation. The 6 inch [150 mm] downsize can be eliminated for abutment shafts. Reinforcing steel cages should be based on the bedrock socket diameter.

The drilled shaft diameter for the abutment shafts can be shown as one constant diameter for the full length of the drilled shaft (through bedrock and through soil).

Spiral reinforcement used in the drilled shaft is normally a #4 [#13M] bar at a 4½ inch [115 mm] pitch with a spiral diameter of 6 inches [150 mm] less, out to out of spiral cage than the drilled shaft diameter. (Note AASHTO specifications do not recognize a 4½ inch [115 mm] pitch as meeting spiral requirements definition 8.18.2.2.3) When steel casing is left in place, a pitch of 12 inches [300 mm] should be used for the spiral reinforcing.

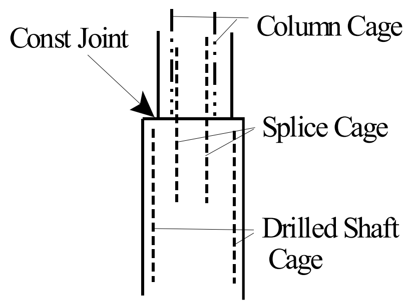
Drilled shafts with diameter s of less than 3'-0" [915 mm] are not recommended.

The diameter of the drilled shafts should be 6 inches [150 mm] larger than the pier column diameter so that if the drilled shaft is slightly misaligned, the pier column can still be placed at plan location, although the pier column would not be exactly centered on a misaligned-drilled shaft.

For record and project use, each drilled shaft for a structure shall be individually identified by a unique number. The designer may choose to number the drilled shafts on the individual substructure plan sheet or on a separate drilled shaft foundation layout sheet.

A construction joint between the drilled shaft and any column will be required. Therefore the designer will need to specify reinforcing steel, incorporating the required lap splices, at the construction joint.

The designer should develop a lap splice that will allow both for required lap and minimum cover due to mis-alignment of the drilled shaft versus the column. Possible alternatives are two cages, one for the drilled shaft diameter and a second splice cage for the lap to the column.



When the exposed length of the pier columns is relatively short, one full length reinforcing steel cage, from the bottom of the drilled shaft up into the pier cap, should be designed. The steel cage should be designed to provide a 3 inch [75 mm] concrete cover within the pier column.

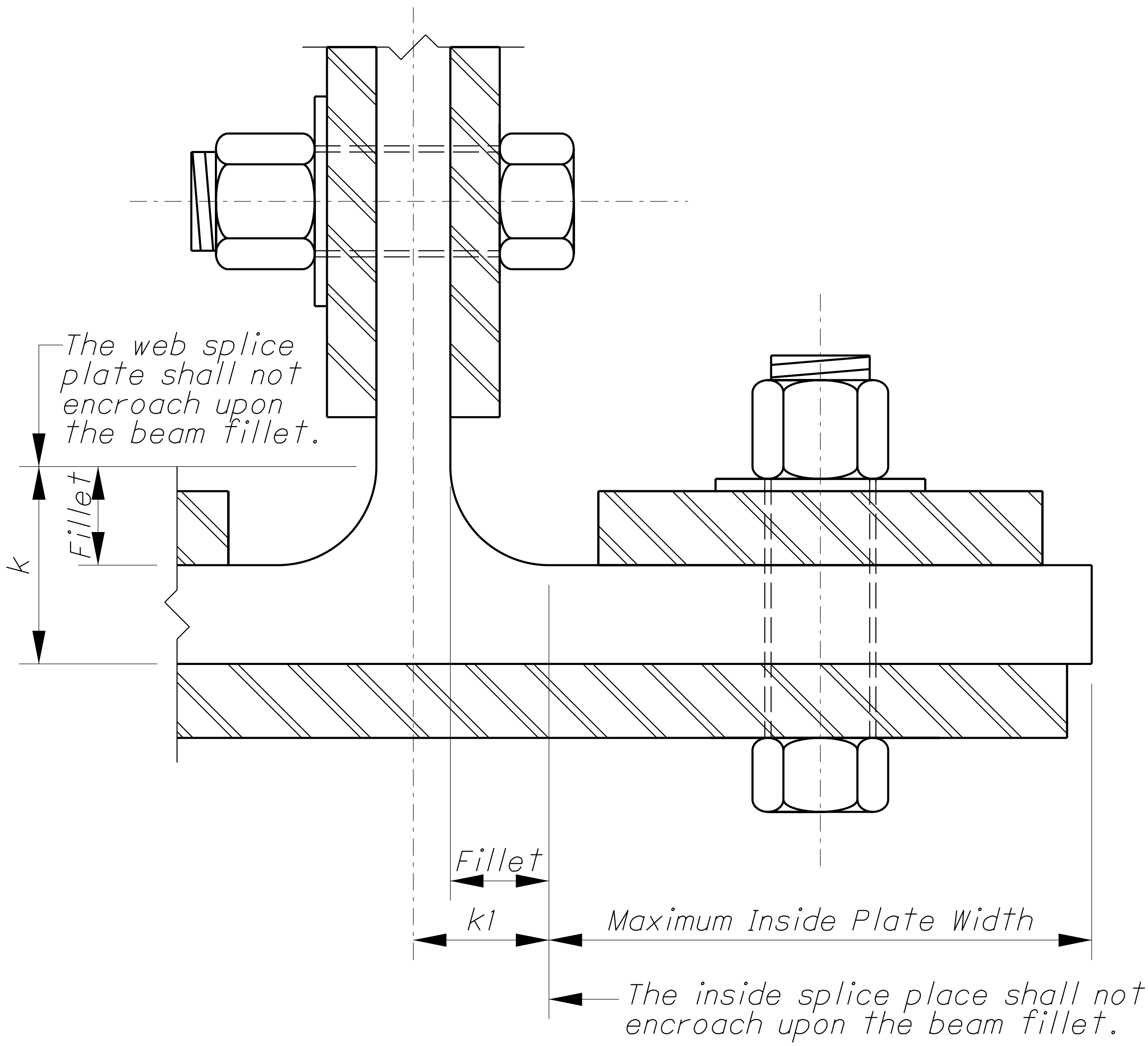
When the drilled shaft is socketed into bedrock, the quantity of the reinforcing steel in the drilled shaft, including the portion extending into the rock socket, should be included with Item 524 “Drilled Shaft, Above Bedrock” for payment. For drilled shafts with friction type design where the tip elevation is known, the reinforcing steel should be paid under Item 524, Drilled Shafts. The Designer shall also specify the reinforcing steel to be epoxy coated according to 709.00.

A general note as listed in Section 600 will be required.

The top of the drilled shaft is defined as 1 foot [0.3 meter] above normal water elevation, for piers in water, and 1 foot [0.3 meter] below the ground surface for piers not in water.

303.5 DETAIL DESIGN REQUIREMENTS FOR PROPRIETARY RETAINING WALLS

Supplemental Specification 840 defines the requirements for construction and design for internal stability for Mechanically Stabilized Earth (MSE) walls. The project plans shall include a reference to SS 840 when MSE walls are shown. Special provisions are required for other types of proprietary walls.



The web splice plate shall not encroach upon the beam fillet.
Fillet
k
Fillet
k1
Maximum Inside Plate Width
The inside splice place shall not encroach upon the beam fillet.

Figure 334

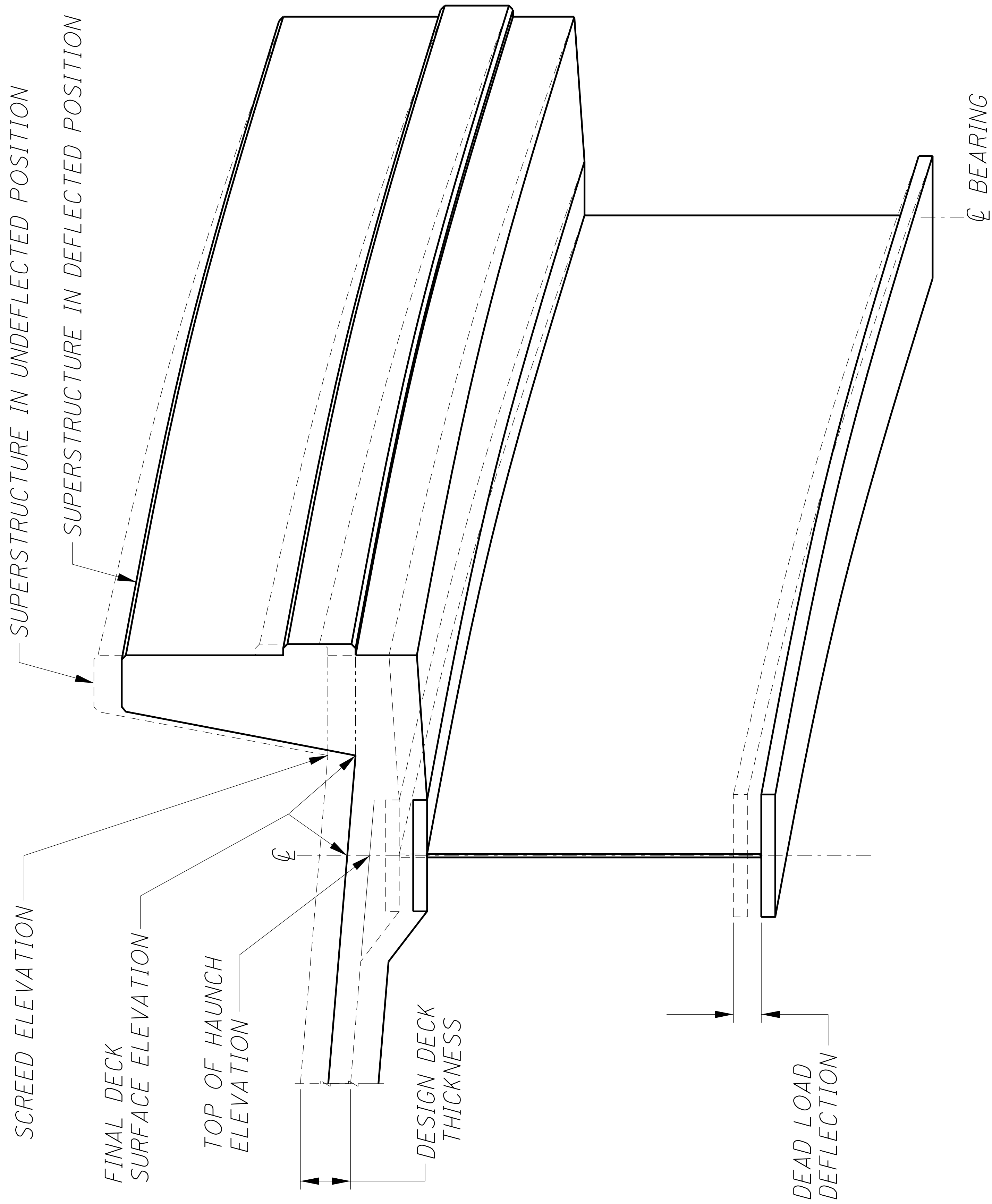

SUPERSTRUCTURE IN UNDEFLECTED POSITION
SUPERSTRUCTURE IN DEFLECTED POSITION
℄ BEARING
℄
SCREED ELEVATION
FINAL DECK SURFACE ELEVATION
TOP OF HAUNCH ELEVATION
DESIGN DECK THICKNESS
DEAD LOAD DEFLECTION

Figure 335

abutment. The design for internal stability shall include an unfactored horizontal strip load from the superstructure of _____ k/ft [kN/m] applied perpendicular to the face of wall at the base of the concrete footing.

606 FOUNDATIONS

606.1 PILES DRIVEN TO BEDROCK

The following note generally will apply where steel-H piles are to be driven to bedrock:

[29] PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating soft bedrock for several inches to a minimum resistance of 20 blows per inch [25 mm] or by contacting hard bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish a driving criteria for each pile type and capacity. Establish the driving criteria to achieve the Ultimate Bearing Value given below for the piles. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven.

The Ultimate Bearing Value is __#__ tons [kN] per pile for the ____ abutment piles. The Ultimate Bearing Value is __#__ tons [kN] per pile for the ____ pier piles.

Abutment piles:
____ piles ____ feet [meters] long, order length

Pier piles:
____ piles ____ feet [meters] long, order length

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Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP 250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live loads required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

606.2 FRICTION TYPE PILES

The following notes, modified to fit the specific conditions for the foundation required, will apply in all cases except where the piles are to be driven to bedrock. Provide the actual calculated Ultimate Bearing Value as shown below:

[30] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is _#_ tons [kN] per pile for the ____ abutment piles. The Ultimate Bearing Value is _#__ tons [kN] per pile for the ____pier piles.

Abutment piles:
____ piles ____ feet [meter] long, order length
____ Dynamic load testing items

Pier piles:
____ piles ____ feet [meter] long, order length
____ Dynamic load testing items

Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP 250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live load required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour depth is significant.

[30a] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is __#__ tons [kN] per pile for the _____ abutment piles. The Ultimate Bearing Value is #_ tons [kN] per pile for the pier piles. The pier piles include an additional __#__ tons [kN] per pile of Ultimate Bearing Value due to the possibility of losing frictional resistance due to scour.

Abutment piles:
____ piles ____ feet [meter] long, order length
____ Dynamic load testing items

Pier piles:

____ piles ____ feet [meter] long, order length

____ Dynamic load testing items

\# Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live loads required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

The following note, modified to fit the conditions, will apply where piles are driven through new embankment and the embankment settlement is expected, by design, to cause calculated downdrag forces.

[30b] PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is __#__ tons [kN] per pile for the _____ abutment piles. The Ultimate Bearing Value is _#_ tons [kN] per pile for the ____ pier piles. The addition of _#_ tons [kN] of Ultimate Bearing Value per abutment pile and _#_ tons [kN] per pier pile is due to the possibility of down drag forces induced by embankment settlement.

Abutment piles:

____ piles ____ feet [meter] long, order length

____ Dynamic load testing items

Pier piles:

____ piles ____ feet [meter] long, order length

____ Dynamic load testing items

\# Designer shall specify the Ultimate Bearing Value, the pile size (i.e. HP 10 x 42 [HP250 x 62], 12 inch [300 mm] Diameter) and the order length. The Ultimate Bearing Value is two (2) times the design load, based on actual dead and live loads required for the pile. Ultimate Bearing Value is not the maximum capacity of the selected pile size.

Provide the following note when Static Load Testing is required according to Section 303.4.2.5. Modify the note as necessary to fit the specific condition.

[30c] STATIC LOAD TEST: Perform dynamic testing on the first two production piles to determine the required blow count for the specified Ultimate Bearing Value. Perform the static load test on either pile. <u>Do not over-drive the selected pile</u>. Drive the third and fourth production piles to 75% and 85% of the determined blow count, respectively and perform dynamic testing on them. The test piles and the reduced capacity piles shall not be battered. After installation of the first four production piles, cease all driving operations on piling represented by the static load testing for a minimum of 7 days. After the waiting period, perform pile restrikes on the four piles (two restrike test items). The Engineer will review the results of the pile restrikes and establish the driving criteria for the remaining piling represented by the testing. Submit all test results to the Office of Structural Engineering.

For subsequent static load tests, upon completion of a 10,000 ft [3000 m] increment of driven length, repeat the above procedure for the initial static load test. If necessary, the Engineer will revise the driving criteria for the remaining piling accordingly.

When performing the restrike, if the pile has not reached the blow count determined for the plan specified Ultimate Bearing Value, continue driving the pile until this capacity is achieved.

Provide the following note when battered piles are specified.

[30d] BATTERED PILES: The blow count for battered piles shall be the blow count determined for vertical piles of the same Ultimate Bearing Value divided an efficiency factor (D). Compute the efficiency factor (D) as follows:

$$D = \frac{1 - UG}{\sqrt{\left(1 + G^2\right)}}$$

U = Coefficient of friction, which is estimated at 0.05 for double-acting air operated or diesel hammers; 0.1 for single-acting air operated or diesel hammers; and 0.2 for drop hammers.

G = Rate of batter (1/3, 1/4, etc.)

606.3 STEEL PILE POINTS

Use the following note where steel points are required, and see Section 202.2.3.2.a.

[31] ITEM 507, STEEL POINTS, AS PER PLAN: Use steel pile points to protect the tips of the proposed steel "H" piling. Furnish steel points from the following manufactures/suppliers: Associated Pile and Fitting Corporation, 262 Rutherford Blvd., Clifton, New Jersey 07014, phone: (973)773-8400, (800)526-9047, fax: (973)773-8442; International Construction Equipment, Inc., 301 Warehouse Drive, Matthews, North Carolina 28015, phone: (704)821-8200, (888)423-8721, fax: (704)821-8201; Dougherty Foundation Products, Inc., P.O. Box 688, Franklin Lakes, New Jersey 07417, phone: (201)337-5748, fax: (201)337-9022; Versa Steel Inc., 1618 N.E. First Ave., Portland, Oregon 97232, phone: (503)287-9822, (800)678-0814, fax: (503)287-7483; Versabite Piling Accessories, 1704 Tower Industrial Dr., Monroe, North Carolina 28110, phone: (800)280-9950, (704)225-1566, fax: (704)225-1567; or by a manufacturer that can furnish a steel point that is acceptable to Director. The material used for the manufacturing of pile points shall conform to ASTM A27/A27M 65/35 [450/240] – Class 2 – Heat Treated or AASHTO M103/M103M 65/35 [450/240] – Heat Treated. Weld the pile points to the pile in accordance with AWS D1.5 or the manufacturer's written welding procedure supplied to the engineer before the welding is performed. Submit a notarized copy of the mill test report to the Engineer.

606.4 PILE SPLICES

Provide the following note when H-piles are specified.

[100] PILE SPLICES: In lieu of using the full penetration butt welds specified in CMS 507.09 to splice steel H-piles, the Contractor may use a manufactured H-pile splicer. Furnish splicers from the following manufacturer:

Associated Pile and Fitting Corporation

262 Rutherford Blvd.

Clifton, New Jersey 07014

Install and weld the splicer to the pile sections in accordance with the manufacturer's written assembly procedure supplied to the Engineer before the welding is performed.

606.5 MINIMUM HAMMER SIZE

[33] Note retired - see appendix

606.6 PILE ENCASEMENT

The following note shall be used where capped pile piers and steel "H" piles are being used for a bridge structure crossing a waterway. The exposed steel piling corrodes at the waterline, or near there. The note should not be used if the capped pile pier standard drawing is being used as standard drawing already specifies pile encasement methods.

[34] ITEM SPECIAL - PILE ENCASEMENT
Encase all steel H-piles for the capped pile piers in Class C concrete. Provide a concrete slump between 6 to 8 inches with the use of a superplasticizer. Place the concrete within a form that consists of polyethylene pipe (707.33), or PVC pipe (707.42). The encasement shall extend from 3 feet [1 meter] below the finished ground surface up to the concrete pier cap. Position pipe so that at least 3 inches [75 mm] of concrete cover is provided around the exterior of the pile.

In lieu of encasing the pile in concrete, galvanize the piles according to 711.02. The galvanizing shall be continuous from a minimum of 3 feet below the finish ground surface up to the concrete pier cap. The galvanized coating thickness shall be a minimum of 4 mils [100 µm]. Repair all gouges, scrapes, scratches or other surface imperfections caused by the handling or the driving of the pile to the satisfaction of the Engineer.

The Department will measure pile encasement by the number of feet. The Department will determine the sum as the length measured along the axis of each pile from the proper bearing. Furnish two shims per beam. The Department will measure this item by the total number supplied. The Department will pay for accepted quantities at the contract price for Item 516 - 1/8" [3 mm] Preformed Bearing Pads. Any unused shims will become the property of the State.

NOTE TO DESIGNER: The plan area of the shim pad shall be the same as the elastomeric bearing.

611.7 CLEANING STEEL IN PATCHES

Use this note with all concrete patching bid items that refer to the cleaning requirements specified in 519.04

[55a] ITEM 519 - PATCHING CONCRETE STRUCTURES, AS PER PLAN: Prior to the surface cleaning specified in 519.04 and within 24 hours of placing patching material, blast clean all surfaces to be patched including the exposed reinforcing steel. Acceptable methods include high-pressure water blasting with or without abrasives in the water, abrasive blasting with containment, or vacuum abrasive blasting.

611.8 CONVERSION OF STANDARD BRIDGE DRAWINGS

The Department's Standard Bridge Drawings are available in English units only. If the project scope has established that Contract Documents will be prepared in metric units, the Designer has two options:

A. Convert the English drawings to metric units. This means removing the standard title blocks and using the converted drawings as plan insert sheets directly in the project plans. In doing so, the Designer assumes responsibility for the accuracy to the converted drawings. CADD files may be downloaded from the ODOT, Office of Structural Engineering web site.

B. Specify the English Standard Bridge Drawings in the Plans and use the following note:

[55b] CONVERSION OF STANDARD BRIDGE DRAWINGS: The Standard Bridge Drawings referenced in this plan are in English units. Any conversion of dimensions required to construct the items shown on the standards is the responsibility of the Contractor. Refer to 109.02 for a listing of Conversion Factors. Conversions shall be appropriately precise and shall reflect standard industry metric values where suitable.

611.9 COFFERDAMS, CRIBS AND SHEETING

Use this note when the plans include detail designs for temporary shoring.

[101] ITEM 503, COFFERDAMS, CRIBS, AND SHEETING, AS PER PLAN:
The design shown on the plans for temporary support of excavation is one representative design that may be used to construct the project. The Contractor may construct the design shown on the plans or prepare an alternate design to support the sides of excavations. If constructing an alternate design for temporary support of excavation, prepare and provide plans in accordance with C&MS 501.05. The Department will pay for the temporary support of excavation at the contract lump sum price for Cofferdams, Cribs, and Sheeting. No additional payment will be made for providing an alternate design.

611.10 DECK PLACEMENT NOTES

611.10.1 FALSEWORK AND FORMS

Use the following note when web depths greater than 84 in. are specified.

[102] ITEM 511, CLASS HP CONCRETE, SUPERSTRUCTURE, AS PER PLAN *

Locate the lower contact point of the overhang falsework at least _**_ inches ± 2 in. above the top of the girder's bottom flange. The bracket contact point location requirements of C&MS 508 do not apply.

NOTE TO DESIGNER:

* Modify the pay item description to fit the specific project requirements.

** The minimum dimension for the location for the lower point of contact should be 76 in. below the bottom of the top flange. Designers should verify the acceptability of the design within the range of tolerance specified.

611.10.2 DECK PLACEMENT DESIGN ASSUMPTIONS

Use the following note on all projects requiring mechanized finishing machines to place deck concrete.

[103] DECK PLACEMENT DESIGN ASSUMPTIONS:

The following assumptions of construction means and methods were made for the analysis and design of the superstructure. The Contractor is responsible for the design of the falsework support system within these parameters and will assume responsibility for superstructure analysis for deviation from these design assumptions.

An eight wheel finishing machine with a maximum wheel load of _____ kips for a total machine load of _____ kips.

A minimum out-to-out wheel spacing at each end of the machine of 103".

A maximum spacing of overhang falsework brackets of 48 in.

A maximum distance from the centerline of the fascia girder to the face of the safety handrail of 65”.

NOTE TO DESIGNER:
Refer to BDM Section 302.2.7.2.c for design information regarding finishing machine loads.

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702.13 PAINTING OF A588/A709 GRADE 50 STEEL

[78] Note retired - see appendix

Provide the following note for bridge superstructures using unpainted A588/A709 Grade 50W steel and having deck expansion joints at the abutments. Modify the note accordingly for structures with intermediate expansion joints. Bridges with an integral or semi-integral type abutment will not require painting of the beam ends.

[79] PARTIAL PAINTING OF A709 GRADE 50W STEEL : Paint the last 10 ft [3 m] of each beam/girder end adjacent to the abutments including all cross frames and other steel within these limits. The prime coat shall be 708.01. The top coat color shall closely approach Federal Standard No. 595B - 20045 or 20059 (the color of weathering steel).

702.14 ERECTION BOLTS

Where erection bolts are specified for attaching crossframes on steel girder or rolled beam bridges, and the expected dead load differential deflection at each end of the crossframes is less than or equal to ½" [13 mm] provide the following note. (Do not use the note if standard drawing GSD-1-96 is being referenced.)

[80] ERECTION BOLTS: The hole diameter in the cross frames and girder stiffeners shall be 3/16" [4 mm] larger than the diameter of the erection bolts. Erection bolts shall be high strength bolts and shall remain in place. Supply two hardened washers with each high strength bolt. Fully torque the bolts or use a lock washer in addition to the two hardened washers. Furnish erection bolts as part of Item 513.

[81] Note Retired – See Appendix

702.15 WELDED ATTACHMENTS

Provide the following note on plans for steel beam or girder bridges:

[82] WELD ATTACHMENT of supports for concrete deck finishing machine to areas of the fascia stringer flanges designated "Compression". Do not weld attachments to areas designated "Tension". Fillet welds to compression flanges shall be at least 1" [25 mm] from edge of flange, be no more than 2" [50 mm] long, and be at least 1/4" [6 mm] for thicknesses up to 3/4" [19 mm] or 5/16" [8 mm] for greater than 3/4" [19 mm] thick.

702.16 DECK ELEVATION TABLES

702.16.1 SCREED ELEVATION TABLES

Screed elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks. Screed elevations are not required for slab bridges. The general criteria for screed elevation tables are defined in BDM Section 302.2.3. Refer to Figure 703 for an example screed table for structural steel members and Figure 704 for an example screed table for composite box beams. In lieu of a table format, the designer may supply screed elevations through the use of a deck plan view showing elevations and stations of the points required in BDM Section 302.2.3.

In addition to the screed elevation table or diagram, provide a screed elevation note similar to the one below to define the elevations that are given. The screed elevation locations should be identified on the transverse section.

[104] SCREED ELEVATIONS shown represent the theoretical deck surface location prior to deflections caused by deck placement and other anticipated dead loads.

702.16.2 TOP OF HAUNCH ELEVATION TABLES

Top of haunch elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam and other superstructure types requiring deck falsework. Top of haunch elevations are not required for slab bridges. The general criteria for top of haunch elevation tables are defined in BDM Section 302.2.3.

In addition to the top of haunch elevation table, provide a top of haunch elevation note similar to the one below to define the elevations that are given. The top of haunch elevation locations should be identified on the transverse section.

[105] TOP OF HAUNCH ELEVATIONS shown represent the theoretical location of the bottom of the deck above the beam/girder haunch prior to deflections caused by deck placement and other anticipated dead loads.

702.16.3 FINAL DECK SURFACE ELEVATION TABLES

Final deck surface elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks including slab bridges. The general criteria for final deck surface elevation tables are defined in BDM Section 302.2.3.

In addition to the final deck surface elevation table, provide a final deck surface elevation note similar to the one below to define the elevations that are given.

[106] FINAL DECK SURFACE ELEVATIONS shown represent the deck surface location after all anticipated dead load deflections have occurred.

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STRUCTURAL STEEL SCREED TABLE								
	SPAN NUMBER (1)							
	Station	Station	Station	Station	Station	Station	Station	Station
Point (2)	Bearing Pt	1/4 Pt	Pt (3)	Mid Span	Pt (3)	Splice Pt	1/4 Pt	Bearing Pt
Left Curb Line								
Phased Const Line								
Centerline								
Right Curb Line								

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 30 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.

Figure 703

COMPOSITE BOX BEAM SCREED TABLE							
	SPAN NUMBER (1)						
Point (2)	Station Bearing Pt	Station 1/4 Pt	Station Pt (3)	Station Mid Span	Station Pt (3)	Station 1/4 Pt	Station Bearing Pt
Left Curb line							
Phased Const Line							
Centerline							
Right Curb Line							

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points and/or mid span points exceeds 30 feet. If the distance does exceed 30 feet, locate the additional point midway between standard points.

Figure 704

STRUCTURAL STEEL SCREED TABLE								
	SPAN NUMBER (1)							
Point (2)	Station Bearing Pt	Station 1/4 Pt	Station Pt (3)	Station Mid Span	Station Pt (3)	Station Splice Pt	Station 1/4 Pt	Station Bearing Pt
Left Curb Line								
Phased Const Line								
Centerline								
Right Curb Line								

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points, mid span and/or splice points exceeds 10 meters. If the distance does exceed 10 meters, locate the additional point midway between standard points.

Figure 703M

COMPOSITE BOX BEAM SCREED TABLE							
	SPAN NUMBER (1)						
Point (2)	Station Bearing Pt	Station 1/4 Pt	Station Pt (3)	Station Mid Span	Station Pt (3)	Station 1/4 Pt	Station Bearing Pt
Left Curb line							
Phased Const Line							
Centerline							
Right Curb Line							

(1) Detail all spans.

(2) Station all Points for screeds.

(3) Additional points required in a span if the distance between bearing points, 1/4 points and/or mid span points exceeds 10 meters. If the distance does exceed 10 meters, locate the additional point midway between standard points.

Figure 704M

SECTION 900 – BRIDGE LOAD RATING

901 INTRODUCTION

For the purpose of this section the following definitions shall be used:

Bridge: All structures under the pavement of a roadway that have a total length, measured along the centerline of the roadway, of 10 ft. [3.048 m] or more.

Buried bridges: All structures including flat slabs, arch structures, frames, box sections, etc., that have a fill or pavement material of 2'-0" [600 mm] or more on top of the structures.

Long span bridge: All structures with at least one span greater than 200 ft. [61 m].

Non-buried bridges: All structures including flat slabs, arch structures, frames, box sections, etc., that have a fill or pavement material of less than 2'-0" [600 mm] on top of the structures.

OSE: ODOT Office of Structural Engineering

Pavement of a roadway: The pavement of a roadway includes all the paved or unpaved portions of a roadway including graded shoulders that may support vehicular traffic.

For load rating of buried bridges, see Section 904.

For load rating of non-buried bridges, see Section 905.

For analysis for special or Superload see Section 908.

For load rating of long span bridges, see Section 911.

902 LOADS TO BE USED FOR LOAD RATING

The analysis shall be performed for AASHTO HS20-44 [MS 18] (truck & lane) loading for both inventory and operating levels, and for four Ohio Legal Loads (2F1, 3F1, 4F1, and 5C1) at operating level, according to Figure 901.

All trucks used for analysis shall have transverse spacing, between centerline of wheels or wheel groups, of 6 ft. [1.830 m].

903 UNIT WEIGHTS & DENSITIES

The following assumptions should be made while performing the load rating analysis, unless more accurate site information is available:

A. Unit weight of soil .... 120 lb/ft^3 [18.9 kN/m^3]
B. Unit weight of asphalt.. 145 lb/ft^3 [22.8 kN/m^3]

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C. Unit weight of concrete .... 150 lb/ft^3 [23.6 kN/m^3]
D. Unit weight of latex modified concrete .... 150 lb/ft^3 [23.6 kN/m^3]
E. Water density .... 62.4 lb/ft^3 [9.8 kN/m^3]

904 LOAD RATING OF BURIED STRUCTURES

904.1 GENERAL

A. All bridges that have the pavement or the fill material of 2’-0” [600 mm] or more on top of the structure shall be load rated according to the provisions of this Section.

B. Complete structure shall be load rated for the Loads as per Section 902.

C. Exception List: Following types of buried structures shall not be load rated under the provisions of this Section.

1. Circular metal pipes
2. Circular plastic pipes
3. Circular concrete pipes
4. Elliptical concrete pipes
5. Buried metal boxes
6. Buried metal frames
7. Junction chambers
8. Manholes
9. Inlets

904.2 LOAD RATING OF NEW BURIED BRIDGES

904.2.1 CAST-IN-PLACE BOX & FRAME STRUCTURES

A. Cast-in-place bridges shall be load rated by the designer of the bridge.
B. BRASS-Culvert program shall be used to load rate the structure. For the BRASS-Culvert Analysis, see Section 910.

904.2.2 PRECAST BOXES

904.2.2.1 PRECAST BOXES OF SPAN GREATER THAN 12’ [3.6 m]

A. The load rating analysis will be performed by the OSE.
B. BRASS-Culvert program shall be used to load rate the structure. For the BRASS-Culvert Analysis, see Section 910.

disk, CD-ROM or separate from the report as an attachment to an E-mail message.

The report must list final inventory and operating ratings of each main bridge member, overall ratings of each structure unit (mainline, ramps, etc.), and the final ratings of the entire bridge summarized in a tabular form. The ratings of each member and the overall ratings of the structure shall be presented for each live load vehicle according to Figure 901.

An example of a Load Rating Report Summary is given as Figure 908.

The inventory and operating ratings for the AASHTO HS20-44 loading shall be expressed in terms of the AASHTO HS20-44 loading (English Units), rounded off to the nearest single decimal point. The operating ratings for each of the Ohio Legal Loads shall be expressed in terms of the gross tonnage of the respective legal load. The summary rating for all of the Ohio Legal Loads shall be the smallest rating factor of the four vehicles expressed as a percentage (i.e. multiplied by 100).

For existing bridges, the report shall state how the material properties were determined. Any specific details about the current conditions and bridge geometry shall be listed.

All calculations related to the load rating should be a part of the load rating report.

910.4 BRASS COMPUTER INPUT AND OUTPUT FILES

Submit electronic copies of the input & output files with extensions "dat," "cus" and "xml."

In addition to the electronic input data file, each copy of the rating report shall also include hard (printed) copies of the computer input and output files.

911 LOAD RATING LONG SPAN BRIDGES

This section applies to long span bridges as defined in BDM Section 901.

911.1 WHEN THE LOAD RATING SHALL BE DONE

Perform the load rating of long span bridges according to BDM Sections 905.4.2, 905.5.2, or 905.6.2.

911.2 HOW THE LOAD RATING SHALL BE DONE

911.2.1 INVENTORY & OPERATING LEVEL RATING USING HS20 LOADING

The provisions of BDM Section 905 shall apply.

911.2.2 OPERATING LEVEL RATING USING OHIO LEGAL LOADS

The provisions of BDM Section 905 shall apply except the live load application shall be in accordance with BDM Section 911.2.2.1, 911.2.2.2, or 911.2.2.3.

911.2.2.1 BRIDGES WITH THREE OR MORE LANES

A. If no permit vehicle is present, apply the following live load:
   1. In the right-most lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.
   2. In all other lanes with traffic in the same direction as 911.2.2.1.A.1, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.
   3. For bridges with two-way traffic, apply the live load for the opposing traffic according to 911.2.2.1.A.1 and 911.2.2.1.A.2.

B. If a permit load vehicle is present, apply the following live load:
   1. In the right-most lane, place one permit load vehicle positioned to produce the maximum live load effect on the component to be rated. In the adjacent lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.
   2. In all other lanes with traffic in the same direction as 911.2.2.1.B.1, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.
   3. For bridges with two-way traffic, place a series of Ohio 5C1 vehicles in the right-most lane. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used. In all remaining lanes, simultaneously place single 5C1 vehicles. These vehicles shall be positioned to produce the maximum live load effect on the component to be rated. Apply the multiple presence factors, AASHTO Standard Specification for Highway Bridges - Section 3.12, accordingly.

911.2.2.2 BRIDGES WITH TWO LANES

A. If no permit vehicle is present, apply the following live load:
   1. In the right-most lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be

spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

2. For bridges with one-way traffic, in the other lane simultaneously place a single 5C1 vehicle positioned to produce the maximum live load effect on the component to be rated.
3. For bridges with two-way traffic, in the other lane place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

B. If a permit load vehicle is present, apply the following live load:
   1. In the right-most lane, place one permit load vehicle positioned to produce the maximum live load effect on the component to be rated.
   2. In the other lane, place a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

911.2.2.3 BRIDGES WITH A SINGLE LANE

A. If no permit vehicle is present, the live load shall be a series of Ohio 5C1 vehicles. The 5C1 vehicles should be spaced such that the distance between the rear axle of the leading vehicle and the front axle of trailing vehicle shall be 36 ft. Place as many 5C1 vehicles as necessary to produce the maximum load effect on the component to be rated. No partial 5C1 vehicles shall be used.

B. If a permit vehicle is present, the live load shall be the one permit vehicle positioned to produce the maximum live load effect on the component to be rated.

912 REFERENCES

A. AASHTO, 1978, “Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members,” and all subsequent Interims.

B. AASHTO, 1983, “Manual for Maintenance Inspection of Bridges.”

C. AASHTO, 1989, “Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges.”

D. AASHTO, 1990, “Guide Specifications for Fatigue Evaluation of Existing Steel Bridges,” and all subsequent Interims.

E. AASHTO, 2000, “Manual for Condition Evaluation of Bridges,” and all subsequent Interims.

F. BRASS-Culvert software developed by the Wyoming Department of Transportation (PO Box 1708, Cheyenne, WY 82003).

G. Duncan, J.M., 1979, “Design Studies For Aluminum Structural Plate Box Culverts,” Kaiser Aluminum and Chemical Sales, Inc.

H. NCSPA, “Load Rating & Structural Evaluation of In-Service Corrugated Steel Structures,” & Design Data Sheet No. 19, National Corrugated Steel Pipe Association (NCSPA, 202-452-1700).

I. SRG, Structure Rating Group Website http://www.dot.state.oh.us/srg/