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

July 17, 2009

To: Users of the Bridge Design Manual

From: Tim Keller, Administrator, Office of Structural Engineering

By: Sean Meddles, Bridge Standards Engineer

Re: 2009 Third Quarter Revisions

Revisions have been made to the ODOT Bridge Design Manual, July 2007. These revisions shall be implemented on all Department projects with a Stage 2 plan submission date after July 17, 2009.

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 July 2007 edition of the Bridge Design Manual may be downloaded at no cost using the following link:

http://www.dot.state.oh.us/Divisions/HighwayOps/Structures/standard/Pages/default.aspx

Attached is a brief description of each revision.

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Summary of Revisions to the July 2007 ODOT BDM

BDM Section	Affected Pages	Revision Description
202.2.1	2-9	The determination of span length for slab bridges has been revised to reflect changes to the ODOT slab standard as defined in CPA-1-08 and CS-1-08.
202.2.3.2.b	2-15	This revision reflects a change in the design of capped pile piers as detailed on CPP-1-08. CPP-1-08 is designed for a maximum unsupported pile length of 20-ft. The “exposed” portion of pile may make up only a portion of the total unsupported pile length.
202.2.3.2.h	2-17	This revision clarifies the “design” flood used in LRFD 2.6.4.4.2 scour considerations versus the design year flood requirements specified in the BDM.203.2.
204.5	2-26	This revision reflects the design parameters in CPP-1-08.
204.6.2.1	2-28	This revision eliminates a discrepancy between the definition of MSE wall height (H) as specified in SS840 and the BDM.
302.2.7.2.c	3-17.7	This revision corrects an error in the Load Data for Dead Load of Concrete.
302.3	3-18	The reduction in spacing of final deck surface elevations from 30-ft to 25-ft is consistent with the July 2008 revisions for Deck Elevation Requirements in BDM Section 302.2.3.
302.4.1.8	3-23	The reduction in spacing of camber and deflection locations from 30-ft to 25-ft is consistent with the July 2008 revisions for Deck Elevation Requirements in BDM Section 302.2.3.
303.2.2.2	3-51	This revision reflects the change in the capped pile abutment design as shown in CPA-1-08.
303.3.1.1	3-57	This revision reflects the change in the capped pile pier design as shown in CPP-1-08.
303.3.2.5	3-59 through 3-60	These revisions reflect the change in the capped pile pier design as shown in CPP-1-08.
306.3.2	3-86	This revision reflects the change in the capped pile pier and capped pile abutment designs as shown in CPP-1-08 & CPA-1-08 respectively.
Figure 303.5.1-2		This figure was revised to reflect a revision to SS840. Specifically, the required 3-ft layer of Item 304 material at the base of the MSE wall fill has been eliminated.
Figure 303.5.1-3		This figure was revised to reflect a revision to SS840. Specifically, the required 3-ft layer of Item 304 material at the base of the MSE wall fill has been eliminated.
Figure 303.5.1-5		This figure was revised to reflect a revision to SS840. Specifically, the required 3-ft layer of Item 304 material at the base of the MSE wall fill has been eliminated.

BDM Section	Affected Pages	Revision Description
Figure 303.5.1-6		This figure was revised to reflect a revision to SS840. Specifically, the required 3-ft layer of Item 304 material at the base of the MSE wall fill has been eliminated.
Figure 303.5.1-7		This figure was revised to reflect a revision to SS840. Specifically, the required 3-ft layer of Item 304 material at the base of the MSE wall fill has been eliminated.
802.1.2	8-2	The N-value correction table has been revised to reflect the ODOT Specifications for Geotechnical Explorations sampling requirements and the AASHTO LRFD Bridge Design Specification Article 10.4.6.2.4 correction factors.
Figure 801-1		A new metal wall supplier, Centria, has been added to the approved list. Also, the company name for the acrylic wall supplier, Cyro Industries, has changed to Evonik Cyro LLC.

vertical clearance; treatment of slopes around the ends and under the bridge; channel changes; soil boring locations; centerline of temporary structure and temporary approach pavement; stationing of bridge limits (i.e. the bridge ends of approach slabs); limits of channel excavation shown by crosshatching with a description provided in a legend; the location and description of bench marks; the following earthwork note: “EARTHWORK limits shown are approximate. Actual slopes shall conform to plan cross sections.”; and guardrail stationing.

When providing guardrail stationing: for bridges with twin steel tube bridge railing, station the first top mounted post on the bridge, for bridges with deep beam railing and concrete barrier railing, station the first guardrail post off the bridge. Typically stationing should be given to the nearest 1/100th of a foot [mm]. Guardrail stationing may be changed during the detail design phase and then revised on the Site Plan.

In addition to the requirements of Section 201.2.2, the Final Structure Site Plan should show the following information in the profile view: profile gradient percent; embankment slopes and top of slope elevations; proposed footing elevations; type of foundations; top of bedrock elevations at each boring location; and shaded areas of the bridge that represent the new bridge components.

For geometrical clarification: spans on straight (tangent) alignments should be measured from center to center of bearings along the centerline of survey; spans on curved alignments should be measured along a reference line, a chord drawn from centerline to centerline of abutment bearings at the centerline of survey or extended tangent along the centerline of survey; spans for concrete slab bridges are measured from a line 1’-0” behind the face of abutment substructure or breastwall; skews should be given with respect to the centerline for straight alignments or to reference lines (chords or tangents) for curved alignments; for straight alignments, the bearing of the centerline of survey should be shown; for curved alignments, the bearing of the reference line (chord or tangent) should be shown; and a superelevation transition table or diagram similar to Figure 209.1.1-1, should be provided if the bridge crown (superelevation) changes across the structure, reference should be made to the table or diagram when detailing the typical bridge transverse section.

Descriptive data for the proposed structure should be shown in a “Proposed Structure” block. The “Proposed Structure” block should be placed in the lower right hand corner for the 22” x 34” [559 x 864 mm] sheet size. An “Existing Structure” block should be shown on the Site Plan if applicable and be placed above the “Proposed Structure” block. Structure blocks should be approximately 6½” [165 mm] wide for 22” x 34” [559 x 864 mm] sheet size.

202.2.2 FINAL MAINTENANCE OF TRAFFIC PLAN

In addition to the Preliminary Maintenance of Traffic Plan requirements of Section 201.2.7, the Final Maintenance of Traffic Plan should include the following information:

A. Plan views and preliminary working drawings to ensure constructability

B. Temporary barrier anchorage details and requirements

C. Location and type of temporary shoring (See Section 208)

D. Location of structural members that require strengthening

E. Temporary structure design information (See Section 500)

F. Additional notes and/or details necessary

For concrete slabs, early standard drawings called for the main reinforcement to be placed perpendicular to the abutments when the skew angle became larger than a certain value. This angle has been revised over the years as new standard drawings were introduced. When considering staged construction requirements and the orientation of the cutline, screen existing concrete single span slab bridges according to the following criteria:

Prior to 1931 the slab bridge standard drawing required the main reinforcement to be placed perpendicular to the abutments when the skew angle was equal to or greater than 20 degrees. This angle was revised to 25 degrees in 1931, 30 degrees in 1933 and finally 35 degrees in 1946. The standard drawing in 1973 required the main reinforcement to be parallel with the centerline of roadway regardless of skew angle. Existing exposed reinforcing steel may be used to confirm the direction of the reinforcing steel.

If the skew angle of the bridge is equal to or greater than the angles listed above for the year built, a temporary longitudinal bent will have to be designed to support the slab where it is cut or if possible locate the cutline parallel to the reinforcing if sufficient room exists. For example a bridge built in 1938 with a 25 degree skew does not require a bent, however a bridge built in 1928 with a 25 degree skew does require a bent to be designed.

When utilizing semi-integral construction, the stability of the new part-width superstructure is to be considered. There exists the potential of the superstructure to move laterally either from the effects of the traffic using the new deck or the lateral earth pressure against the approach slab. See Standard Bridge Drawing “SEMI-INTEGRAL CONSTRUCTION DETAILS” for more information.

202.2.3 FOUNDATION REPORT

The Bridge Preliminary Design Report shall include a Foundation Report in accordance with the ODOT Specifications for Geotechnical Explorations. The Foundation Report shall include:

A. Investigational Findings

B. Analyses and Recommendations

C. Boring Logs

D. Test Data

Where the scour evaluation has identified a potential problem, the probable scour depths, 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 unsupported 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. Refer to BDM Section 204.5 for additional information.

202.2.3.2.c DOWNDRAG 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 downdrag loads on the piles should be considered. The potential downdrag load should be computed according to *LRFD 3.11.8*. Use either the traditional method or neutral plane method for calculating downdrag. The traditional method is described in *LRFD 3.11.8* and in "Design and Construction of Driven Pile Foundations" (1996), FHWA-HI-97-013. The neutral plane method is discussed in NCHRP Report 393 (Briaud & Tucker, 1997). Both methods are described in "Design and Construction of Driven Pile Foundations" (2006), FHWA-NHI-05-042.

If using the traditional method to calculate the downdrag load, consider the transient loads concurrent with the downdrag load and permanent load. If using the neutral plane method, use the permanent loads plus the downdrag load or the permanent loads plus the transient loads, whichever is greater. For the neutral plane method, do not consider the transient loads concurrent with the downdrag load.

For piles driven to refusal on bedrock, include the factored downdrag load in the total factored load provided in the structure general notes.

For friction piles, the geotechnical resistance required to support the structure loads plus downdrag is only provided by the soil below the lowest soil layer contributing to downdrag. However, during pile installation, the soil within the downdrag zone provides positive side resistance. With time, the soil settles and this positive side resistance reverses direction and causes the downdrag load. Therefore, the Ultimate Bearing Value must include the downdrag loads and the positive side resistance within the downdrag zone. Determine the Ultimate Bearing Value (R_{ndr}) using the procedure shown below (also see *LRFD C10.7.3.7* for more explanation). A plan note is available in BDM Section 600.

$$R_{ndr} = \frac{\sum \eta_i \gamma_i Q_i}{\phi_{DYN}} + \frac{\eta_i \gamma_p DD}{\phi_{DYN}} + R_{Sdd}$$

Where:

R_{ndr} = Ultimate Bearing Value (kips)

$\sum \eta_i \gamma_i Q_i$ = Total factored load per pile, not including downdrag load (kips)

$\mathbf{N}_{DYN}$ = Resistance factor for driven piles. (See BDM Section 202.2.3.2.b.)

$\eta_i \gamma_p DD$ = Factored downdrag load per pile (kips)

R_{Sdd} = Additional amount of Ultimate Bearing Value to account for side friction that must be overcome during driving through the downdrag zone. This value is equal to the nominal (i.e. **unfactored**) side resistance for the soil in the downdrag zone and shall be calculated using static analysis methods, *LRFD 10.7.3.8.6.* (kips)

If using the traditional method to calculate the downdrag load, include permanent and transient loads in the total factored load given in the equation above. If using the neutral plane method, include only the permanent loads in the total factored load given in the equation above. For the neutral plane method, also ensure that the Ultimate Bearing Value is greater than the factored permanent and transient loads (do not include downdrag) divided by $\mathbf{N}_{DYN}$ ($R_{ndr} \geq (\Sigma \eta_i \gamma_i Q_i) / \mathbf{N}_{DYN}$).

Determine the estimated length for friction piles using static analysis methods to calculate the length of pile necessary to develop the required Ultimate Bearing Value as described in BDM Section 202.2.3.2.b.

To resist downdrag forces, consider using larger H-piles or increasing the number of piles and reducing the applied load per pile. To reduce or eliminate downdrag loads, consider preloading the soil so settlement occurs before pile installation. Also consider installing wick drains to decrease the amount of time required for settlement to occur.

202.2.3.2.d PILE WALL THICKNESS

Minimum pipe pile wall thicknesses are specified by a formula in CMS 507.

202.2.3.2.e PILE HAMMER SIZE

According to Item 507, the contractor will select a pile hammer large enough to achieve the specified Ultimate Bearing Value and perform a dynamic load test to verify that the Ultimate Bearing Value is achieved. Refer to Section 303.4.2 for specific pile testing requirements.

202.2.3.2.f CONSTRUCTION CONSTRAINTS

For construction constraints regarding pile installation and embankment construction, see Section 600 of this Manual.

202.2.3.2.g PREBORED HOLES

The Designer shall specify prebored holes, CMS 507.11, for each pile on a project to be driven through 15 ft [4.5 m] or more of new embankment. Clearly indicate the locations and lengths of all prebored holes in the plans. For design purposes, ignore the effect of skin friction along the length of the prebored holes. The length shall be the height of the new embankment at each pile location.

202.2.3.2.h SCOUR CONSIDERATIONS

Where the scour evaluation has identified a potential problem, any pile resistance provided by soil in the scour zone shall be neglected. The depth of scour resulting from the design flood, per *LRFD 2.6.4.4.2*, shall be considered at the Strength and Service Limit States. The depth of scour resulting from the check flood shall be considered at the Extreme Event II Limit State.

For friction piles, the soil within the scour zone provides side resistance during pile installation. Therefore, the Ultimate Bearing Value must also include the side resistance from the soil within the scour zone. Determine this larger Ultimate Bearing Value using the procedure shown below. A plan note is available in BDM Section 600.

$$R_{ndr} = \frac{\sum \eta_i \gamma_i Q_i}{\phi_{DYN}} + R_{Ssc}$$

Where:

R_{ndr} = Ultimate Bearing Value (Kips)

$\sum \eta_i \gamma_i Q_i$ = Total factored load for highest loaded pile at each substructure unit (kips)

N$_{DYN}$ = Resistance factor for driven piles. (See BDM Section 202.2.3.2.b)

R_{Ssc} = Additional amount of Ultimate Bearing Value to account for side friction that must be overcome during driving through the scour zone. This value is equal to the nominal (i.e. **<u>unfactored</u>**) side resistance for the soil in the scour zone and shall be calculated using static analysis methods, *LRFD 10.7.3.8.6.* (kips)

Determine the estimated length for friction piles using static analysis methods to calculate the length of pile necessary to develop the larger Ultimate Bearing Value as described in BDM Section 202.2.3.2.b.

Because the pile will lose support along the scour depth, the Designer should investigate the structural capacity of the pile considering the depth of the scour as an unbraced length. The maximum factored structural resistances listed in BDM Section 202.2.3.2.a do not apply.

202.2.3.2.i UPLIFT RESISTANCE OF PILES

When a pile must resist uplift loads, the uplift resistance shall be calculated in accordance with *LRFD 10.7.3.10.* Use static analysis methods (*LRFD 10.7.3.8.6*) to determine the nominal uplift resistance due to side resistance.

Where the estimated pile length is controlled by the required uplift resistance, specify a minimum penetration pile tip elevation. A plan note is available in BDM Section 600.

The Ultimate Bearing Value is not shown on the plans for piles driven to a tip elevation, so the plans must specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Perform a drivability analysis to estimate the nominal driving resistance at the required tip elevation. Calculate the minimum pile wall thickness using the formula in CMS 507.06, with the Ultimate Bearing Value equal to the nominal driving resistance.

202.2.3.3 DRILLED SHAFTS

Drilled shafts should be considered when their use would:

A. Prevent the need of cofferdams

B. Become economically viable due to high design loads (eliminates the need of large quantities of pile)

C. Provide protection against scour

D. Provide resistance against lateral and uplift loads

E. Accommodate sites where the depth to bedrock is too short for adequate pile embedment but too deep for spread footings

F. Accommodate the site concerns associated with pile driving process (vibrations, interference due to battered piles, etc.).

Drilled shafts shall be designed in accordance with *LRFD 10.8* and constructed in accordance with CMS 524.

Drilled shafts that support pier columns shall be 6 in. [150 mm] larger in diameter than the pier column diameter. The minimum diameter for drilled shafts that support pier columns shall be 42 in. [1065 mm]. The minimum diameter for all other drilled shafts shall be 36 in. [915 mm]. Drilled shaft diameters of less than 36 in. [915 mm] are not recommended.

Underreams or belled shafts should not be used. Belled shafts are difficult to construct under water or slurry and the bell will collapse in non-cohesive soils. Cleaning and inspecting the base of the drilled shaft within the bell are also very difficult.

Drilled shaft diameters shall be shown on the Final Structure Site Plan. For drilled shafts with friction type design, the tip elevation shall also be shown. For drilled shafts supported on bedrock, the tip elevation should not be given. Instead, the approximate top of the bedrock elevation and the length of the bedrock socket shall be shown in the profile view on the Final Structure Site Plan.

Designers should neglect the contribution to skin friction provided by the top 2 ft. [610 mm] of the rock socket.

The Foundation Report shall include the following drilled shaft information:

aesthetically pleasing structure.

The spill-thru slope should intersect the face of abutment a minimum of one foot [300 mm], or as specified in a standard bridge drawing, below the bridge seat for stringer type bridges. For concrete slab and prestressed box beam bridges this distance should be 1’-6” [450 mm].

204.3 ABUTMENT TYPES

Preference should be given to the use of spill-thru type abutments. Generally for stub abutments on piling or drilled shafts the shortest distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4’-0” [1200 mm]. For stub abutments on spread footing on soil, the minimum dimension shall be 5’-0” [1525 mm]. For any type of abutment, integral design shall be used where possible, see Section 205.8 for additional information.

Wall type abutments should be used only where site conditions dictate their use.

204.4 ABUTMENTS SUPPORTED ON MSE WALLS

When conditions are appropriate, the designer may consider stub type abutments with piling or spread footings supported on MSE walls. Use spread footings to support the abutment if the MSE wall is on bedrock or shale. If the MSE wall is on soil, then the selection of spread footings or piles to support the abutment should consider possible settlement of the MSE wall. Use piles to support the abutment if the bridge is a continuous multi-span structure, or if the bridge is constructed part width in phases. If the bridge is a single-span structure and is not constructed part width in phases, then either spread footings or piles may be used to support the abutment. Piles require a minimum 15-foot embedment below the MSE wall.

Refer to Sections 201.2.6, 202.2.3 and 204.6.2 for the staged review requirements for MSE walls. Consult the Office of Structural Engineering for additional design recommendations.

204.5 PIER TYPES

For highway grade separations, the pier type should generally be cap-and-column piers supported on a minimum of 3 columns. The purpose for this provision is to reduce the potential for total pier failure in the event of an impact involving a large vehicle or its cargo. This requirement may be waived for temporary conditions that require caps supported on less than 3 columns. Typically the pier cap ends should be cantilevered and have squared ends.

For bridges over railroads generally the pier type should be T-type, wall type or cap and column piers. Preference should be given to T-type piers. Where a cap and column pier is located within 25 feet [7.6 meters] from the centerline of tracks, crash walls will be required.

For waterway bridges the following pier type should be used:

A. Capped pile type piers; generally limited to an unsupported pile length of 20 feet [6 meters]. For unsupported pile lengths greater than 15 feet [4.5 meters], the designer should analyze the piles as columns above ground. Scour depths and the embedded depth to fixity of the driven piles shall be included in the determination of unsupported length.

B. Cap-and-column type piers.

C. Solid wall or T-type piers.

Note the use of T-type piers, or other pier types with large overhangs, makes the removal of debris at the pier face difficult to perform from the bridge deck. For low stream crossings with debris flow problems and where access to the piers from the stream is limited, T-type piers, or other similar pier types, should not be used.

For unusual conditions, other types may be acceptable. In the design of piers which are readily visible to the public, appearance should be given consideration if it does not add appreciably to the cost of the pier.

204.6 RETAINING WALLS

In conformance with Section 1400 of the ODOT Location and Design Manual, Volume Three, a Retaining Wall Justification shall be included in the Preferred Alternative Verification Review Submission for a Major Project or in the Minor Project Preliminary Engineering Study Review Submission. A description of the Retaining Wall Justification is provided in Section 1404 of the ODOT Location and Design Manual, Volume Three. Generally, the justification compares the practicality, constructability and economics of the various types of retailing walls listed below:

A. Cast-in-place reinforced concrete

B. Prestressed concrete

C. Tied-back

D. Adjacent drilled shafts

E. Sheet piling

F. H-piling with lagging

G. Cellular (Block, Bin or Crib)

H. Soil nail

I. Mechanically Stabilized Earth (MSE)

Refer to SS840 for accredited MSE wall systems. Contact the Office of Structural Engineering for modular block wall systems. For wall systems that utilize geogrid reinforcements, the wall height shall be limited to 30 ft.

204.6.1 DESIGN CONSTRAINTS

Below are some design constraints to consider in the wall justification study to establish acceptable wall types:

A. Future use of the site (future excavations cannot be made in Mechanically Stabilized Embankments)

B. Deflection and/or differential settlements

C. Accessibility to the construction site

D. Aesthetics, including wall textures

E. Right-of-way (or other physical constraints)

F. Cost (approximate cost analysis)

G. Stage construction

H. Stability (long-term and during construction)

I. Railroad policies

204.6.2 STAGE 1 DETAIL DESIGN SUBMISSION FOR RETAINING WALLS

When a justification study has determined that a retaining wall is required, generally the wall will be a cast-in-place reinforced concrete wall or some type of proprietary wall system. The use of proprietary wall systems should be considered when the wall quantity for the project exceeds 5000 ft^2 [450 m^2].

204.6.2.1 PROPRIETARY WALLS

If a proprietary wall is justified, the Design Agency shall include the following information in the Stage 1 Detailed Design Submission: wall alignment; footing elevations; factored bearing resistance at the leveling pad elevation; a global stability analysis; the effect of settlement and settlement calculations; and any construction constraints, such as soil improvement methods, that may be required. Refer to Section 303.5 for plan requirements for Detail Design.

The alignment of proprietary retaining walls should be straight and with as few corners or curves as is practical. When changes in wall alignment are required, use gradual curves or corners with an interior angle of at least 135 degrees whenever possible. Do not use corners with interior angles of less than 90 degrees (acute corners).

The design of the wall shall be in conformance with the 4th Edition of the *AASHTO LRFD Bridge Design Specifications* and the following:

A. Determine the height of the wall (H) as follows:

1. If the wall is not located at an abutment, measure (H) as the elevation difference between the top of the coping and the top of the leveling pad.
2. If the wall is located at an abutment, measure (H) as the elevation difference between the profile grade at the face of the wall and the top of the leveling pad.

B. The soil reinforcement length shall not be less than 70% of the wall height (H) or 8’-0” [2.5 m], whichever is greater. Only increase this minimum soil reinforcement length as necessary to meet external stability requirements (sliding, bearing resistance, overturning, overall global stability). Generally, the soil reinforcement length should not be greater than 150% of the wall height (H). Provide calculations with the Foundation Report, BDM Section 202.2.3, that justify soil reinforcement lengths exceeding 0.70H.

C. The thickness of the unreinforced concrete leveling pad shall not be less than 6 inches [150 mm]. The minimum distance from the top of the leveling pad the ground surface at a point located 4’-0” [1.2 m] from the face of the wall shall be the larger of 3’-0” [900 mm] or the frost depth. Refer to Figure 303.5.1-4 for more information.

D. The minimum thickness of the precast reinforced concrete face panels may be assumed to be 5½ inches [140 mm].

E. The maximum allowable differential settlement in the longitudinal direction (regardless of the size of panels) is one (1) percent. Provide slip joints if the estimated differential settlement is greater than one (1) percent.

F. Use the following soil parameters for design:

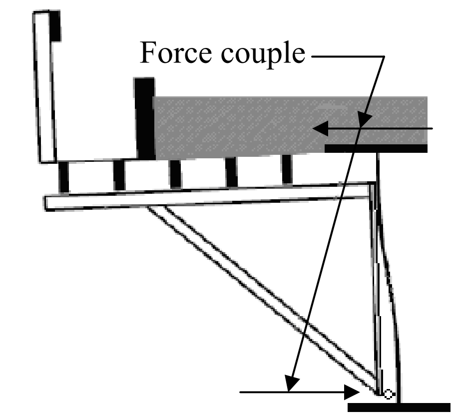
Fill Zone	Type of Soil	Soil Unit Weight	Friction Angle	Cohesion
Reinforced Zone	Select Granular Embankment Material	120 lb/ft^3 [18.9 kN/m^3]	34º	0
Retained Soil	On-site soil varying from sandy lean clay to silty sand	120 lb/ft^3 [18.9 kN/m^3]	30º	0

Determine soil parameters for the foundation soils based on the soils encountered by the soil borings.

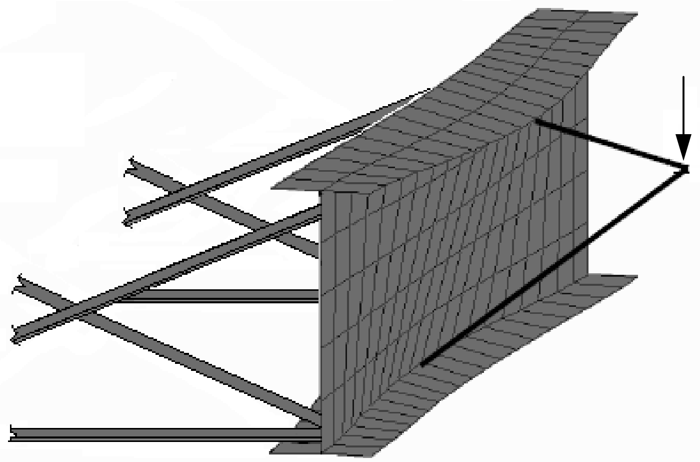
G. Compute the coefficient of lateral earth pressure, k_a, using the Coulomb equation.

H. MSE walls located within 25’-0” [7.6 m] of the centerline of tracks, or other distance as specified by an individual railroad, shall be protected by a crash wall as specified in Section 209.8 and the AREMA Manual for Railway Engineering. The MSE wall system does not meet the definition of a crash wall as defined by the AREMA Manual for Railway Engineering.

I. For MSE walls supporting abutments on spread footings, the minimum distance between the back face of the MSE wall panels and the toe of the bridge abutment footing shall be 3’-0” 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:

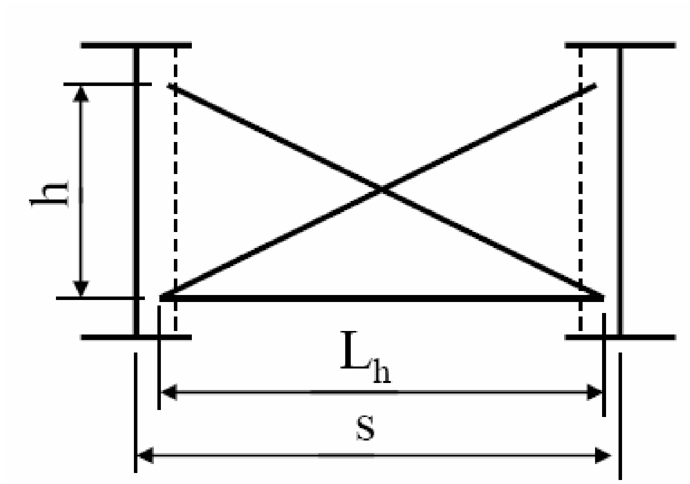
$$I_x = \frac{h^2 s}{4L_d^3\left(\frac{1}{A_d L_h^2} + \frac{L_h}{A_h L_d^3 + A_d L_h^3}\right)}$$

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.

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 sequence of construction.

302.3 CONTINUOUS OR SINGLE SPAN CONCRETE SLAB BRIDGES

Continuous reinforced concrete slab bridge design shall be in accordance with *LRFD 4.6.2.3*.

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 25'-0".

302.4 STRUCTURAL STEEL

302.4.1 GENERAL

Structural steel should be designed utilizing a composite section. Refer to *LRFD 6.10.10.1* and BDM Section 1006 for more information.

A non-composite design may be used only if the design is the most economical.

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 slab construction, etc. using laterally unsupported lengths that reflect actual bracing conditions.

302.4.1.1 MATERIAL REQUIREMENTS

Types of steel to be selected for use in the design and construction of bridges is as follows:

A. ASTM A709[M] grade 50W shall be specified for an un-coated weathering steel bridge.

B. ASTM A709[M] grade 50 shall be specified for a coated steel bridge.

C. ASTM A709[M] grade 36 is not recommended and is being discontinued by the steel mills.

D. High Performance Steel (HPS), A709[M] grade 70W, un-coated weathering steel is most economical when used in the flanges of hybrid girders. Consult the Office of Structural Engineering for recommendations prior to specifying its use. A plan note is provided in the appendix.

between points of 25'-0".

In cases of special geometry, i.e. spirals, horizontal or vertical curves, superelevation transitions, etc., additional points are to be detailed in the deflection and camber table if the normally required points do not adequately define a beam or girder required curvature.

The required shop camber shall in all cases be the algebraic sum of the computed deflections, vertical curve adjustment, horizontal curve adjustment and adjustment due to heat curving. Camber shall be measured to a chord between adjacent bearing points.

A camber diagram shall be provided showing the location of the points developed above and giving vertical offset dimensions at the bearing points from a “Base” or “Work” line between abutment bearings.

302.4.1.9 FATIGUE DETAIL CATEGORY

In order to allow for future rehabilitation involving welded attachments less than 2.0 in., the fatigue limit state loading for steel members in the negative moment regions should not exceed the nominal fatigue resistance for Detail Category C.

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 REQUIREMENTS 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

For galvanized structures, the bolt hole size requires a 1/16 inch [1.5 mm] increase over a standard hole size to allow for the additional thickness of the zinc coating. This increase in hole size decreases the splice capacity.

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 requirements of *LRFD 6.13.6.1.3* shall be met.

the concrete (1 to 3 percent per yard [meter] of the abutment, pier or wall)

D. Generic formliner patterns shall be specified. An alternative of at least three suppliers listed. Listing of a formliner pattern only available from one supplier will not be accepted.

303.2.2.1.a COUNTERFORTS FOR FULL HEIGHT ABUTMENTS

For full height abutments exceeding 30 feet [10 000 mm] in height, counterforts should be considered.

Reinforcing steel in the back, sloping, face of the counterfort should be placed in two rows with a 6 inch [150 mm] clearance between rows. Reinforcing steel splices should be staggered a minimum of 3'-0" [1000 mm], by row.

Reinforcing extending from the footing of a counterforted wall into the highly reinforced areas of the counterforts shall have reinforcing steel splices staggered.

In counterforted walls, each pocket formed by the intersection of the counterfort and wall shall be drained.

303.2.2.1.b SEALING STRIP FOR FULL HEIGHT ABUTMENTS

Use an impervious fabric across the expansion joints in full height abutments or retaining walls to eliminate leakage. The impervious fabric should be CMS 512 Type 2 Waterproofing, 3 feet [1000 mm] wide, centered over, and extending the full length of the joint to the top of the footing. See Section 303.2.5 on requirements for expansion joints in abutments.

303.2.2.2 CONCRETE SLAB BRIDGES ON RIGID ABUTMENTS

For a continuous concrete slab bridge supported on rigid abutments, eliminate the A801 bar shown in CPA-1-08; trowel smooth the joint between the deck slab and the top of the abutment; and recess a continuous strip of elastomeric material into the abutment seat before placement of the superstructure concrete. This bearing system should conform to temperature movement and bearing design requirements of this Manual.

303.2.2.3 STUB ABUTMENTS WITH SPILL THRU SLOPES

If a stub abutment is to support a bridge having provision for relative movement between the superstructure and the abutment, two rows of piles are required and the front row shall be battered 1:4.

Where two rows of piles are used, the forward row shall have approximately twice the number of piles as the rear row, with the rear piles placed directly behind alternate front piles.

The perpendicular distance from the surface of the embankment to the bottom of the toe of the footing should be at least 4'-0" [1200 mm].

The maximum spacing of piles in a single row or in the front row of a double row shall be 8 feet [2500 mm].

For phased construction projects, do not design an abutment phase supported on less than three (3) piles or two (2) drilled shafts.

303.2.2.4 CAPPED PILE STUB ABUTMENTS

For capped pile stub abutments that do not provide for relative movement between the superstructure and the abutment, one row of vertical piles shall be used.

The construction joint at the top of the footing for cap pile abutments should be shown as optional.

For phased construction projects, do not design an abutment phase supported on less than three (3) piles or two (2) drilled shafts.

To avoid non-redundant designs, capped pile abutments should be supported on at least 5 piles. Refer to *LRFD 10.5.5.2.3* for more information.

303.2.2.5 SPREAD FOOTING TYPE ABUTMENTS

Where foundation conditions warrant the use of an abutment on a spread footing, the bottom of the footing should be at least 5'-0" [1525 mm] below the surface of the embankment.

The perpendicular distance from the surface of the embankment to the bottom of the toe of the footing should be at least 5'-0" [1525 mm].

In no case shall the top of the footing be less than 1'-0" [300 mm] below the surface of the embankment.

303.2.2.6 INTEGRAL ABUTMENTS

Integral Abutment use is limited as defined in Section 200 of this Manual. Integral design should not be used with curved main members or main members that have bend points in any stringer line.

For an integral design to work properly, the geometry of the approach slab, the design of the wingwalls, (see section 303.2.4) and the transition parapets must be compatible with the freedom required for the integral (beams, deck, backwall, wingwalls and approach slab) connection to rotate and translate longitudinally.

on bedrock.

The minimum width of footing supported by a drilled shaft is the diameter of the shaft.

Where piling is used to support free-standing piers, the distance between centers of outside piles, measured across the footing, generally shall be not less than one-fifth the height of the pier.

Widths greater than the above shall be provided if required for proper bearing area or to accommodate the required number of piles.

The height of the pier shall be measured from the bottom of the footing to the bridge seat.

For multiple span bridges with continuity over piers, where the height of pier is more than 50 percent of the length of superstructure from the point of zero movement to such pier, it may be assumed that the pier will bend or tilt sufficiently to permit the superstructure to expand or contract without appreciable pier stress. This assumption is not permissible if the piers are skewed more than 30 degrees. The above rule does not apply to rigid frame or arch bridges.

Slender columns of either concrete or steel may be designed to bend sufficiently to permit the superimposed superstructure to expand and contract, but the resulting bending stresses shall not exceed the allowable.

During phased construction of a capped pile pier, do not design a pier phase to be supported on less than three (3) piles. For cap and column piers, do not design a phase to be supported on less than two (2) columns.

For a new or replacement structure, individual free-standing columns without a cap are not permitted.

303.3.1.1 BEARING SEAT WIDTHS

Pier bearing seat widths for reinforced concrete slab bridges should conform to Standard Bridge Drawing CPP-1-08. Also see Section 303.3.2.5 of this Manual.

Pier caps on piles, drilled shafts or on columns are normally a minimum of 3'-0" [915 mm] wide. This is the standard width used for continuous span prestressed box beams and I-beams. Bearing seat widths of 3'-0" [915 mm], while normally adequate must be verified by the designer of the structure. Large bearings, skew angle, intermediate expansion devices, AASHTO earthquake seat requirements, etc. may require additional width.

303.3.1.2 PIER PROTECTION IN WATERWAYS

See Section 200 of this Manual for piling protection requirements and Section 600 for a plan note to be added to design drawings when the Capped Pile Pier Standard Bridge Drawing is not referenced.

303.3.2 TYPES OF PIERS

303.3.2.1 CAP AND COLUMN PIERS

When designing the cantilever portions of cap and column piers, the design moments shall be calculated at the actual centerline of the column.

The uppermost layers of longitudinal reinforcing steel in the pier cap shall not be lap spliced at the centerline of a column.

Longitudinal reinforcing shall conform to AASHTO. Round columns shall be reinforced with spiral reinforcing placed directly outside the longitudinal bars.

Round columns are preferred and normally should be 36" [915 mm] diameter.

Cap dimensions should be selected to meet strength requirements and to provide necessary bridge seat widths. Caps should be cantilevered beyond the face of the end column to provide approximately balanced factored dead load moments in the cap. The end of the cantilevered caps should be formed perpendicular to the longitudinal centerline of the cap to allow for uniform development lengths for the reinforcing steel. Cantilevered pier caps may have the bottom surface of the cantilever sloped upward from the column toward the end of the cap. Cantilevered caps may be eliminated for waterway crossing where debris removal access is an issue.

Minimum column diameters of 36 in. [915 mm] are generally used with spiral reinforcing. Spirals are made up of #4 [#13M] bars at 4.5 in. [115 mm] c/c pitch with a 30 in. [765 mm] outside core diameter. Using the circumference of the spiral as the out-to-out of the reinforcing steel bar, this column size normally has a relatively large ratio of the column's axial load capacity to the axial load (e.g. more than 1.5). Therefore, while this spiral reinforcement does not conform to *LRFD 5.7.4.6* and *5.10.6.2* it is acceptable if the ratio of axial load to axial capacity is over 1.5.

For columns where the ratio of axial capacity to axial load is less than 1.5, the spiral reinforcing should conform to *LRFD 5.7.4.6* and *5.10.6.2.*.

In no case shall column reinforcement not meet minimum cross section area, shrinkage and temperature requirements of AASHTO.

303.3.2.2 CAP AND COLUMN PIERS ON PILES

Piers supported on piles generally should have separate footings under each column.

Column piers shall have at least 4 piles per footing.

For grade separation structures, the top of the pier's footings should be a minimum of 1'-0" [300 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is located in a raised earth median barrier.

303.3.2.3 CAP AND COLUMN PIERS ON DRILLED SHAFTS

Where columns are supported on a drilled shaft foundation, the drilled shaft should be at least 6 inches [150 mm] larger in diameter than the column. This is to allow for field location tolerances of the drilled shaft. A drilled shaft foundation is defined as starting 1 foot [0.3 meter] below ground level or 1 foot [0.3 meter] above normal water.

303.3.2.4 CAP AND COLUMN PIERS ON SPREAD FOOTINGS

Cap and column piers on spread footings, placed on existing soils or on embankment fills, should have continuous footings which should extend beyond the center of the end column a distance equal to approximately 1/3 of the distance between the end column and the adjacent column, in order to provide approximately balanced moments.

Cap and Column piers with spread footings on bedrock shall have separate footings under each column.

For grade separation structures, the top of pier footings should be a minimum of 1'-6" [450 mm] below the level of the bottom of the adjacent ditch. This applies even though the pier is located in a raised earth median barrier. In no case should the bottom of the footings in existing soil or on embankment fills be above the frost line.

The width of footing for a free-standing pier generally shall be not less than one-fourth the height of the pier where founded on soil and not less than one-fifth the height of the pier where founded on bedrock.

303.3.2.5 CAPPED PILE PIERS

Steel H piles shall be a minimum HP12x53 [HP310 x 79]**.** The piles should be shown on the plans with the flanges of the H-section perpendicular to the face of the pier cap.

The distance from the edge of a concrete pier cap to the side of a pile shall be not less than 9 inches [230 mm].

The diameter of the exposed portions of cast-in-place reinforced concrete piles generally should be 16 inches, but if exposed length, design load or other conditions make it necessary, larger diameter cast-in-place piles should be used. Cast-in-place piles shall be reinforced with a reinforcement cage composed of 8-#6 reinforcing bars with a 12 inch outside diameter, #4 spiral, with a 12 inch pitch. The cage length should extend from the finished top of the pile to 15 feet below the assumed point of fixity (minimum 15-ft below ground level). The reinforcing steel shall be shown in the structure's reinforcing bar list and be included in item 507 for payment. The use of cast-in-place piles greater than 16 inches in diameter will require an increase in the width of the cap of Standard Bridge Drawing CPP-1-08. See Section 303.4.2.3.

Exposed H piles and unreinforced concrete piles shall have pile protection. Refer to the description in Standard Bridge Drawing CPP-1-08. A plan note is also available. Also See Section 200 for a description of pile protection.

For pile embedment requirements into concrete, see Section 303.4.2.3.

An optional construction joint shall be shown at the top of pier caps for reinforced concrete slab bridges. This joint is optional as some machine finishing equipment for slab bridge decks require a uniform depth of freshly placed concrete in order to obtain best results.

For phased construction projects, do not design a pier or abutment phase to be supported on less than three (3) piles.

To avoid non-redundant designs, capped pile piers should be supported on at least 5 piles. Refer to *LRFD 10.5.5.2.3* for more information.

303.3.2.6 STEEL CAP PIERS

If at all possible this alternative should not be selected. This is a fracture critical design that has historically shown both steel member and weld metal cracking problems. As specified in Section 301.2, these structure types require a concurrent detail design review to be performed by the Office of Structural Engineering.

If a steel box girder is required as a pier cap, the design shall allow reasonable access to the interior for maintenance, inspection and repair purposes. The physical dimensions of the box shall be large enough to allow access to the interior for inspection. Access hatches of the box girder should be bolted and sealed with a neoprene gasket. Access hatches should also be light enough for an inspector to easily remove them. One recommended lightweight material is ABS plastic.

Designers shall ensure that all governmental agency regulations regarding to enclosed spaces, ventilation, lighting, etc. are complied with within any enclosed steel pier cap design.

Box designs with cut away webs to allow for stringers to continue through the box are generally not considered acceptable alternatives.

306.2.8 TOOTH TYPE, FINGER TYPE OR NON-STANDARD SLIDING PLATE EXPANSION DEVICES

Finger or sliding plate joints are another alternative type of expansion device where movements exceed the capacity of either strip or compression seal devices. This type of expansion device generally competes against Modular joints. Their advantage is their simplicity of design. Their disadvantage is their inability to seal against intrusion of water and debris. Consult the Office of Structural Engineering for recommendations prior to completion of the project plans. Example plan notes are provided in the appendix.

Use of a tooth type expansion device also requires neoprene drainage troughs and a suitable drainage system to carry away the water. Both the neoprene trough and downspout to drainage trough connection must be detailed completely. Special attention should be paid to developing a complete seal at the downspout to trough connection.

Vulcanization is recommended over adhesive for sealing.

Finger devices shall be designed for fatigue and conform to fracture critical requirements if the design has fracture critical components in it.

Fabricators of finger or sliding plate devices shall be pre-qualified 513 Level UF fabricators. Review Section 302.4.1.3 and contact the Office of Structural Engineering for recommendations.

306.3 EXPANSION DEVICE USES – BRIDGE OR ABUTMENT TYPE

306.3.1 INTEGRAL OR SEMI-INTEGRAL TYPE ABUTMENTS

No allowance for temperature need be made.

The vertical joint between abutment backwall and approach slab should be finished as per Standard Bridge Drawing AS-1-81, Detail B.

306.3.2 REINFORCED CONCRETE SLAB BRIDGES

The following table specifies joint requirements. Expansion length is defined as the total length if no fixed bearing exists, or length from fixed bearing to proposed expansion device location, if one exists.

Expansion Length (ft)	(m)	**Joint Required**	**Approach Slab Joint**
0 - 40	0 - 12	None	AS-1-81 detail B
40 - 200	12 - 60	None **(1)** PM **(2)**	AS-1-81 detail B **(1)** AS-1-81 detail D **(2)**
200 +	60 +	PM	AS-1-81

(1) = flexible abutments and piers (CPP-1-08 and CPA-1-08)

(2) = abutments and/or piers fixed or rigid

PM = Polymer Modified Asphalt Joint

306.3.3 STEEL STRINGER BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.

Expansion Length (ft)	(m)	**Joint Required**	**Approach Slab Joint**
0 - 30	0 - 10	None	AS-1-81 detail B
30 - 125	10 - 38	PM **(1)** or EXJ-4-87	AS-1-81 detail B
125 - 400	38 - 125	EXJ-4-87	AS-1-81
400 +	125 +	TTED or MED	

PM = Polymer Modified Asphalt Joint

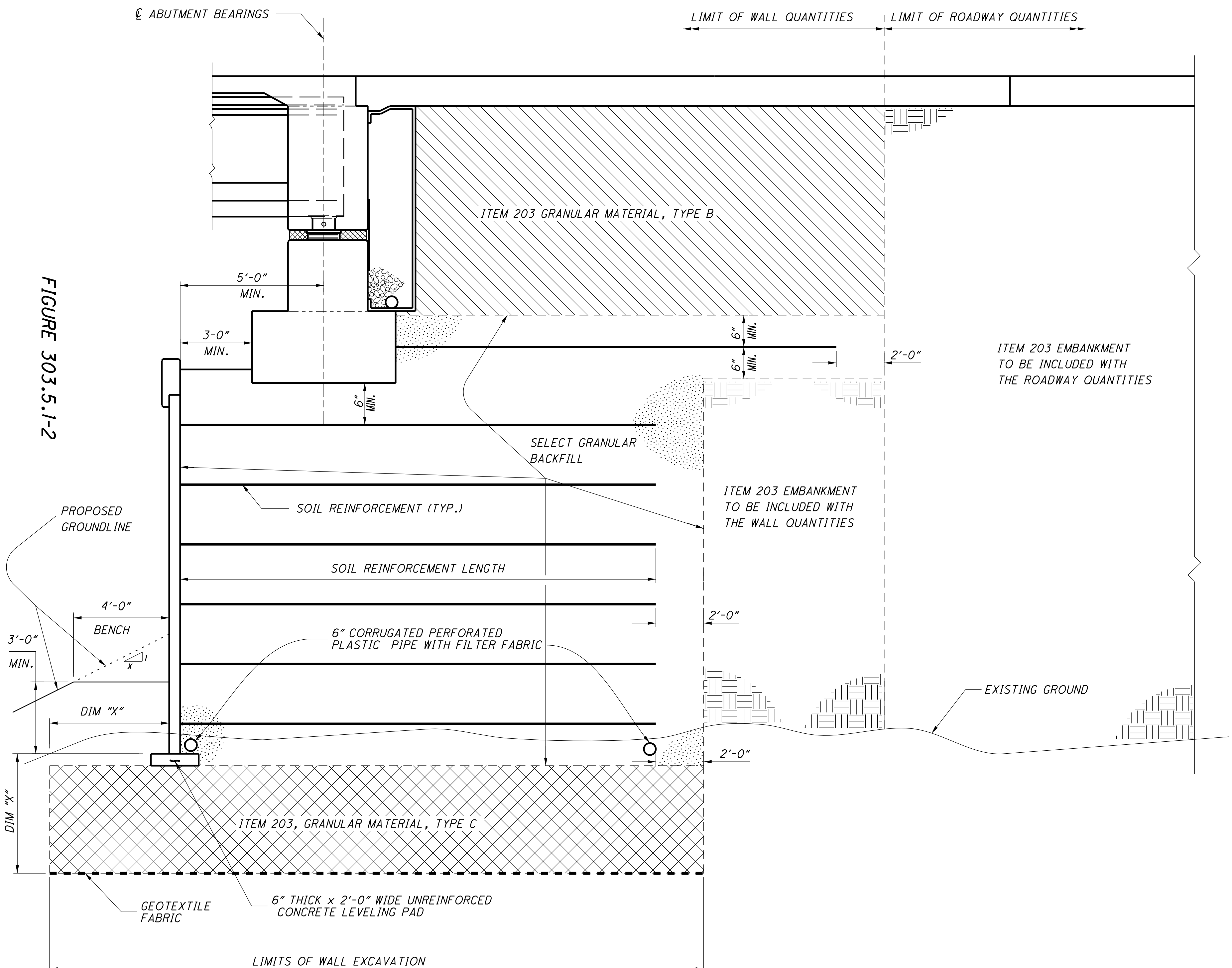
TTED = Tooth Type expansion device

MED = Modular Expansion Device

(1) = Stringer bridges with sidewalks should not use polymer modified expansion joint systems.

306.3.4 PRESTRESSED CONCRETE I-BEAM BRIDGES

The following table specifies joint requirements based on expansion length defined as the total length of structure, if no fixed bearing exists, or length from fixed bearing to proposed expansion device location if one exists.



SECTION A-A

FIGURE 303.5.1-2

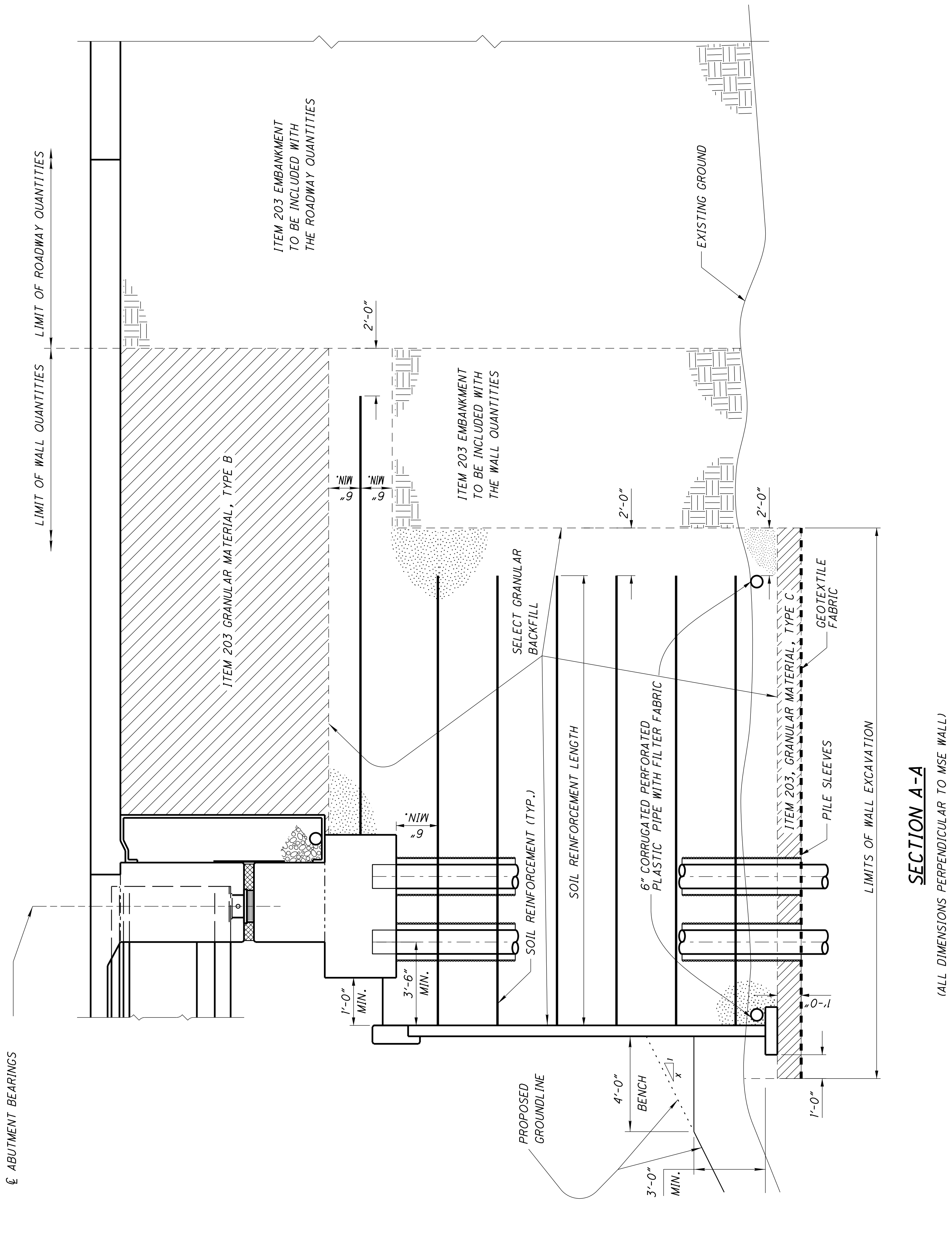
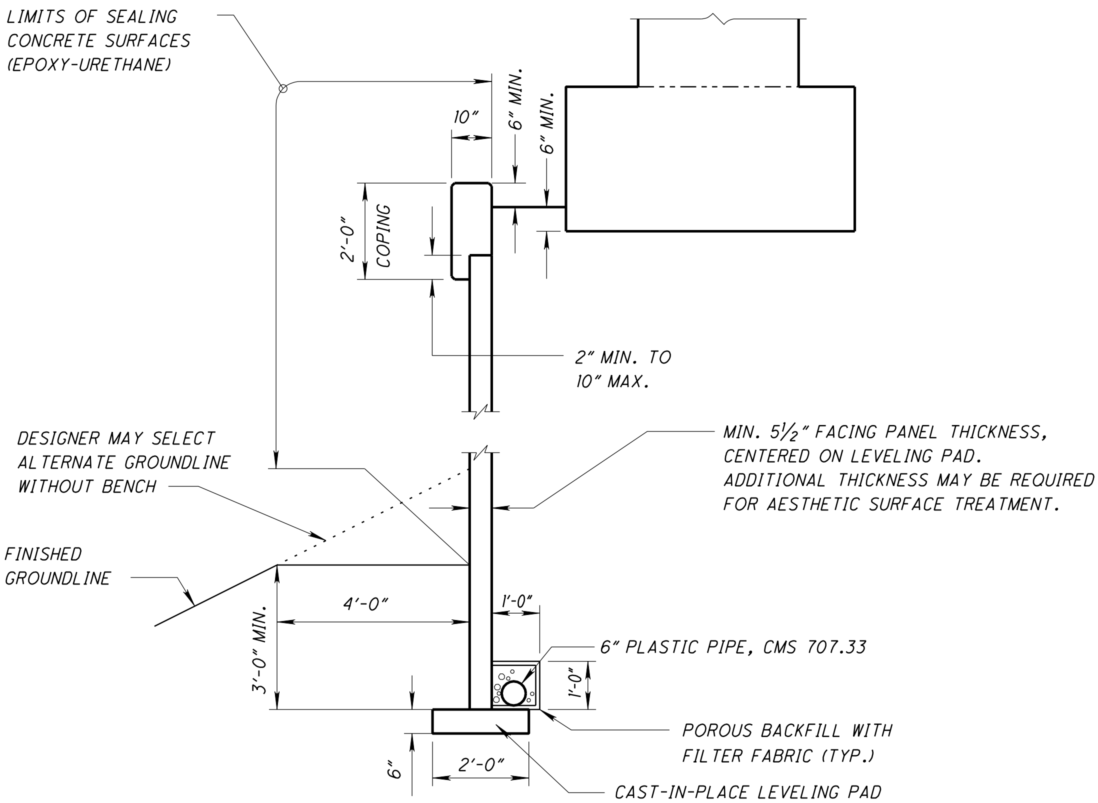
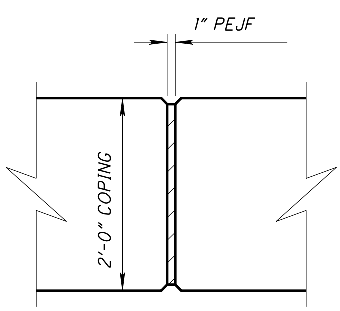


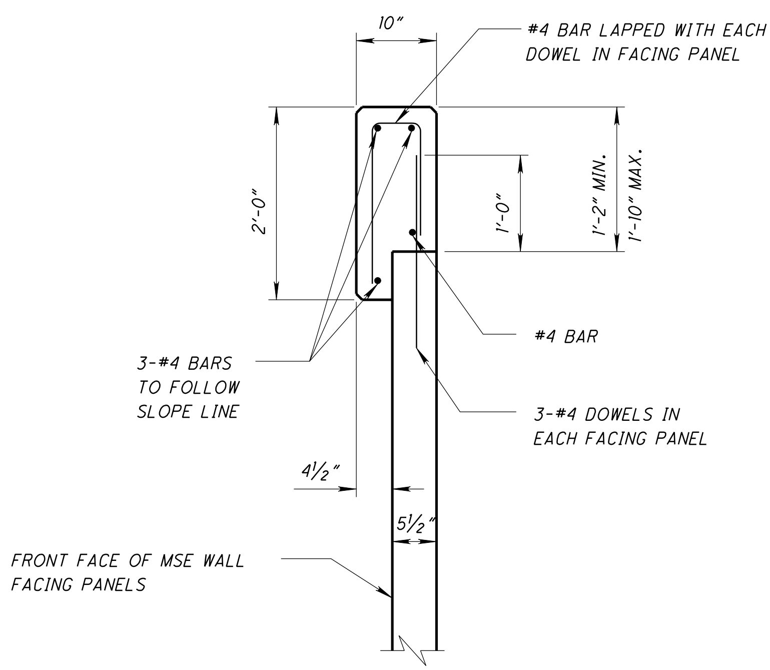
FIGURE 303.5.1-3



MSE WALL AND COPING DETAIL



COPING EXPANSION JOINTS



MSE WALL COPING

ALL REINFORCING STEEL TO BE EPOXY COATED

FIGURE 303.5.1-4

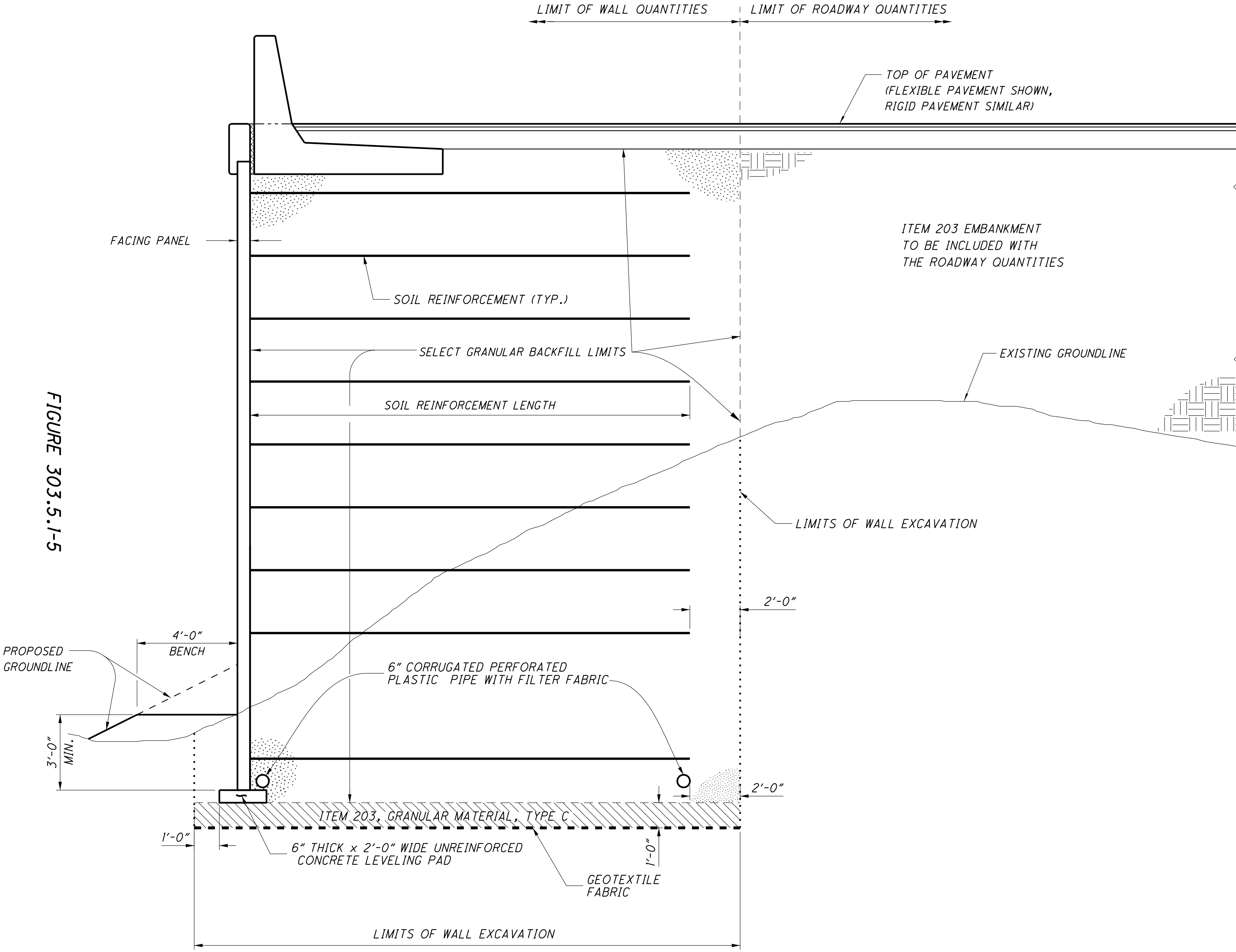


FIGURE 303.5.1-5

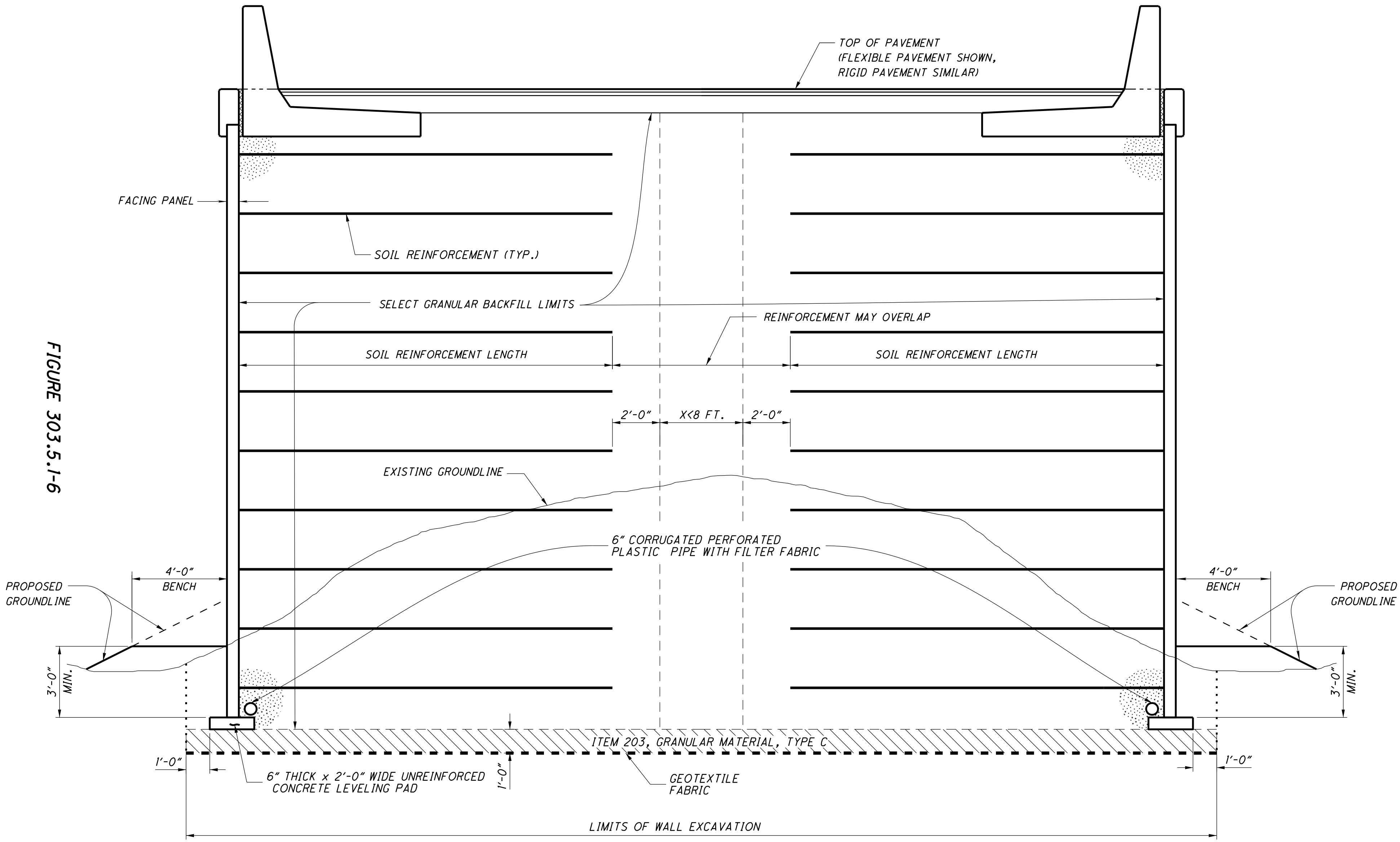
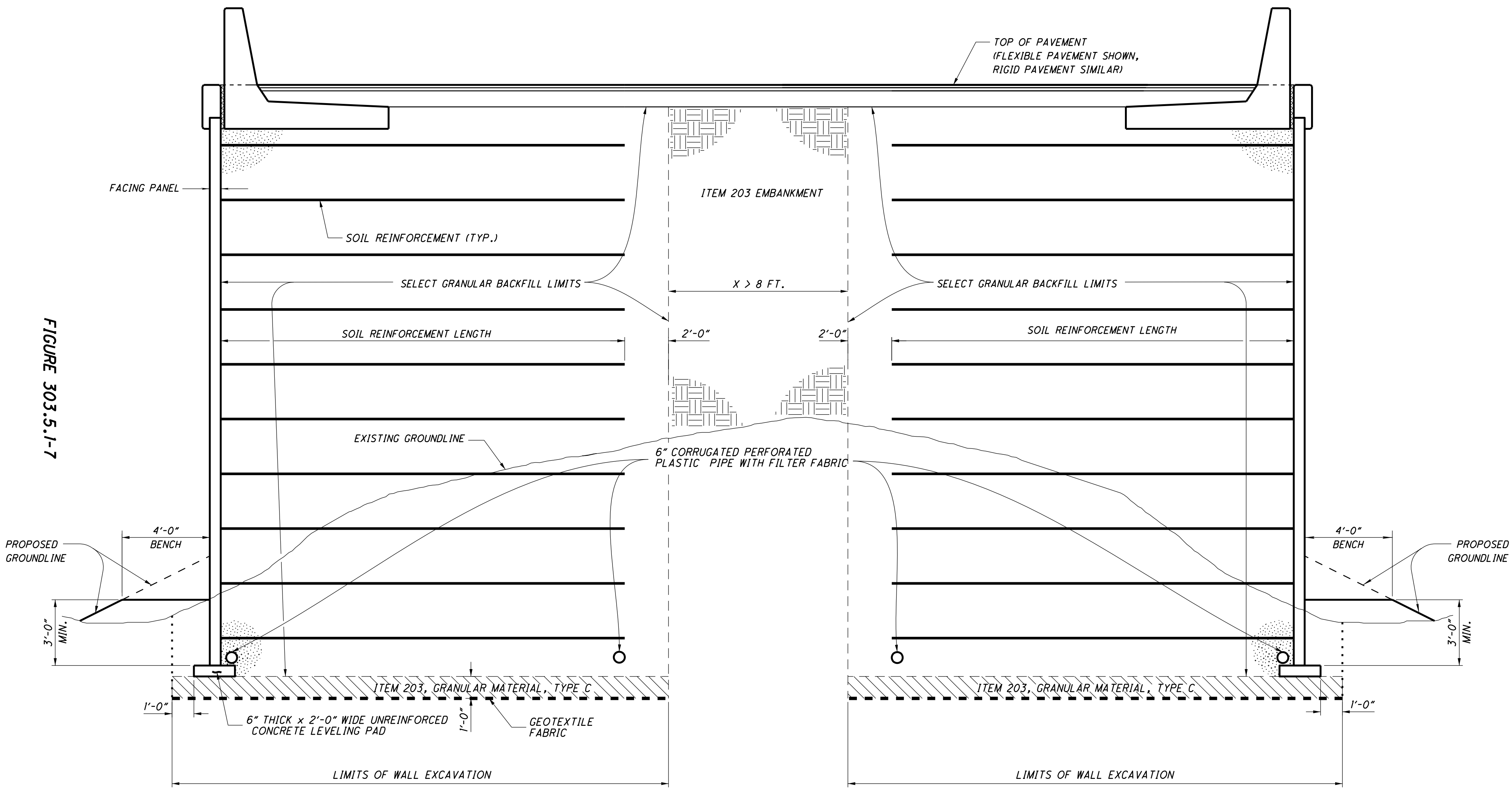


FIGURE 303.5.1-6

IF X IS LESS THAN 8 FT., USE SELECT GRANULAR BACKFILL MATERIAL BETWEEN SOIL REINFORCEMENT.

SEE ROADWAY PLANS FOR PAVEMENT BUILD UP



IF X IS MORE THAN 8 FT., USE ITEM 203 EMBANKMENT BETWEEN SOIL REINFORCEMENT.

SEE ROADWAY PLANS FOR PAVEMENT BUILD UP

FIGURE 303.5.1-7

SECTION 800 – NOISE BARRIERS

801 INTRODUCTION

According to Section 1400 of the ODOT Location and Design Manual, a Noise Wall Justification shall be included in the Preferred Alternative Verification Review Submission for Major Projects or in the Minor Project Preliminary Engineering Study Review Submission. When noise barriers are necessary, the Office of Environmental Services will furnish the required noise barrier height, length and location(s). The detail design for noise barriers shall be included in the Stage 2 Detailed Design Review Submission.

Design specifications for ground mounted precast concrete noise barrier walls are provided in Standard Bridge Drawing NBS-1-09. Associated designer notes are provided in Design Data Sheet NBSDD-1-09. The Department occasionally permits the use of noise barrier walls consisting of material types other than precast concrete. These wall types are pre-approved according to the requirements of the Department's Standard Procedure 27-005(SP) for new products and this Manual. Alternate noise barrier material types currently approved include: metal; fiberglass; brick or masonry; and acrylic. A complete listing of approved noise barrier suppliers for material types other than precast concrete are provided in Figures 801-1 and 801-2.

802 DESIGN CONSIDERATIONS

802.1 NOISE BARRIER FOUNDATIONS

802.1.1 GENERAL

The Design Agency shall perform a subsurface investigation at all noise barrier locations. The subsurface work shall be in accordance with the most current revision of the ODOT Specifications for Geotechnical Explorations. The noise barrier borings shall be included in the plans with the soil profile/foundation investigation sheets.

The standard foundation for noise barrier walls is a 30-inch diameter drilled shaft with a maximum length of 30-ft. Consult the Office of Structural Engineering when specifying longer or larger diameter drilled shafts.

In regions of poor soils or where obstructions (e.g. underground utilities, drainage facilities, mse wall components, etc.) preclude the use of 30-inch diameter drilled shafts as the appropriate foundation type, consult the Office of Structural Engineering for the use of an alternate foundation type (e.g. larger diameter drilled shafts, spread footings, etc.). If bedrock is anticipated within the drilled shaft length required by BDM Section 802.1.2 and the bedrock has an unconfined compressive strength of 7500 psi or better, provide the required shaft length or a reduced length with a 3-ft minimum length rock socket. For weaker bedrock, provide the required shaft length or a reduced length with a 5-ft minimum length rock socket.

802.1.2 DRILLED SHAFT DESIGN

The following foundation design procedure applies to only 30-inch diameter drilled shafts. For shafts of other diameter or for design parameters that exceed those herein, the foundation shall be designed in accordance with the AASHTO LRFD Bridge Design Specifications, Section 10.

A. At each noise barrier boring location determine the SPT "N" blow counts from 2.5-ft to 25-ft in 2.5-ft increments. The SPT "N"-value is the total number of blows required to drive the sampler from 6" to 12" and from 12" to 18".

B. Correct the N-values for hammer efficiency and depth. Use the following depth correction factors:

Depth (ft.)	Correction Factor	Depth (ft.)	Correction Factor
2.5	1.6	15.0	1.0
5.0	1.4	17.5	0.96
7.5	1.2	20.0	0.91
10.0	1.1	22.5	0.88
12.5	1.1	25.0	0.84

C. The final design "N"-value used to establish the required minimum shaft length should be based on either average or lowest corrected "N"-values as follows. When the corrected "N"-values are consistent with depth or when the corrected values increase with depth, the final design "N"-value shall be the average of the corrected values along the length of drilled shaft. Otherwise, the final design "N"-value shall be the lowest corrected value along the length of the drilled shaft. The following examples assumes a drilled shaft with a design length of 15-ft.

Depth (ft.)	Corrected "N"-value	
5	8	
10	15	
15	16	Design "N" = (8+15+16)/3=13
20	25	
25	30	

Depth (ft.)	Corrected "N"-value	
5	16	
10	8	
15	15	Design "N" = 8
20	25	
25	30	

D. Establish the soil type as granular or cohesive at each boring. Soil should be considered granular when the plasticity index is less than 7.

E. Select the Granular Soil Foundation Depth Table (Figure 802.1.2-1) or the Cohesive Soil Foundation Depth Table (Figure 802.1.2-2) to determine the required drilled shaft length for the assumed post spacing and wall height at each boring location. Refer to the Design Data

Approved Reflective Barrier Suppliers

Type	Suppliers	Drawings & Notes
Metal	Empire International 36744 Constitution Drive Trinidad, Colorado 81082 Telephone: (719)846-2300 Fax: (719)846-7466 SB1 (3/11/92) Reflective Horizontal/Continuous Wall	SB1 (3/11/92) Reflective Horizontal/One Span Shadowbox SB1 (3/11/92) Reflective Horizontal/Two Span Shadowbox SB2 (3/11/92) Reflective Vertical/Continuous Wall SB2 (3/11/92) Reflective Vertical/One Span Shadowbox SB2 (3/11/92) Reflective Vertical/Two Span Shadowbox
	Centria 1005 Beaver Grade Road Moon Township, PA 15108 Telephone: (888)348-2406 or (412)299-8170 www.ecosoundbarrier.com	Centria Reflective Eco-Sound Barrier System - Steel Panel Option (02/19/09)
Fiberglass Wood Panel	CorTec Company 3401 S. Delaware Milwaukee, Wisconsin 53207 Telephone: (414)486-1876 or 1-800-879-4377 Fax: (740)636-3250	Crane CorTec Sound Barrier System Manual (4/1/90)
Brick or Block Masonry	Masonry Institute of Dayton 2077 Embury Park Road P.O. Box 14026 Dayton, Ohio 45414 Telephone: (937)278-7821 Fax: (937)599-3683	Block Masonry Drawing Rev. (4/83) Notes Rev. (3/83) Brick Masonry Drawings Rev. (3/83)(2/85) Notes Rev. (12/91)
	Advanced Masonry Technology, Inc. 2786 Center Road P.O. Box 878 Brunswick, Ohio 44212 Telephone: (330)225-9496 Fax: (330)273-0046	Advanced Masonry Technology Brick Masonry Sound Barrier Panel Drawings: SBC1, SB1, SB2 & SB3 Dated 8/6/02
Fiberglass	Carsonite International 10 Bob Gifford P.O. Box 98 Early Branch, South Carolina 29916 Telephone: (803)943-1172	#9106121-02-2 (3/5/92) Product Binder #30-02180-92 #SBSA1001, Sheets 1 thru 8, (10/22/97) #SBSA1002, Sheet 1, (10/22/97)
	M.H. Corbin, Inc. 9021G Heritage Drive Plain City, Ohio 43064 Telephone: (614)873-8216 or 1-800-380-1718 Fax: (614)873-8095	FiberCor Composite Reflective Noise Barrier System sheets 1, 2 & 3 dated 3/8/04
Acrylic	Durisol Inc./Evonik Cyro LLC 67 Frid St. Hamilton, Ontario Canada, L8P 4M3 Telephone: (905)521-8658 Email: Edwards@Durisol.com	Paraglas Soundstop Noise Barrier Sheet 20 mm Paraglas Soundstop Noise Barrier Sheet 25 mm Paraglas Soundstop GSCC Noise Barrier Sheet 20 mm (Structure Mounted Applications) Paraglas Soundstop GSCC Noise Barrier Sheet 25 mm (Structure Mounted Applications) Paraglas Soundstop Ready-Fit Panels Paraglas Soundstop TL4 System
	Faddis Concrete Products & Plaskolite 3515 Kings Hwy Downingtown, Pennsylvania 19335 Telephone: 1-800-777-7973 or (610)269-4685 Fax: (215)873-8431 or (610)873-8431	AcoustaClear Noisebarrier with Optix NB Extruded Acrylic Drwg. No. GSF-B-2008-01 (18 sheets) dated 5/8/08

Figure 801-1

Approved Absorptive Barrier Suppliers

Type	Suppliers	Drawings & Notes
Metal	Empire International 36744 Constitution Drive Trinidad, Colorado 81082 Telephone: (719)846-2300 Fax: (719)846-7466	88SSA-101 Empire Silent Screen (1 of 2)(6/6/88) 88SSA-102 Empire Silent Screen (2 of 2)(6/6/88) 88SSA-301 Empire Sight and Sound Wall (1 of 2)(6/6/88) 88SSA-302 Empire Sight and Sound Wall (2 of 2)(6/6/88)
	Industrial Acoustics Company, Inc. 1160 Commerce Ave. Bronx, New York 10462 Telephone (718)931-8000 of (718)863-1138	Industrial Acoustics Metal Sound Absorptive Dwg. C-9413-444-1 issue B dated 9/2/94
	Faddis Concrete Products 3515 Kings Hwy. Downingtown, Pennsylvania 19335 Telephone: 1-800-777-7973 or (610)269-4685 Fax: (215)873-8431 or (610)873-8431	Faddis – Metal Noise Barrier "ACOUSTAX" (AIR FORCE 1) Sound Absorptive Noise Barrier Panels Drawings 1 thru 8 dated 6/7/01

Figure 801-2