SECTION 4 - SUBSTRUCTURES

4.1 - FOUNDATIONS

All foundation designs should be preceded by a thorough **subsurface investigation**. The first step is to gather all existing subsurface explorations and pile driving records, if available.

For **projects designed in-house**, the Authority's Geotechnical Engineer will develop a subsurface exploration plan based on a review of the existing information (if available) and the proposed foundation construction. The designer must provide the following information to the Geotechnical Engineer:

1. Plan and Elevation Drawing showing the following:
    a. Existing and Proposed Substructures.
    b. Bottom of Footing Elevations of Existing and Proposed Footings.
    c. Type of Abutments.
2. Proposed Pile Loads (if applicable).
3. Staging Information.
4. Design and Construction Schedules.

The Authority's Geotechnical Engineer will then make arrangements for the drilling and laboratory testing.

For **projects designed by a Consultant**, the Consultant will develop a subsurface exploration plan which is then submitted to the Authority’s Geotechnical Engineer for review and approval. The Consultant will then obtain three bids for drilling and laboratory testing based on the approved subsurface exploration plan and the Authority’s subsurface exploration specification (refer to the appendices in the **DRM** for a copy of the specification). All explorations shall be progressed under the supervision of a **qualified** drilling inspector. The required experience for drilling inspectors is found on the page following the specification. Resumes for proposed drilling inspectors should be submitted to the Authority’s Geotechnical Engineer for approval. All new foundation designs will require a **Foundation Design Report (FDR)**.

For **projects designed in-house**, the Authority’s Geotechnical Engineer will prepare the **FDR**. For **projects designed by a Consultant**, the Consultant will prepare the **FDR** which is then submitted to the Authority’s Geotechnical Engineer for review and approval. As a minimum, the **FDR** will include a description of the existing foundation conditions, recommendations for the proposed foundation type and details, seismic considerations (evaluation of liquefaction potential and seismic design site coefficient), erosion protection and dewatering recommendations (if appropriate), excavation and backfill recommendations including temporary excavation support and all necessary foundation notes to be placed on the Contract Plans. Please note that the minimum requirements for Consultant-prepared **FDR**s have been formalized for use in structure design scopes (refer to the appendices in the **DRM** for a copy of “Foundation Designs for Structures – Minimum Requirements for Consultant Design”).

<u>4.1.1 - FOOTINGS ON ROCK</u>

Substructure footings proposed to be founded on rock shall be designed and detailed in accordance with the **FDR** and the following subsections.

<u>4.1.1.1 - ROCK LINES</u>

Rock lines should be shown on the plans only when the footings are on rock or when drilled shafts or caissons are to be placed to rock. When rock lines are shown on the plans, they shall be marked as "Assumed Rock Surface." The elevations of the rock are not to be labeled.

When it is planned to place footings on or key footings into rock, the plans shall show the top of footing elevation and the minimum depth of footing. This will enable adjustments to be made in the depth of footing if the actual rock elevation differs from that assumed during design, while keeping the top of the footing elevation constant.

<u>4.1.1.2 - KEYING OR DOWELING FOOTINGS INTO ROCK</u>

Rock removal shall be avoided whenever possible in the construction of footings. Footings shall not be detailed with keys or dowels into rock unless dictated by design requirements or other special circumstances. This will be noted in the **FDR**. When a footing must be keyed into rock, usually the entire footing is keyed into rock to simplify construction. When a footing is doweled into rock, the dowels shall be #9 reinforcing bars or larger and shall be embedded into the footing as well as into the rock to a depth noted in the **FDR**. The designer shall determine the required spacing between the rows of dowels, but in no case shall there be greater than 3.0 feet between rows or less than two rows.

Doweling is generally preferred to keying except where the rock is shale or the rock scour susceptible. Doweling shall not be used in shale rock or scour susceptible rock. The recommendation of whether to key or dowel is contained in the **FDR**.

4.1.2 - STEPPED FOOTINGS

Steps in footings for wingwalls, retaining walls, abutments, piers, and precast panel wall systems should be governed by the following guidelines. Proposed steps in footings shall be shown on the **ADP**'s for both in-house and consultant designed structures.

4.1.2.1 - FOOTINGS ON SOIL

A. Stepping of abutment or pier footings on either spread foundations or piles should be avoided. If the Designer has reason (such as intruding or sloping bedrock) to step the footings for these substructures, he shall seek approval from the **DSD**.
B. Steps should not be used in footings less than 20 feet in length. If permitted, the minimum length of step section shall be 11 feet long. The depth of the step should not be less than 2 feet. Footing continuity should be considered, however, it is not mandatory. A vertical joint shall be required between the wall stems at the location of the step. The joint type shall be the same type as in the footing. Typically, a keyed expansion joint is

used. Stepping of the footings for wingwalls and retaining walls shall be approved by the **DSD**.

C. Stepping of the leveling pad for a mechanically stabilized earth system on embankments is permitted. The minimum length of a step section may be the width of one panel. The minimum depth of a step for this type of wall system is 3 feet, which includes one-half panel height plus the leveling pad. The manufacturer of the mechanically stabilized earth system shall set the final configuration of the leveling pad as part of their panel layout.

4.1.2.2 - FOOTINGS ON ROCK

Stepping of footings on rock is acceptable for all footing lengths greater than 12 feet. Portions of the wall should usually be at least 8 feet long and have a step not less than 2 feet. Footing continuity is not required. The **FDR** will show an assumed top-of-footing elevation for each step.

4.1.3 - DESIGN FOOTING PRESSURES AND PILE CAPACITIES

Spread footing foundations or pile foundations shall be detailed on the plans. The appropriate note(s) to be included in the plans will be contained in the **FDR**. The following subsection will provide instructions for filling in the blank spaces of these notes.

4.1.3.1 - SUBSTRUCTURES ON SPREAD FOOTINGS

When the foundation consists of a spread footing on rock or soil, the following note shall be shown on the substructure Footing Detail Sheet(s).

"The footing for the _____ is designed to exert a maximum bearing pressure of _______ ksf."

For **spread footings on soil**, the maximum bearing pressure exerted by the footing is usually very close to the maximum allowable bearing pressure given in the **FDR**. For this reason, for footings on soil, the maximum allowable bearing pressure from the **FDR** should be used in this note (The **FDR** will show this plan note with the value already filled in).

For **spread footings on rock**, the maximum bearing pressure exerted by the footing is usually quite a bit smaller than the maximum allowable bearing pressure given in the **FDR**. For footings on rock, this value will be left blank in the **FDR** plan note and is to be filled in by the designer with the actual maximum bearing pressure exerted by the footing.

For closed box culverts, the word "culvert" shall be substituted for "footing" in the above note. Wingwalls attached to culverts will require a separate note, since the pressure under the wingwall footing is generally greater than that under the culvert floor.

4.1.3.2 - SUBSTRUCTURES ON PILES

The appropriate pile notes to include in the plans will be contained in the **FDR**. The designer and the Geotechnical Engineer should discuss maximum allowable pile loads for the project prior to finalization of the **FDR**. If these loads change after issuance of the **FDR**, the Geotechnical Engineer should be informed, and a supplemental **FDR** will be issued.

4.1.4 - PILES

4.1.4.1 - STEEL H-PILES

The footing thickness shall not be less than 2.5 feet for steel H-Piles. Footing areas shall be so proportioned that pile spacing shall be not less than 3 feet center-to-center for steel piles. The maximum pile spacing shall be 9 feet. The tops of steel piles shall project no less than 1 foot into the footing. The minimum distance from the center of a pile to the nearest footing edge shall be 1.5 feet, but in no case shall the distance from the edge of the pile to the nearest edge of the footing be less than 9 inches.

For integral abutment stems the minimum pile spacing shall be not less than 1 foot center-to-center for steel piles. The maximum pile spacing shall be 9 feet. A single row of piles shall be placed at the centerline of the integral abutment stem unless otherwise noted in Subsection 4.6 – Integral Abutments. The minimum stem thickness for integral abutments is 3 feet unless otherwise noted in Subsection 4.6. Piles, for integral abutments shall be imbedded sufficiently into the stem to insure fixity for developing the plastic moment capacity of the pile, but no less than 2 feet. The minimum distance from the center of a pile to the nearest stem edge shall be 20 inches, but in no case shall the distance from the edge of the pile to the nearest edge of the footing be less than 1 foot. Where a reinforced concrete beam is used as a bent cap supported by piling, the minimum pile spacing shall be not less than 3 feet center-to-center for steel piles. The maximum pile spacing shall be 9 feet. The minimum distance from the center of the pile to the nearest cap edge shall be 1.5 feet, but in no case shall the distance from the edge of the pile to the nearest edge of the cap be less than 1 foot. The piles shall project at least 2 feet into the cap. Cap plates are not required for steel bearing piles.

4.1.4.1.1 - SPLICES FOR STEEL H-PILES

When steel bearing piles are specified and the estimated length exceeds 30 feet, the designer should include either the standard specification item for splicing steel bearing piles or the NYSDOT special specification for splicing steel bearing piles. The standard specification item requires steel bearing piles be spliced using full penetration groove welds. Welded splices are required for piles subject to uplift loads. The special specification item allows for the use of mechanical splices for steel bearing piles. **Mechanical splices are not acceptable for steel H- piles subject to uplift loads.**

The **FDR** will contain recommendations regarding the use of mechanical pile splices for H-piles. Regardless of the specification used, the quantity of splices will generally be 1/3 the number of piles driven. The estimated quantity will be stated in the **FDR**. Splicing of steel bearing piles is a contingency item that is used when actual driven pile length exceeds the estimated pile length by more than 5 feet. Refer to the most recent version of **NYSDOT BD**-MS5 (Miscellaneous Pile Details) for the appropriate splicing details to include in the contract plans.

4.1.4.2 - CONCRETE CAST-IN-PLACE (CIP) PILES

The footing thickness shall not be less than 2 feet for **CIP** piles. Footing areas shall be so proportioned that pile spacing shall be not less than 3 feet center-to-center for **CIP** piles. The maximum pile spacing shall be 9 feet. The tops of **CIP** piles shall project no less than 6 inches into the footing. The minimum distance from the center of a pile to the nearest footing edge shall be 1.5 feet, but in no case shall the distance from the edge of the pile to the nearest edge of the footing be less than 9 inches.

For integral abutment stems the minimum pile spacing shall be not less than 3 feet center-to-center for **CIP** piles. The maximum pile spacing shall be 9 feet. A single row of piles shall be placed at the centerline of the integral abutment stem unless otherwise noted in Subsection 4.6 – Integral Abutments. The minimum stem thickness for integral abutments is 3 feet unless otherwise noted in Subsection 4.6. **CIP** piles for integral abutments shall be embedded sufficiently into the stem to insure fixity for developing the plastic moment capacity of the pile, but no less than 2 feet. The minimum distance from the center of a **CIP** piles to the nearest stem edge shall be 20 inches, but in no case shall the distance from the edge of the pile to the nearest edge of the footing be less than 1 foot.

Where a reinforced concrete beam is used as a bent cap supported by **CIP** piles, the minimum pile spacing shall be not less than 3 feet center-to-center for steel piles. The maximum pile spacing shall be 9 feet. The minimum distance from the center of the **CIP** piles to the nearest cap edge shall be 1.5 feet, but in no case shall the distance from the edge of the **CIP** piles to the nearest edge of the cap be less than 1 foot. The **CIP** piles shall project at least 2 feet into the cap.

4.1.4.2.1 - SPLICES FOR CIP-PILES

Cast-in-place pile shells are spliced by either welding with a backing ring or by using a mechanical splice. Welded splices are required for piles subject to uplift loads. **Mechanical splices are not acceptable for CIP pilessubject to uplift loads.** There is no splice item for **CIP** piles, as splicing is included in the pile item. The **FDR** will contain recommendations regarding the use of mechanical pile splices for **CIP** piles. Refer to the most recent version of **NYSDOT BD**-MS5 (Miscellaneous Pile Details) for the appropriate splicing details to include in the contract plans.

4.1.4.3 - PILE TIP REINFORCEMENT (H AND C.I.P. PILES)

All H-piles shall be equipped with a either a regular reinforced shoe or an APF HP77750 “hard bite” shoe (or equivalent). All cast-in-place concrete pile shells shall be equipped with either a flat plate or a conical tip. The type of tip treatment is dependent on the soil conditions and will be specified in the **FDR**. Refer to the most recent version of **NYSDOT BD**-MS5 (Miscellaneous Pile Details) for the appropriate shoe details to include in the contract plans.

4.2 - EXCAVATION AND BACKFILL

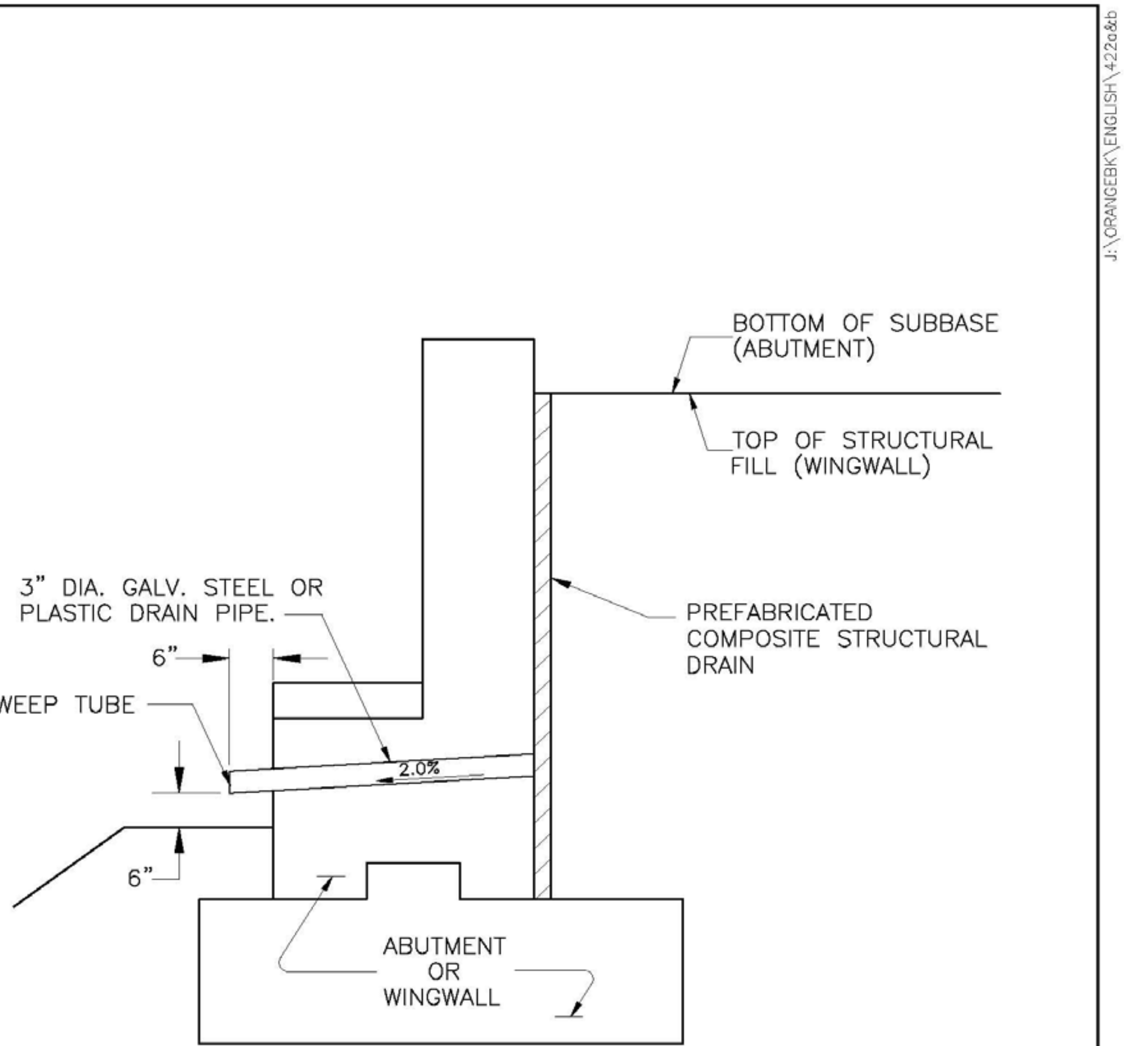
Excavation and backfill details are important aspects of design, which are commonly overlooked. Problems due to settlement or hydraulic pressure buildup can be difficult to solve and cause maintenance headaches in the future. The details provided in the following subsections represent the current policy concerning these elements.

4.2.1 - EXCAVATION AND BACKFILL AT STRUCTURES

The details and payment lines shall be shown on all contract plans and shall conform to the details shown in this manual and the appropriate **BD** Sheet.

4.2.2 - DRAINAGE OF BACKFILL

Prefabricated Composite Structural Drain shall be placed behind the back of all walls, arches, and abutments, except integral abutments. See Detail 4.2.2.a. Prefabricated Composite Integral Abutment Drain shall be placed behind the back of all integral abutments. See Detail 4.2.2.b. In addition, 3 inch diameter weep holes through the structure shall be provided at approximately 16 foot maximum centers with a minimum of three per abutment. The weep tubes shall be flush with the back face of the wall. The weep shall be outletted 6 inches above the finished grade in front of a structure, except in the case of stream bridges, where they are to be outletted 6 inches above low water. Weep tubes shall extend 6 inches beyond the substructure stem at the outlet. When weep holes are placed in the backwall, weep tubes shall extend 6 inches beyond the bridge seat. In certain instances, such as an abutment immediately adjacent to a sidewalk or roadway, a closed drainage system may be required to carry water away from the area.

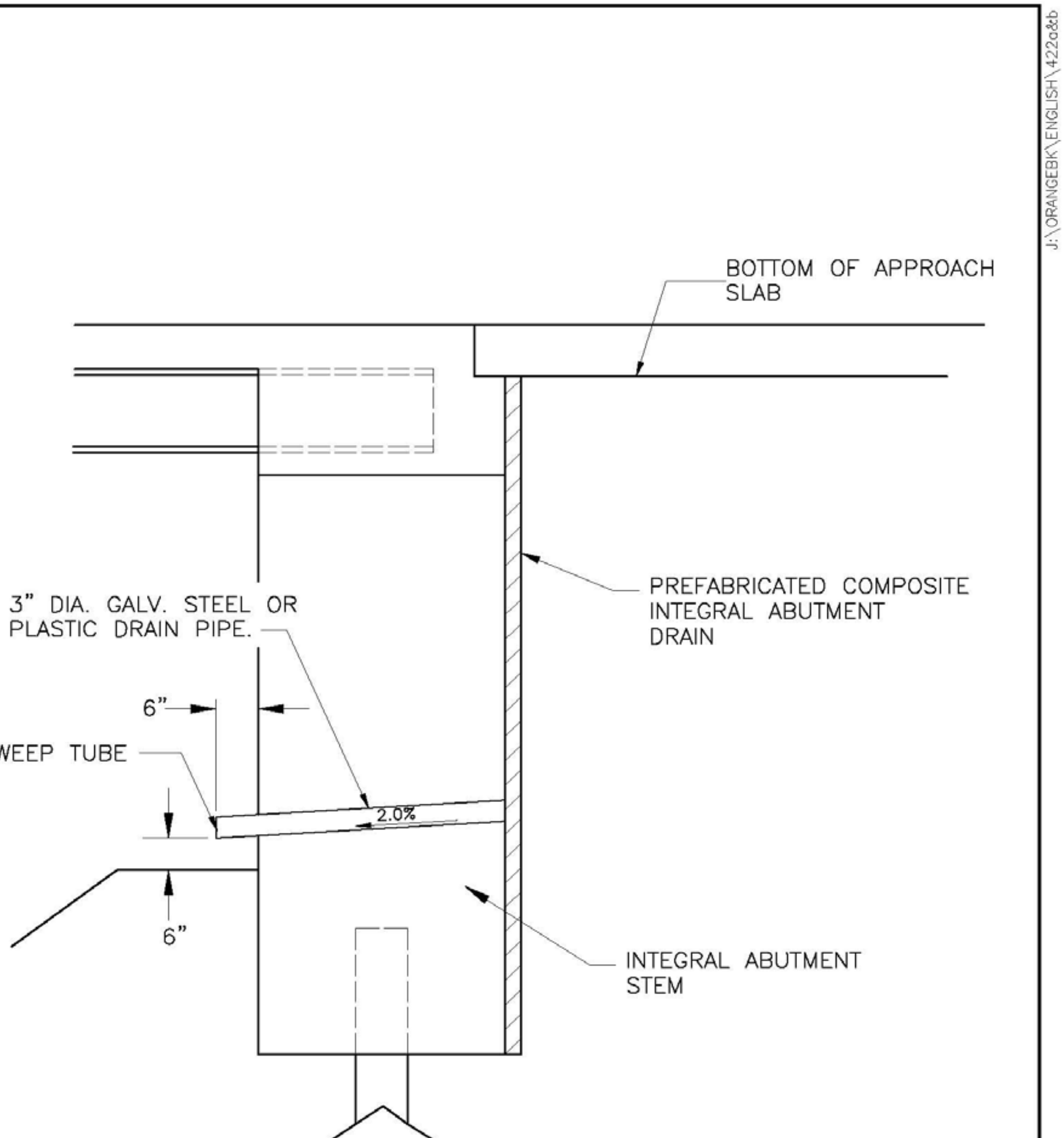


BACKFILL DRAINAGE DETAIL

CANTILEVER TYPE STRUCTURES ONLY

DETAIL 4.2.2.a

N.T.S.



BACKFILL DRAINAGE DETAIL
INTEGRAL ABUTMENTS ONLY

DETAIL 4.2.2.b
N.T.S.

4.2.3 - SHEETING AND COFFERDAMS

Sheeting and cofferdam recommendations will be included in the **FDR** report. The **NYSTA** Geotechnical Engineer is available to assist designers with any sheeting or cofferdam issues that may arise. The Geotechnical Engineer will require information from the **HADR** in order to develop the correct recommendations. Refer to **NYSDOT** Bridge Manual, Section 4 – Excavation, Sheeting, and Cofferdams for design and placement requirements.

4.2.4 - TREMIE SEAL

A tremie seal (concrete placed under water) is used when sheet piling cannot be driven sufficiently deep to eliminate water intrusion into the cofferdam. Refer to **NYSDOT** Standard Specifications Subsection 555-3.05, Depositing Structural Concrete Under Water. The need for a tremie seal and its thickness shall be indicated in the **FDR**. The thickness of the tremie seal is based on **ordinary high water** elevation **(O.H.W.)**, from the **HADR**. Therefore the cofferdam should be designed to flood when the water level exceeds **O.H.W.** to prevent uplift on the tremie seal prior to the placement of the footing. This requirement is handled by using the appropriate plan note from Appendix B, as indicated in the **FDR**.

4.3 - EMBANKMENT AND SLOPE PROTECTION

The following subsections provide information concerning embankment materials and slope protection guidelines.

4.3.1 - SLOPE PROTECTION

The preliminary drawings for each bridge shall show the slope protection to be used on slopes under the structure. The slope protection shall extend a minimum of 3 feet beyond the fascia lines of the structure, and from the abutment face to the edge of pavement. Refer to Details 4.3.1.1.a and 4.3.1.1.b. The guidelines in this subsection indicate suggested materials for use in particular situations.

Other materials may be used when there are special circumstances that warrant them, in which case, approval by the **DSD** is required. A **DBE** that prefers slope protection material other than that indicated on the **preliminary plans** , may so indicate with comments on the drawing. These guidelines may be varied somewhat from division to division, depending on preference. When existing slope protection exists, its reuse shall be determined based on the material condition and its suitability to the site.

4.3.1.1 - BRIDGES OVER LOCAL ROADS AND HIGHWAYS

The selection criteria for slope protection used on structures which span over local roads and highways shall be as follows:

1. Concrete Block Paving, 6 inches thick laid on a 3 inch thick sand cushion or Stamped Concrete Slope Protection shall be used for slope protection at abutments for structures crossing over local roads and highways. For the stamped concrete, five stamp pattern options have been selected which may be chosen from as abutment

slope protection profiles. The patterns are Running Bond Used Brick, Flagstone, Herringbone Granite, Basketweave Used Brick, and River Rock. See Details 4.3.1.1.a, 4.3.1.1.b, and 4.3.1.1.c.

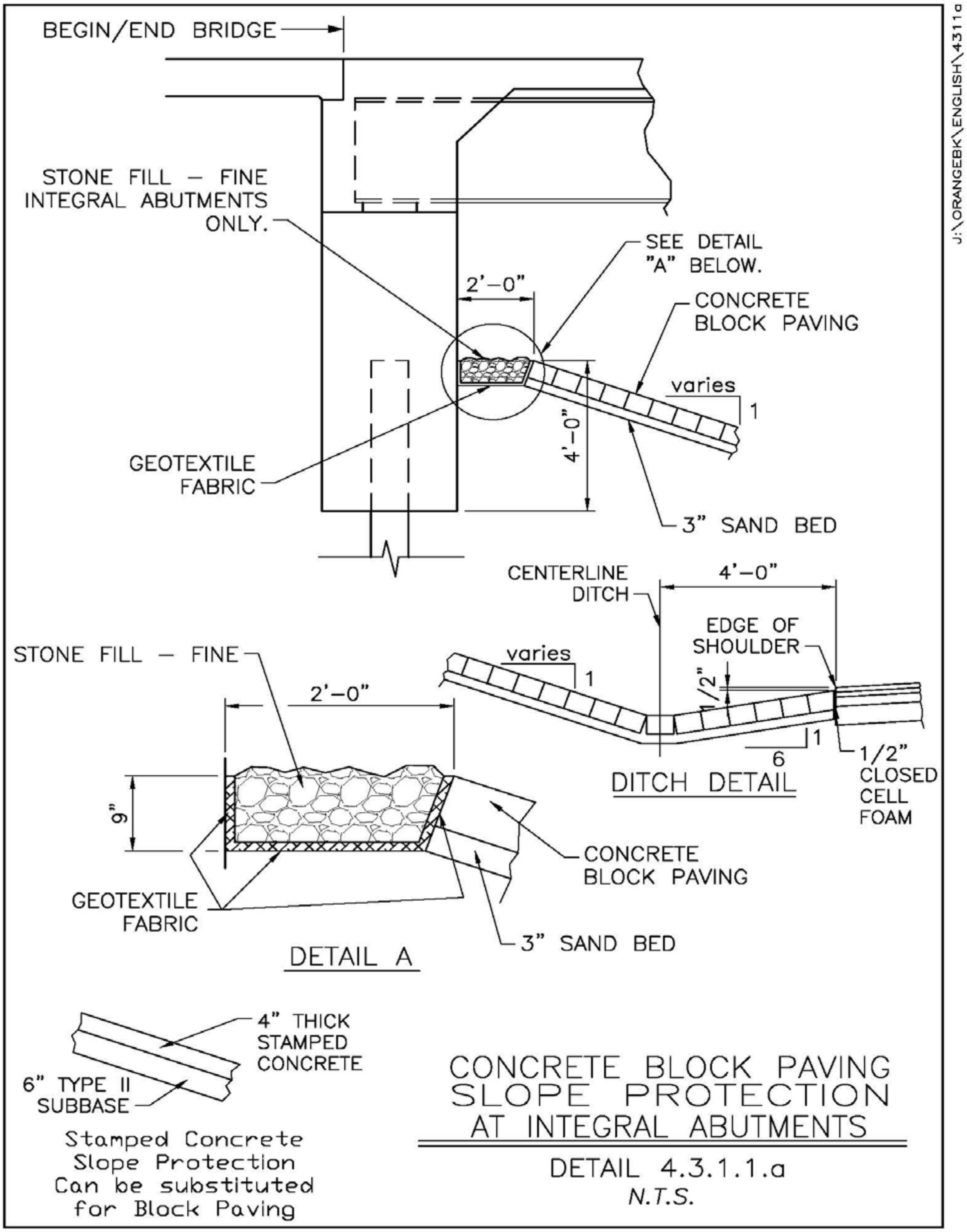
2. Select Granular Fill, Slope Protection (Structures), 8 inches thick, may be used in place of the block paving or stamped concrete when specifically requested by the **DBE**.

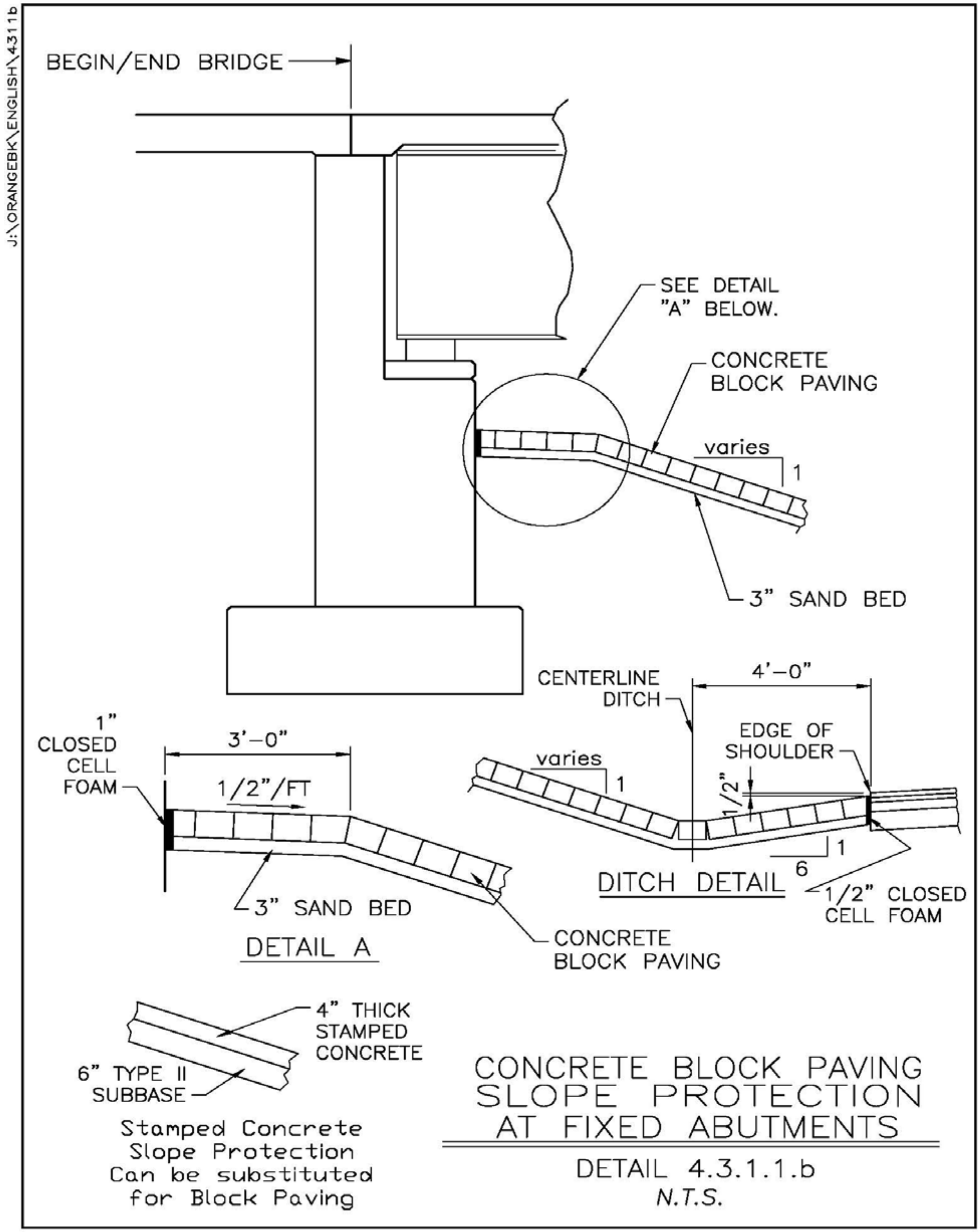
4.3.1.2 - BRIDGES OVER RAILROADS

Select Granular Fill, Slope Protection (Structures), 8 inches thick, shall be used for slope protection at abutments for structures crossing over railroads.

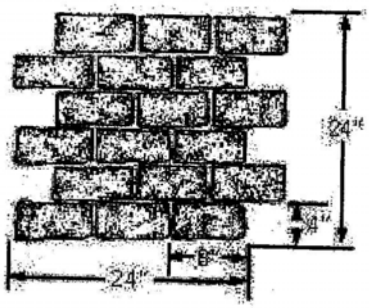
4.3.1.3 - BRIDGES OVER STREAMS (Refer to Subsection 1.3 – Hydrology & Hydraulics)

Stone filling of the type shown in Table 4.3.1.3 and/or as specified in the **HADR** shall be provided to an elevation 1 foot above design high water except in cases of navigable waterways where wave action is a consideration. In these cases, protection is provided to an elevation 3 feet above maximum navigable water elevation. Stone filling shall be extended laterally to protect stream banks disturbed during construction. The new stone fill shall extend at least to the ends of the wingwalls, but in no case less than that required in the **HADR**.

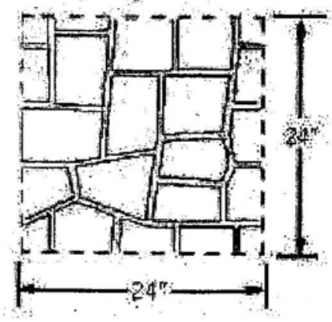

J:\ORANGEBK\ENGLISH\4311a
BEGIN/END BRIDGE
STONE FILL – FINE
INTEGRAL ABUTMENTS
ONLY.
SEE DETAIL
"A" BELOW.
2'–0"
CONCRETE
BLOCK PAVING
varies
1
4'–0"
GEOTEXTILE
FABRIC
3" SAND BED
CENTERLINE
DITCH
4'–0"
EDGE OF
SHOULDER
STONE FILL – FINE
varies
1
1/2"
2'–0"
6
1
1/2"
CLOSED
CELL
FOAM
DITCH DETAIL
9"
CONCRETE
BLOCK PAVING
GEOTEXTILE
FABRIC
3" SAND BED
DETAIL A
4" THICK
STAMPED
CONCRETE
6" TYPE II
SUBBASE
Stamped Concrete
Slope Protection
Can be substituted
for Block Paving
CONCRETE BLOCK PAVING
SLOPE PROTECTION
AT INTEGRAL ABUTMENTS
DETAIL 4.3.1.1.a
N.T.S.


J:\ORANGEBK\ENGLISH\4311b
BEGIN/END BRIDGE
SEE DETAIL "A" BELOW.
CONCRETE BLOCK PAVING
varies
1
3" SAND BED
CENTERLINE DITCH
4'-0"
EDGE OF SHOULDER
1/2"
1" CLOSED CELL FOAM
3'-0"
1/2"/FT
varies
1
1
6
DITCH DETAIL
1/2" CLOSED CELL FOAM
3" SAND BED
DETAIL A
CONCRETE BLOCK PAVING
4" THICK STAMPED CONCRETE
6" TYPE II SUBBASE
Stamped Concrete Slope Protection Can be substituted for Block Paving
CONCRETE BLOCK PAVING SLOPE PROTECTION AT FIXED ABUTMENTS
DETAIL 4.3.1.1.b
N.T.S.

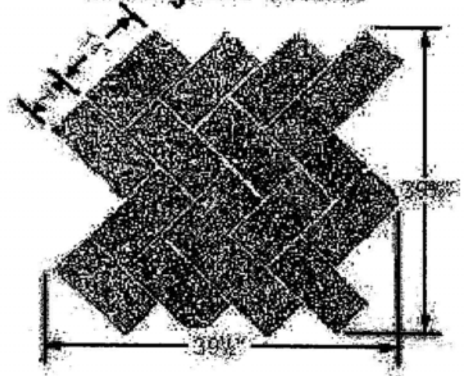
Running Bond Used Brick



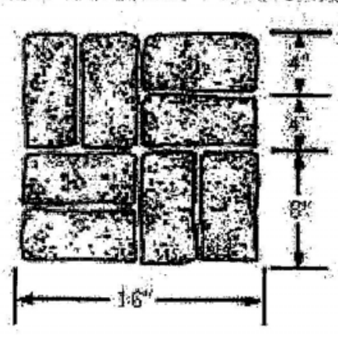
Flagstone



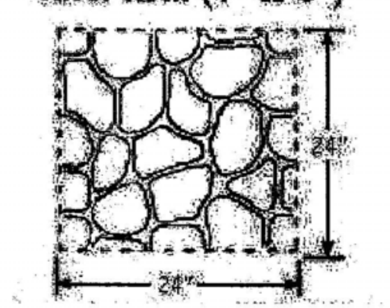
Herringbone Granite



Basketweave Used Brick



River Rock (4" to 6")



DETAIL 4.3.1.1.c

APPROXIMATE STONE FILLING GUIDELINES

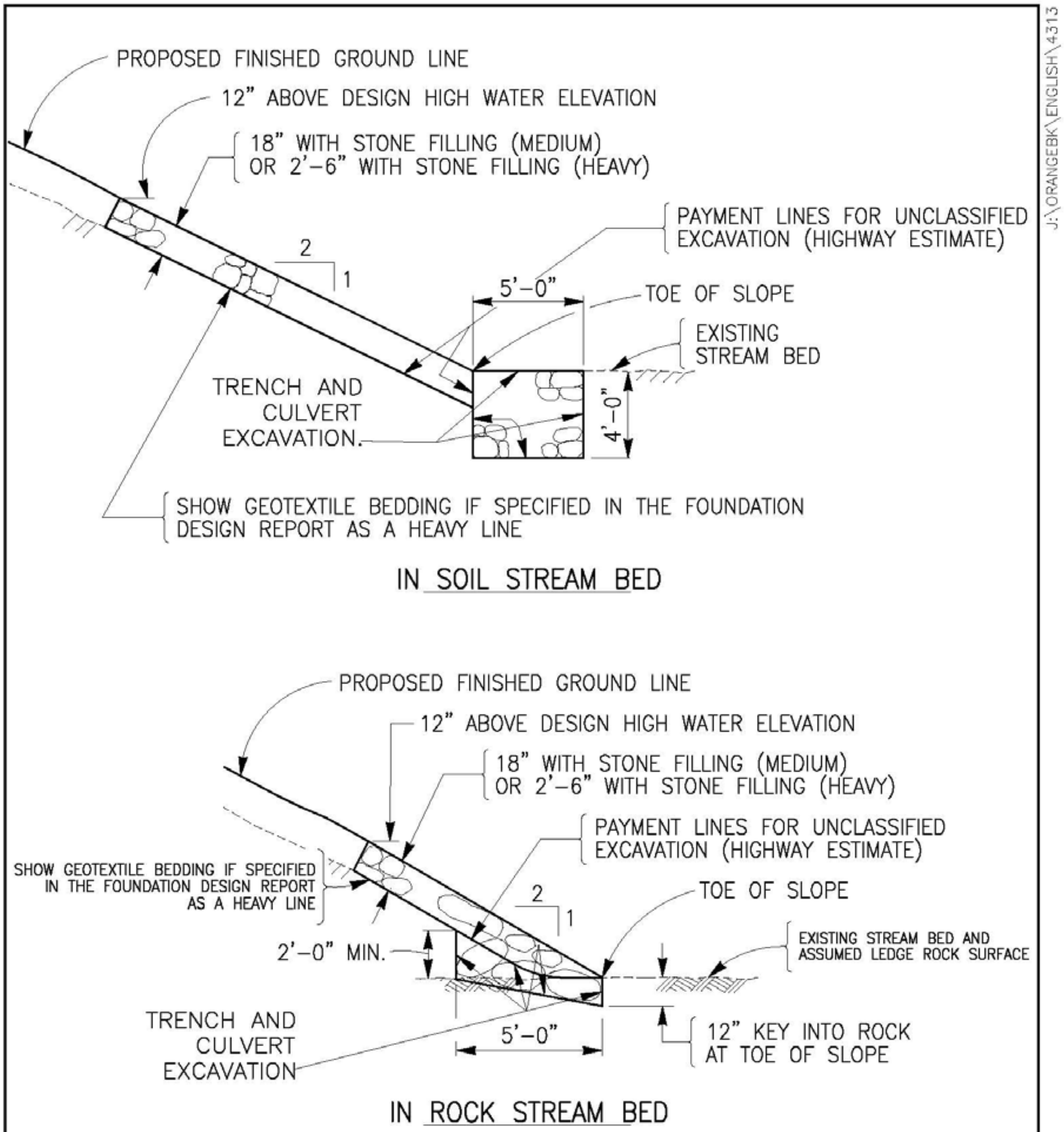
Stream Velocity	Type Stone Filling	Thickness
≤ 10.0 fps	Medium	1.5 feet
> 10.0 fps	Heavy	2.5 feet

Table 4.3.1.3

Stone filling normally will be placed up to the abutment. In cases where there is a considerable distance (10 feet or more) between the required top of stream bank protection (1 foot above design high water) and the abutment, Select Granular Fill, Slope Protection (Structures), 8 inches thick, shall be placed on the intervening area. See Detail 4.3.1.3.

4.3.2 - BERMS

Berms with a 3 foot width are used on most bridges. Berms may be omitted at those bridges where bearing inspection is not required, such as those with integral abutments. The effect of the berm with respect to hydraulic effects should be considered. All berms (paved or not paved) shall have a grade of 8.0% on their surface sloping away from the abutment face.



TYPICAL
SLOPE PROTECTION AT STREAMS
(DIMENSIONS MAY VARY PER HADR)
DETAIL 4.3.1.3

N. T. S.

4.4 - SUBSTRUCTURE MATERIALS

The primary materials used in the construction of substructure elements are Class HP concrete and galvanized bar reinforcement.

4.4.1 - STRUCTURAL CONCRETE ITEM FOR SUBSTRUCTURE CONCRETE

All footing, stem, backwall and pedestal concrete shall be detailed, estimated, and bid under the appropriate HP concrete item (units = cy). For footings on rock, backfill concrete (Class A) placed below the planned bottom of footing elevation shall be paid for under the same item. Concrete shall be designed using a compressive strength of f 'c = 3,000psi.

4.4.2 - GALVANIZED REINFORCEMENT IN SUBSTRUCTURES (See Section 5 – Reinforcement)

Galvanized reinforcement shall be used in all elements of new substructure units in order to protect the reinforcement from corrosion accelerated by chloride saturation into the concrete. Chlorides from roadway salt are dissolved in water draining from the roadway above, as well as in water splashed from the roadway below. Galvanized reinforcement shall also be used in the repair and/or rehabilitation of existing substructure units regardless of the type of existing reinforcement used. Reinforcement shall be **American Society for Testing and Materials (ASTM)** A615 and galvanized in accordance **ASTM** A767. Reinforcement shall be designed for a yield stress of F_y = 60.0ksi and an allowable stress of F_b = 24.0ksi. The minimum tensile reinforcement used in all reinforced concrete members shall be in accordance with the following:

- **ASD** Design – **AASHTO 17th Edition** Subsection 8.17.1.2

- **LFD** Design – **AASHTO 17th Edition** Subsection 8.17.1.2
- **LRFD** Design – **AASHTO LRFD 4th Edition** Subsection 5.7.3.3.2

4.5 - ABUTMENT FEATURES

4.5.1 - GENERAL

This subsection describes the various features of abutment elements. The material cost of the concrete is the cheapest part of the total cost of concrete items. The forming, concrete placement, and labor, constitute the major portion of the cost. Therefore, the shape of the concrete abutment should be made as simple as possible. The shape should be such that large flat forms and large pours may be employed. New abutments shall be designed to conform to **AASHTO** seismic design criteria.

4.5.2 – ABUTMENT STEMS

Abutment stems shall be solid. Refer to Subsection 1.1 – Aesthetics, for information on surface treatments. The stem shall be designed as a cantilever retaining wall resisting soil pressures on the back as described in the **FDR** and loading from the superstructure. The bridge seat may either be continuously sloped with individual pedestals or stepped. The top of bridge seat shall be reinforced with No. 8 bars at 6 inch minimum centers to provide adequate reinforcing for bearing anchor bolts. When individual pedestals are used, the top surface of the bridge seat between pedestals shall have a 4.0% wash toward the front face of the abutment stem. Concrete cover for reinforcing at all locations in abutment stems shall be 3 inches. Abutment stems shall have weep tubes as described in

Subsection 4.2.2. Protruding filleted seats (also known as corbels), which are used at the bridge seat elevation below the backwall to widen the seat without increasing abutment stem thickness should be avoided. The form work for these details is very costly. The concrete quantity savings would only prove economical on taller abutments. They should only be considered on abutment stem heights of 30 feet or more. If the filleted seats are used, the toe and/or heel of the footing should be made wide enough so that if the Contractor elects to pour the wall solid to eliminate the protrusion, the wall will fit on the footing. Also, to facilitate the forming, the distance from the top of the footing to the bottom of the protruding fillets should be made constant, rather than having the fillets parallel to the bridge seat. The height variation can be made between the fillet and top of the header.

4.5.2.1 - LOCATION OF PEDESTALS ON ABUTMENTS

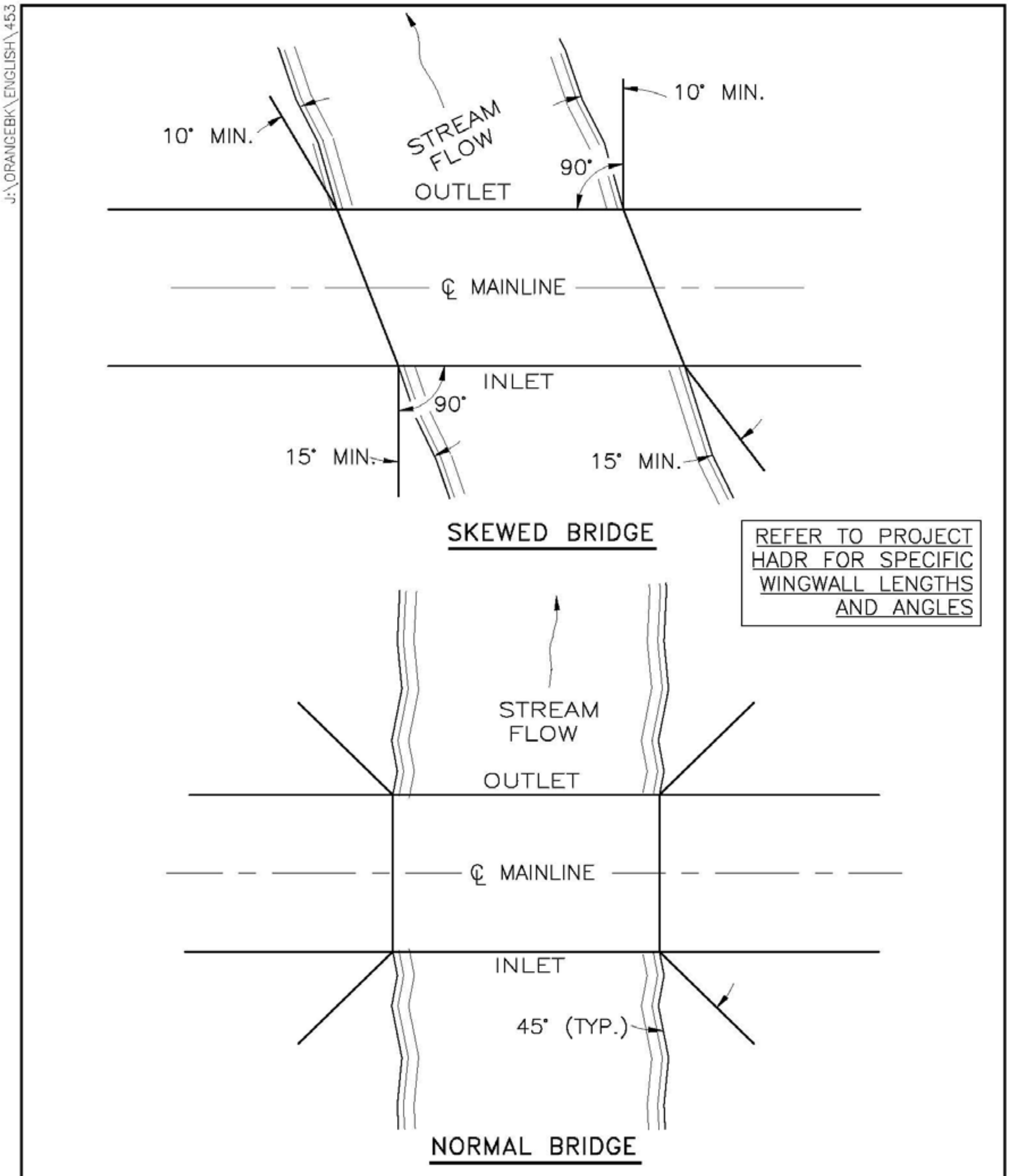
On all abutments with pedestals on a bridge seat, the front face of the pedestal shall be flush with the front face of the bridge seat. Pedestal height shall be between 6 inches and 18 inches. The top surface under the bearing device shall be level. The remaining top surface shall have 2.0% wash toward the front and away from the bearing.

4.5.3 - CANTILEVERED WINGWALLS (IN-LINE AND FLARED)

Cantilevered wingwalls are retaining walls rigidly connected to, and supported by, the abutment stem. They have no foundation below the wingwall stem. Because of this arrangement, the length of these walls is limited. These walls shall be designed horizontally due to the bending at the abutment interface and vertically due to shear at the abutment foundation outside piles, spread footing, or fascia girder, depending on the abutment type used. The actual length limitation will vary depending on the abutment foundation capacity, the soil pressures on the back of the wall, the thickness of the wall, and the amount of reinforcing in the wall. In-line wingwalls are preferred where practical. In cases where the wingwalls are subject to hydraulic effects, flared wingwalls are the preferred option. The length and angle of the wingwalls shall be as specified on the **HADR**. Detail 4.5.3 shows the typical configuration for wingwalls on a skewed waterway crossing and a normal waterway crossing. Flared wingwalls may also be used on highway or railroad crossings in an effort to reduce the height and length of the walls.

4.5.4 – FREE-STANDING WINGWALLS (IN-LINE AND FLARED)

Free-standing wingwalls are retaining walls vertically cantilevered off an independent foundation (spread footing or piles with cap as specified in the **FDR**). They may or may not be rigidly connected to the abutment stem depending on the abutment type used. These wingwalls shall be designed as cantilevered retaining walls resisting a 1 on 2 slope and surcharge in most cases. Design of the wall will vary from end-to-end if the wingwall height varies significantly. Tapering the wingwall thickness may be an economical option in this case if the wingwall is long. The quantity of concrete savings resulting from tapering longer walls should be considered. If the wingwall is rigidly connected to the abutment the designer may design for 67% of the maximum height of wall. In this case the wingwall stem will be a constant thickness. In-line wingwalls are preferred where practical. In cases where the wingwalls are subject to hydraulic effects, flared wingwalls are the preferred option.



EXAMPLE WINGWALL LAYOUT AT STREAMS
DETAIL 4.5.3

N.T.S.

The length and angle of the wingwalls shall be as specified on the **HADR**. Detail 4.5.3 shows the typical configuration for wingwalls on a skewed waterway crossing and a normal waterway crossing. Flared wingwalls may also be used on highway or railroad crossings in an effort to reduce the height and length of the walls.

4.5.5 – BATTERED and STEPPED WINGWALLS

A battered wingwall is a free-standing wingwall with a battered face. Battering is done on taller walls to reduce concrete quantities. The base of the wall is made thicker where it is needed and then tapers up to a narrower top. Battered forms are more expensive than vertical forms and should be avoided whenever possible, especially on short wingwalls. Stepping to vary a wall stem thickness is always preferable to battering.

If battered forms are used, the batter should always remain constant, and the width of the wall at the top of the batter should be wide enough so the form can be extended beyond the top of the batter and still have enough room between the front and rear forms to easily place the concrete. Batters that extend partially up a wall should be avoided. If partial batters are used, the height of the battered portion should always be made a constant height. If the height of the wall varies, the height of the battered portion should be constant with respect to the top of the footing, and the variation in height should be made up in the upper vertical portion of the wall. This will allow the battered forms to be reused and so reduce the unit cost of the concrete.

4.5.6 - U-WALL WINGWALLS

A U-wall wingwall is a free-standing wingwall positioned parallel to the bridge roadway extending from the abutment stem back into the approach fill. They should be used only when **R.O.W.** is limited in the area of the abutment or to prevent approach fill from spilling into a stream or wetland.

4.5.7 - CURVED WINGWALLS

A curved wingwall is a free-standing wingwall with a horizontally curved outside face. Several existing curved wingwalls can be seen on the Thruway. Their use was primarily for aesthetic reasons. Curved wingwalls are very expensive to build due to the complicated forming and reinforcement placement issues. They should not be used on any new structures. When replacing existing curved wingwalls it is best to place a widened footing and wall on a chord and curve only the outside face of the wall where possible.

4.5.8 – OTHER WINGWALL TYPES

In lieu of utilizing a poured concrete retaining wall, the Designer may select from the retaining wall types listed in the **NYSDOT** Bridge Manual – Section 11.4. Included in this section is a description of the wall type as well as effective height ranges that the walls are typically used for. The Designer should request the assistance of the Authority's Geotechnical Engineer if any of these alternate wingwall types are being considered. The most common alternate wingwall types used for highway structures are **Mechanically Stabilized Earth Systems (MSES)** and **precast concrete modular wall systems**.

DOT's standard specification (Section 554) may be used for **MSES** wingwalls and abutments. Further guidance on the use of **MSES** abutments can be found in the **NYSDOT** Bridge Manual – Section 11.5.1.4. The Designer shall ensure that a seismic analysis for the proposed wall (or abutment) is performed by the wall manufacturer. **MSES** retaining walls shall be designed for a design life of 75 years. **MSES** abutments shall be designed for a design life of 100 years. These requirements shall be specifically indicated on the plans. The Thruway has developed a special specification for **precast concrete modular wall systems**. This special specification differs from DOT's standard specification (Section 632) by limiting substitutions of new wall systems to the systems that have been reviewed, approved and used successfully by DOT. Currently, these systems are Sta-Wal, T-Wall, and Doublewal. See the Thruway Standard Sheets for appropriate details.

4.6 - INTEGRAL ABUTMENTS

The integral abutment consists of a concrete stem cast around, and supported by, a single row of piles. The superstructure slab is cast monolithically with the top of stem encasing the girder ends in concrete. Since the abutment is supported on a single line of piles, a concrete footing is not required. One of the primary advantages of integral abutments is the elimination of the bridge deck expansion joints, thereby reducing construction and maintenance costs. The integral abutment bridge concept is based on the theory that due to the flexibility of the piling below the bottom of the abutment stem, thermal stresses are transferred to the foundation by way of a rigid connection between the superstructure and substructure. The concrete abutment contains sufficient bulk to be considered a rigid mass. A positive connection with the ends of the beams or girders is created by their encasement in reinforced concrete at the top of stem. This connection provides for load transfer from the superstructure into the abutment stem and into the abutment piling.

4.6.1 - GUIDELINES ON USE

Integral Abutments are the preferred abutment type on Thruway projects because of the elimination of bridge deck expansion joints and bearings. The criteria for the use of integral abutments include limitations on the expansion span lengths, site geometry, site conditions, and the existing soil conditions. The use of integral abutment bridges will only be limited by the constraints described in the following subsections:

4.6.1.1 - EXPANSION LENGTHS

The movement of an integral abutment is largely attributed to thermal expansion and contraction of the superstructure. The longer the expansion span length, the larger the longitudinal movement (and rotation on taller stems) of the abutment. The expansion span length of an integral abutment structure is equal to half the abutment centerline to abutment centerline dimension for single span structures, and the abutment centerline to fixed pier centerline dimension for multi-span structures. As the abutment pushes against the backfill during expansion, it is loaded horizontally by the passive resistance of the backfill (passive earth pressure) or the compressive resistance of the selected compressible inclusion material. Compressible inclusion material should meet the requirements of **ASTM** C578. The larger the longitudinal movement of the abutment, the higher the passive resistance. In order to keep these pressures reasonable, the Thruway Authority has established expansion span length limitations. Expansion lengths up to 300 feet may be used without restriction. Expansion lengths between 300 feet and 400 feet shall only be used with approval from the **DSD**. Integral abutments shall not be used when expansion lengths exceed 400 feet.

4.6.1.2 – SOIL CONDITIONS

All integral abutments shall be supported on piles. All piles require sufficient depth of penetration, 12 feet minimum (below preaugered holes described below) into acceptable soil layers. The purpose of this is to avoid a stilt effect (foundation rotation about the bottom of the piles). Additional length may be required as specified in the **FDR** to provide sufficient vertical and/or lateral support for the pile and scour protection.

4.6.1.3 - HORIZONTAL ALIGNMENT

Only straight beams will be allowed. Curved superstructures will be allowed provided the beams are straight and continuous between the abutments. Curved steel beams were eliminated to guard against the possibility of bottom flange and web buckling caused by the beams trying to expand between the restrained abutments. All beams shall be parallel to each other. The abutments and any intermediate piers shall also be parallel to each other.

4.6.1.4 - GRADE

The maximum local vertical curve gradient between abutments shall be 5%.The maximum straight grade allowed on integral abutment bridges is 10%.

4.6.1.5 - SKEW ANGLE

There is no skew limitation on integral abutment bridges. However, the effects of skew must be analyzed and accounted for in the design of all of the structural components.

4.6.1.6 - UTILITIES **(Refer to Subsection 1.6 - UTILITY COMMUNICATION AND COORDINATION)**

Rigid utility conduits, such as gas, water and sewer, cannot pass through integral abutments. The anticipated longitudinal movement of the superstructure and resultant rotational and translational movement of the substructure make provision for these movements in rigid conduits difficult. Conduits of this type should be located off the integral abutment. Flexible conduits for electrical, telephone or cable TV utilities that are properly sleeved through the integral abutment are acceptable.

4.6.2 – DESIGN AND DETAIL CONSIDERATIONS

4.6.2.1 - FOUNDATION TYPES

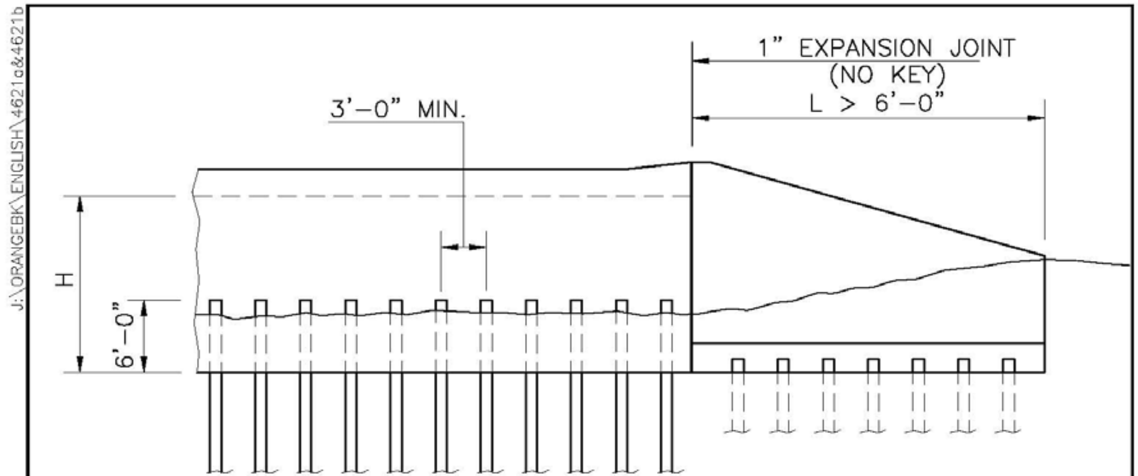
All integral abutments shall be supported on piles. Steel H or **CIP** piles shall be used as recommended in the **FDR**. All piles shall be in one single line. When steel H-piles are used, upon initial sizing recommendation from the **FDR**, the designer shall verify the size and orientation using the Rational Design Approach for Integral Abutment Bridge Piles. Guidelines for this design process can be found in the Structures Library in ProjectWise. If the pile size indicated in the **FDR** must be changed, the Geotechnical Engineer shall be informed, and a supplemental **FDR** will be issued. All piles shall have sufficient depth of penetration, 12 feet minimum into acceptable soil layers (below the preaugered holes described below). The exact length required shall be specified in the **FDR**. A complete **HADR** (that includes a depth of scour analysis) of the site and proposed structure is required where a structure crosses over a waterway. For these structures, the piles are designed to gain all of their capacity below the scour elevation. The **CIP** Piles or Steel Bearing H Piles at each abutment shall be inserted in preaugered holes of a diameter as determined by Table 4.6.2.1.

INTEGRAL ABUTMENT
PRE-AUGURED PILE HOLE DIAMETER CRITERIA

PILE SIZE	PILE TYPE	HOLE DIAMETER
12 inches	CIP	20 inches
14 inches	CIP	22 inches
10 inches	HP	20 inches
12 inches	HP	22 inches
13 inches	HP	23 inches
14 inches	HP	24 inches

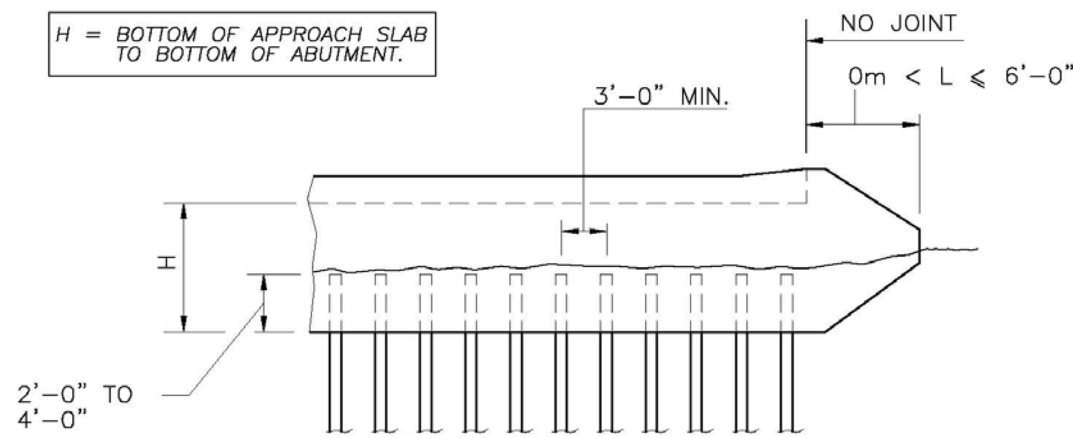
TABLE 4.6.2.1

These holes shall extend to a depth of 8 feet below the bottom of abutment stem. It shall be noted on the plans that it is the Contractor's responsibility to keep the preaugered hole open during pile driving operations so that cushion sand can be placed around each pile after driving. The cost of auguring these holes, casing, and cushion sand shall be included in the unit price bid for the pile Item. For the typical integral abutment detail, the stem will be designed as a continuous beam. In some cases (i.e. small steel girder spans), a stem using a minimum of one pile per girder could be considered. See Details 4.6.2.1.a and 4.6.2.1.b.



INTEGRAL ABUTMENT
TALL ABUTMENT (H > 10'–0")
TYPICAL ELEVATION W/INDEPENDENT
WINGWALLS ON SPREAD FOOTINGS OR PILES

DETAIL 4.6.2.1.a
N.T.S.



INTEGRAL ABUTMENT
SHORT ABUTMENT (H ⩽ 10'–0")
TYPICAL ELEVATION W/CANTILEVERED
WINGWALLS (NO FOOTINGS)

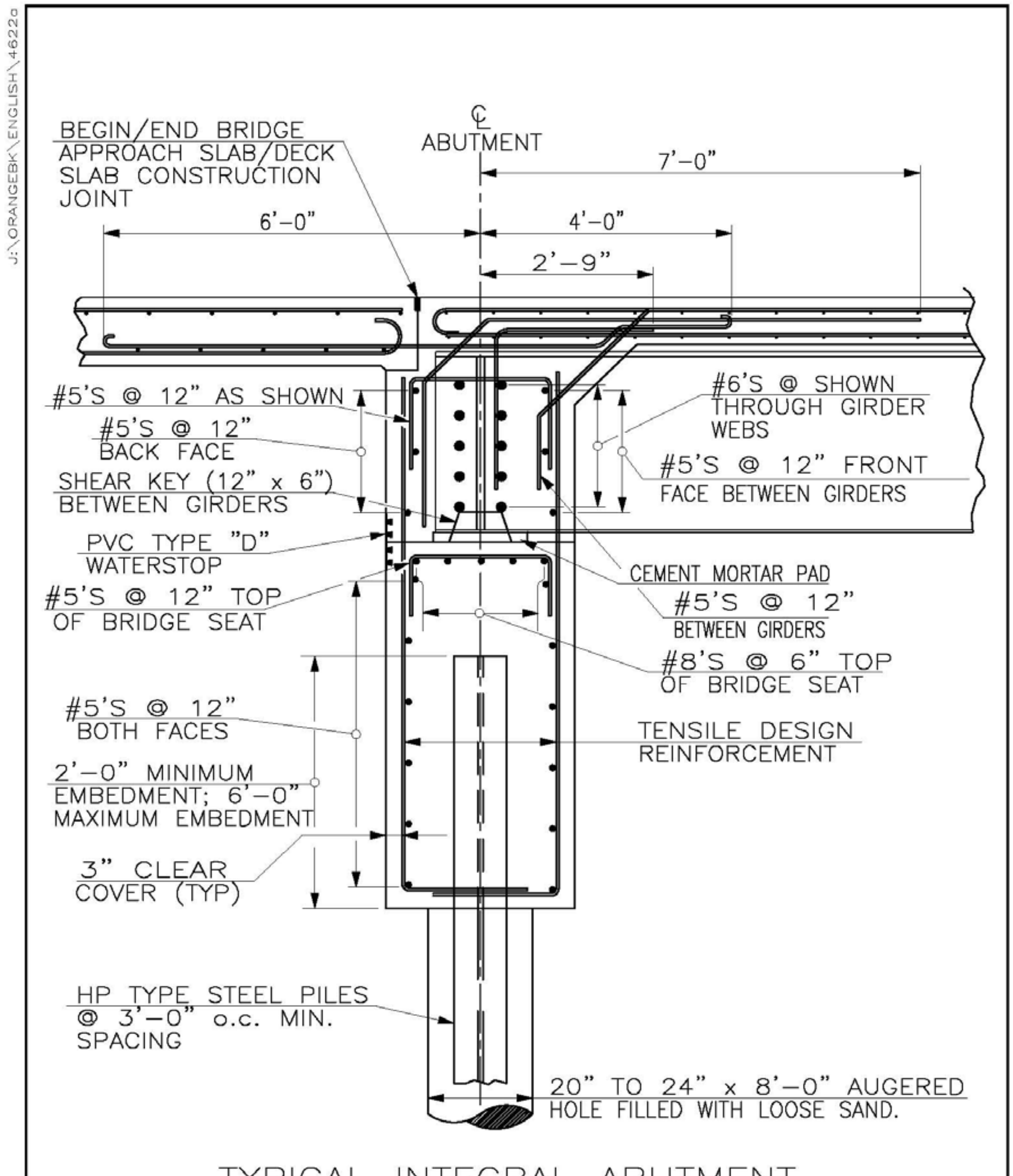
DETAIL 4.6.2.1.b
N.T.S

4.6.2.2 - ABUTMENT STEM

A minimum thickness of 3 feet shall be required for integral abutment stems unless otherwise noted. The stem is designed as a reinforced wall or column fixed at the top and pinned at the bottom. The stem shall be designed to resist the moments induced from the **Superimposed Dead Load (SDL)** and **Live Load (LL)** from the superstructure, thermal expansion, soil loading, and construction loading. See Details 4.6.2.2.a, 4.6.2.2.b, 4.6.2.6.a, 4.6.2.6.b, and 4.6.3.1.

4.6.2.3 - WINGWALLS

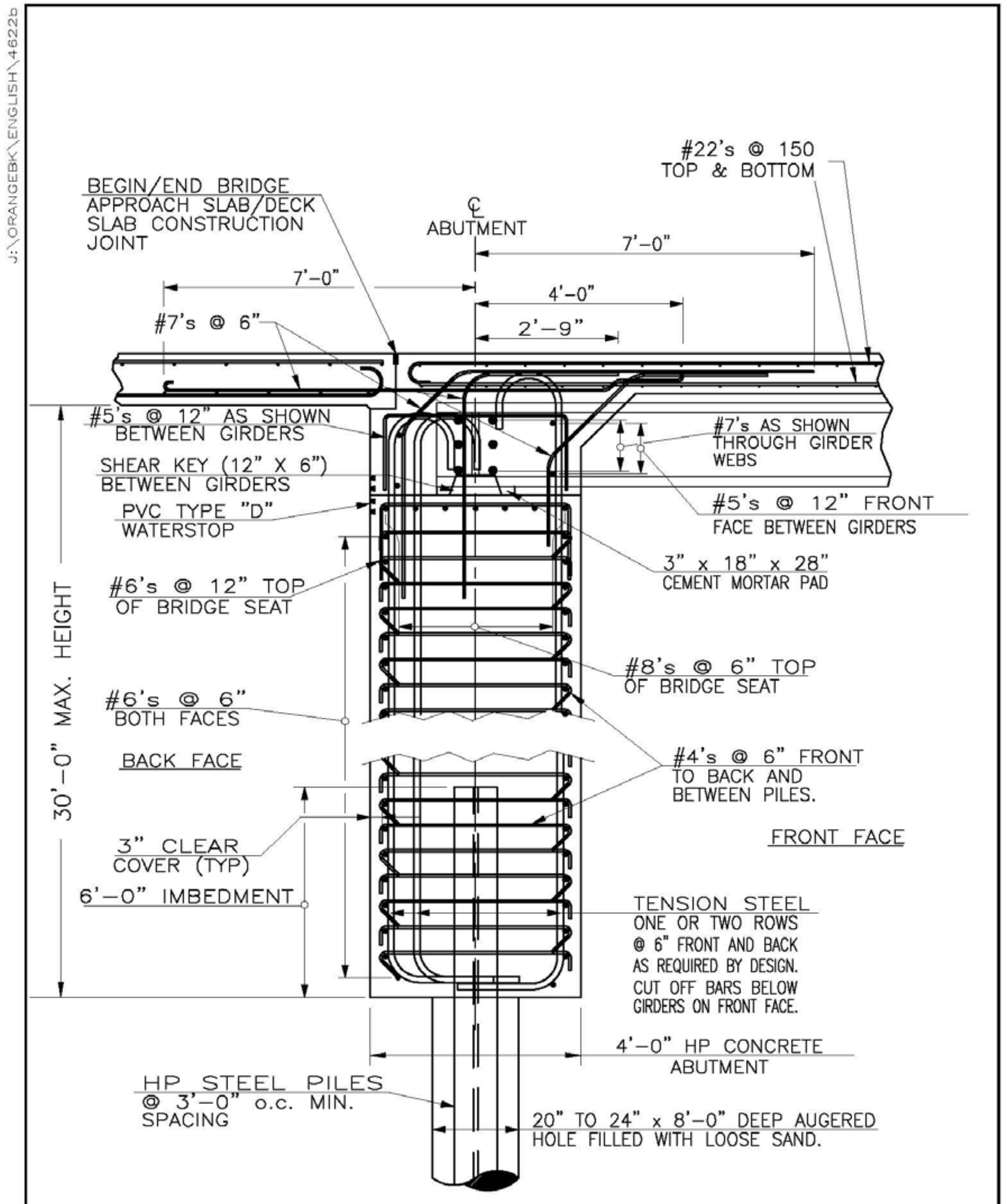
Wingwalls shall be a minimum of 18 inches thick. Wingwalls shall have a constant thickness or be tapered depending on height and length. Tapering the thickness of larger walls from end to end may result in significant concrete quantity savings. In-line wingwalls are the preferred arrangement for integral abutments. Flared walls shall be used at stream crossings and taller abutments where wingwall length may be significantly reduced by flaring. U-walls shall not be used on integral abutment bridges. U-walls are not allowed because they prevent the abutment from moving freely during thermal changes. Wingwalls 6 feet or less in length shall be cantilevered off the integral abutment. See Subsection 4.5.3 and Details 4.6.2.1.b & 4.6.2.3. Wingwalls greater than 6 feet in length shall be self supported on footings or piles and separated from the abutment with a multi-directional (keyless) expansion joint. See Subsection 4.5.4 and Detail 4.6.2.1.a. This joint allows the abutment to deflect under thermal forces without inducing stresses into the wingwall. This joint shall consist of a layer of closed cell foam between the abutment and wingwall and a PVC water stop at the back of the joint. See details in Appendix C.



TYPICAL INTEGRAL ABUTMENT SECTION W/STEEL SUPERSTRUCTURE

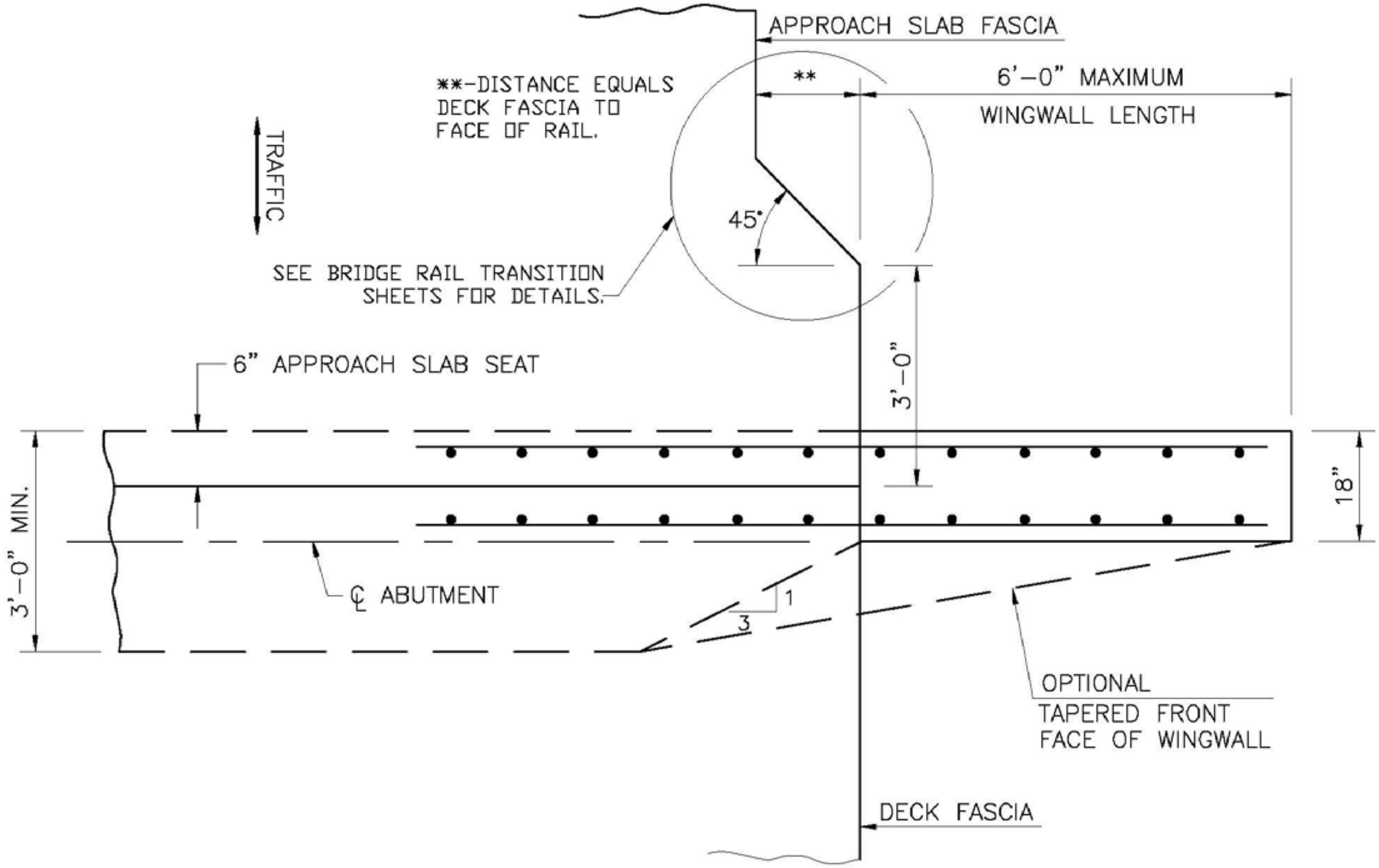
DETAIL 4.6.2.2.a

N.T.S.



RIGID FRAME INTEGRAL ABUTMENT
SECTION W/ STEEL SUPERSTRUCTURE

DETAIL 4.6.2.2.b
N.T.S.



CANTILEVERED WINGWALL DETAIL

INTEGRAL ABUTMENT – PLAN VIEW

DETAIL 4.6.2.3

N.T.S.

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4.6.2.4 - SUPERSTRUCTURE TYPE

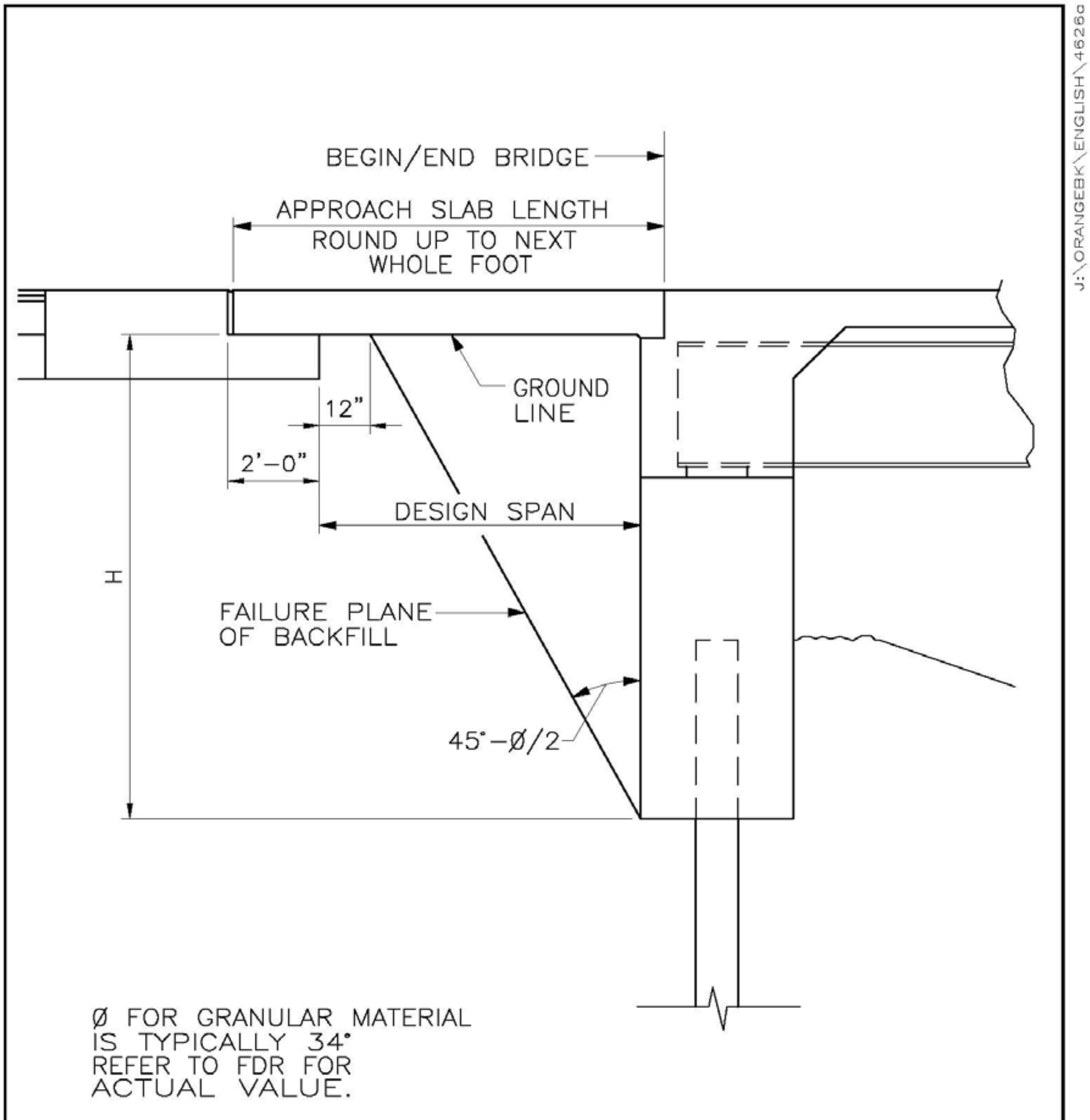
Steel beams or plate girders may be used on both conventional integral abutment bridges and rigid frame integral abutment bridges. Prestressed concrete beams shall only be used on conventional integral abutment bridges. The deck may be either cast-in-place or precast. Refer to Section 3 – Decks, for more information. The design of the superstructure will vary depending on whether the bridge is a conventional integral abutment type structure or a rigid frame integral abutment type structure. Refer to Subsection 4.6.3 – Integral Abutment Bridge Design Procedures for details.

4.6.2.5 - BEARINGS

When steel beams or plate girders are used in the superstructure a rectangular mortar pad will be required under the beams on the abutment bridge seat. The Steel beams will be connected to the abutment with reinforcing steel running horizontally through the web. See Detail 4.6.2.2.a. In addition, on rigid frame structures, the front face abutment vertical reinforcing shall continue up through the bottom flange of each beam. See Detail 4.6.2.2.b. Prestressed beams require individual rectangular plain rubber bearing pads placed perpendicular to the centerline of the beam. Prestressed beams shall be connected to the abutment with anchor rods and reinforcing steel.

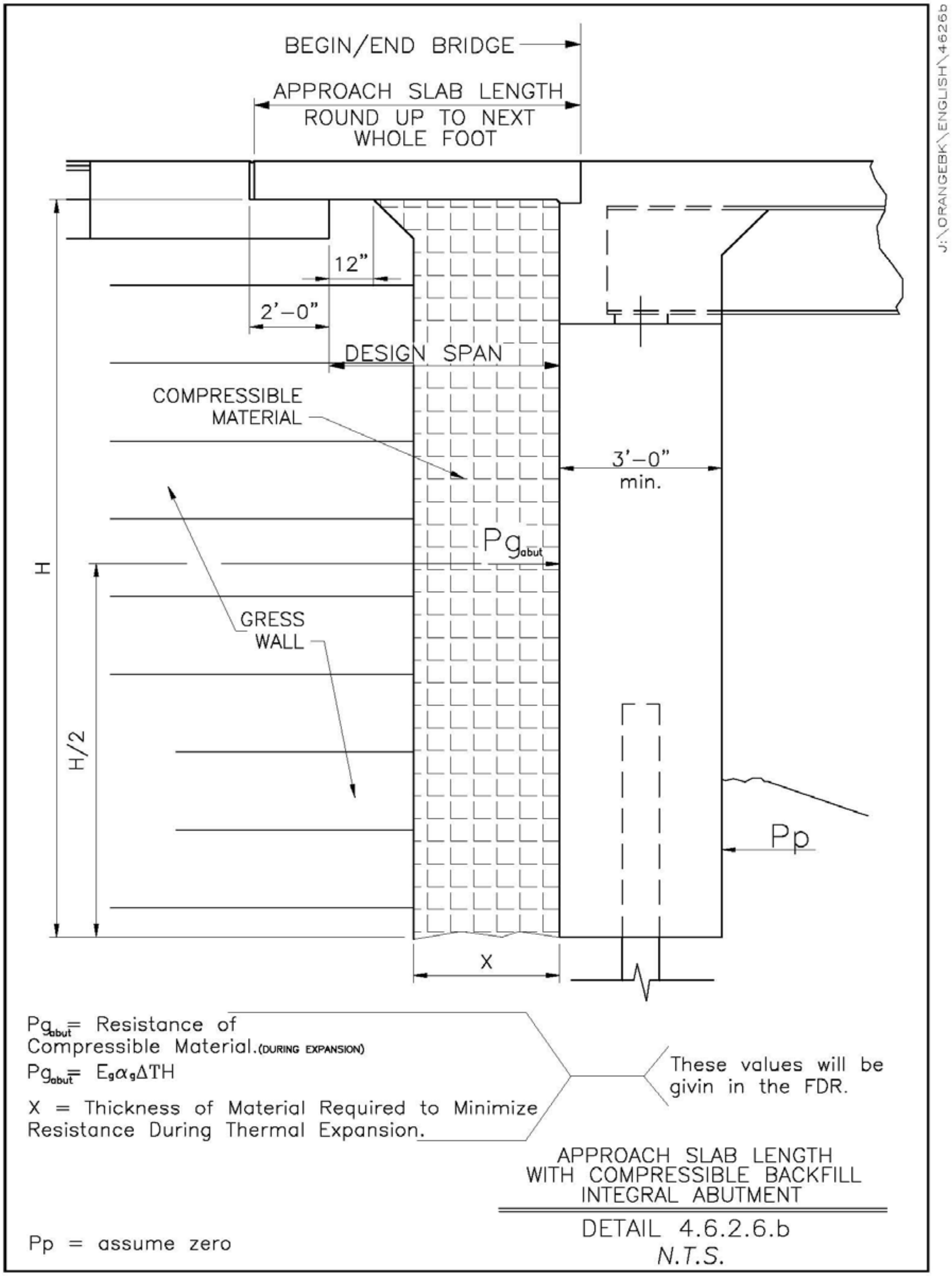
4.6.2.6 - APPROACH SLABS

Approach slabs are required for all integral abutments. The purpose of the approach slab is to bridge the fill directly behind the abutment and provide a transition from the approach pavement to the bridge deck. Approach slab thickness shall be a minimum of 12 inches. Thickness may be greater depending on the design span length of the approach slab. Approach slab lengths vary depending on the height of the abutment and backfill treatment. In most cases the length is determined based on the intercept of the backfill active failure plane from the bottom of the abutment stem to the bottom of the approach slab (ground line). See Detail 4.6.2.6.a. On taller integral abutments, where the backfill is supported independently with a **GRES** wall, the length will be determined as shown on Detail 4.6.2.6.b. The end of the approach slab shall be supported on a sleeper slab. This end of the approach slab shall be perpendicular to the centerline of the roadway and run from face-of-guiderail to face-of-guiderail. The abutment end of the approach slab shall be rigidly connected to the abutment as shown in Details 4.6.2.2.a and 4.6.2.2.b. Polyethylene curing covers shall be placed on top of the subbase prior to pouring the approach slab. This sheet will aid in allowing free thermal movement of the approach slab on the subbase material. The approach slab shall incorporate both top and bottom steel reinforcement. The top mats (transverse and longitudinal) of reinforcement shall be a minimum of #5 Bars @ 12 inch spacing in both directions. The bottom mat longitudinal reinforcement shall be designed for traffic loading (reinforcing parallel to traffic) with the design span being a simple span from the back face of the abutment to 1 foot beyond the intersection with the failure plane. The bottom mat transverse reinforcement shall be for temperature only. Refer to **AASHTO 17 th Edition** Subsection 3.24.3.2 for the minimum requirements when the main reinforcement is parallel to traffic.



APPROACH SLAB LENGTH
INTEGRAL ABUTMENT

DETAIL 4.6.2.6.a
N.T.S.


BEGIN/END BRIDGE
APPROACH SLAB LENGTH
ROUND UP TO NEXT WHOLE FOOT
12"
2'–0"
DESIGN SPAN
COMPRESSIBLE MATERIAL
3'–0" min.
Pg_{abut}
H
H/2
GRESS WALL
Pp
X
Pg_{abut} = Resistance of Compressible Material. (DURING EXPANSION)
Pg_{abut} = $E_g \alpha_g \Delta TH$
X = Thickness of Material Required to Minimize Resistance During Thermal Expansion.
These values will be givin in the FDR.
APPROACH SLAB LENGTH
WITH COMPRESSIBLE BACKFILL
INTEGRAL ABUTMENT
DETAIL 4.6.2.6.b
N.T.S.
Pp = assume zero
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4.6.2.7 - SLEEPER SLABS

The sleeper slab is a buried concrete foundation used to support the free end of the approach slab. The end of the approach slab slides on the end of the sleeper slab. Sleeper slab reinforcement shall be as shown in Details 4.6.2.7.a through 4.6.2.7.d.

4.6.2.8 - JOINTS

A cold formed construction joint should be located between the approach slab and the abutment as described in Subsection 4.6.2.6, at a distance of 6 inches from the back face of the abutment. This joint will provide a controlled crack rather than allowing a random crack to develop in the roadway surface. This joint shall also define the beginning and end of bridge stationing. See Details 4.6.2.2.a and 4.6.2.2.b. Galvanized reinforcing steel shall connect the approach slab to the abutment. This reinforcement provides a positive connection between the two to keep the joint tight. This joint **must** be cold formed. The joint will be sawn and sealed as described in the Approach Slab Notes in Appendix B.

An expansion joint shall be placed at the end of the approach slab between the approach slab and the sleeper slab. The purpose of this joint is to allow for the thermal movement of the integral abutment and approach slab. This is a working joint that opens and closes due to thermal expansion and contraction. The longer the span, the greater the opening and closing. The size of the joint opening shall be indicated on the plans.

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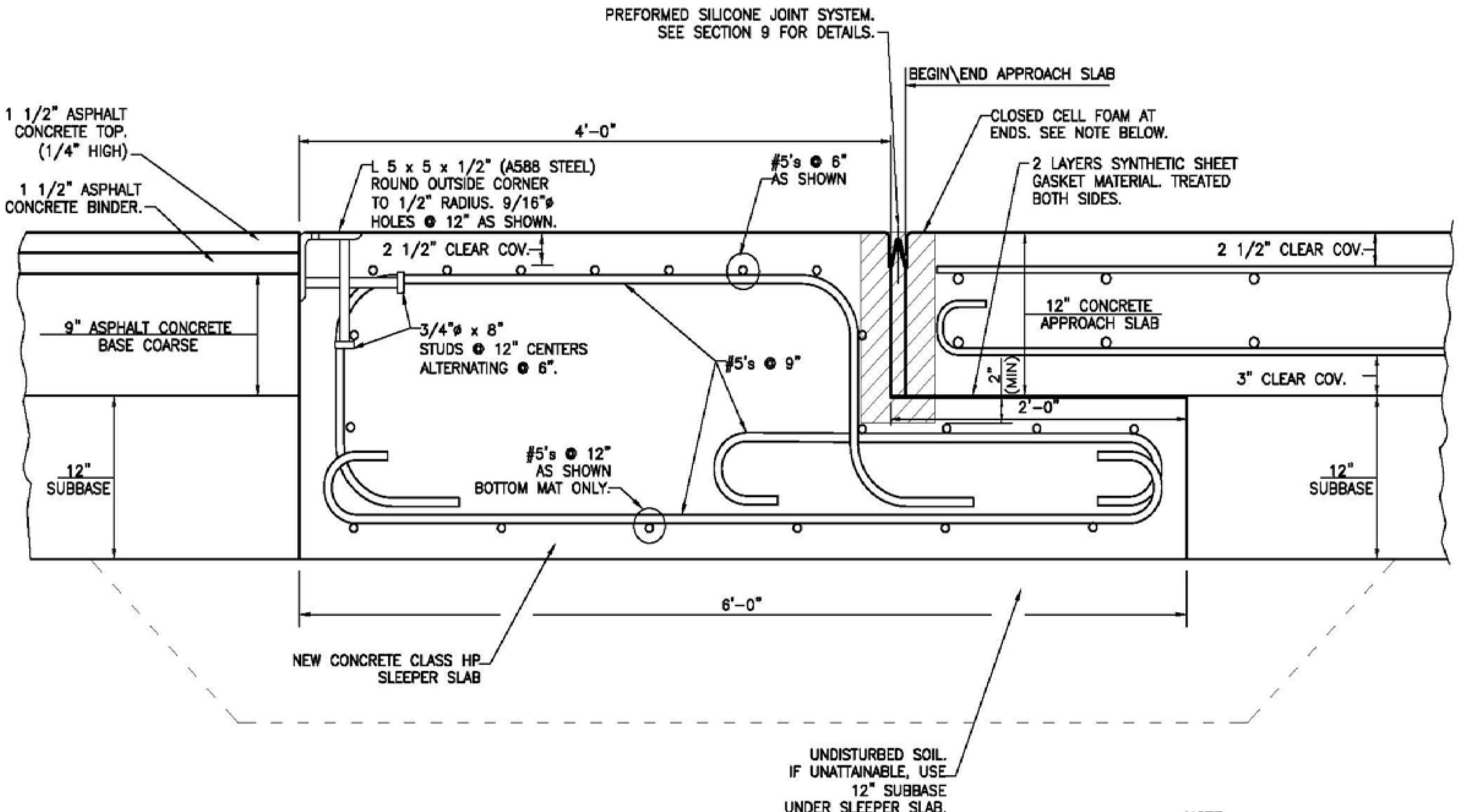

APPROACH SLAB END DETAIL – EXPANSION END.

ASPHALT CONCRETE APPROACH ROADWAY
DETAIL 4.6.2.7.a
N.T.S.

NOTE:
1" x 6" CLOSED CELL FOAM 14" LONG SHALL BE PLACED AS SHOWN AND BONDED TO THE CONCRETE WITH AN APPROVED ADHESIVE AT THE ENDS OF THE JOINT TO PREVENT BACKFILLING OF THE VOID UNDER THE JOINT.

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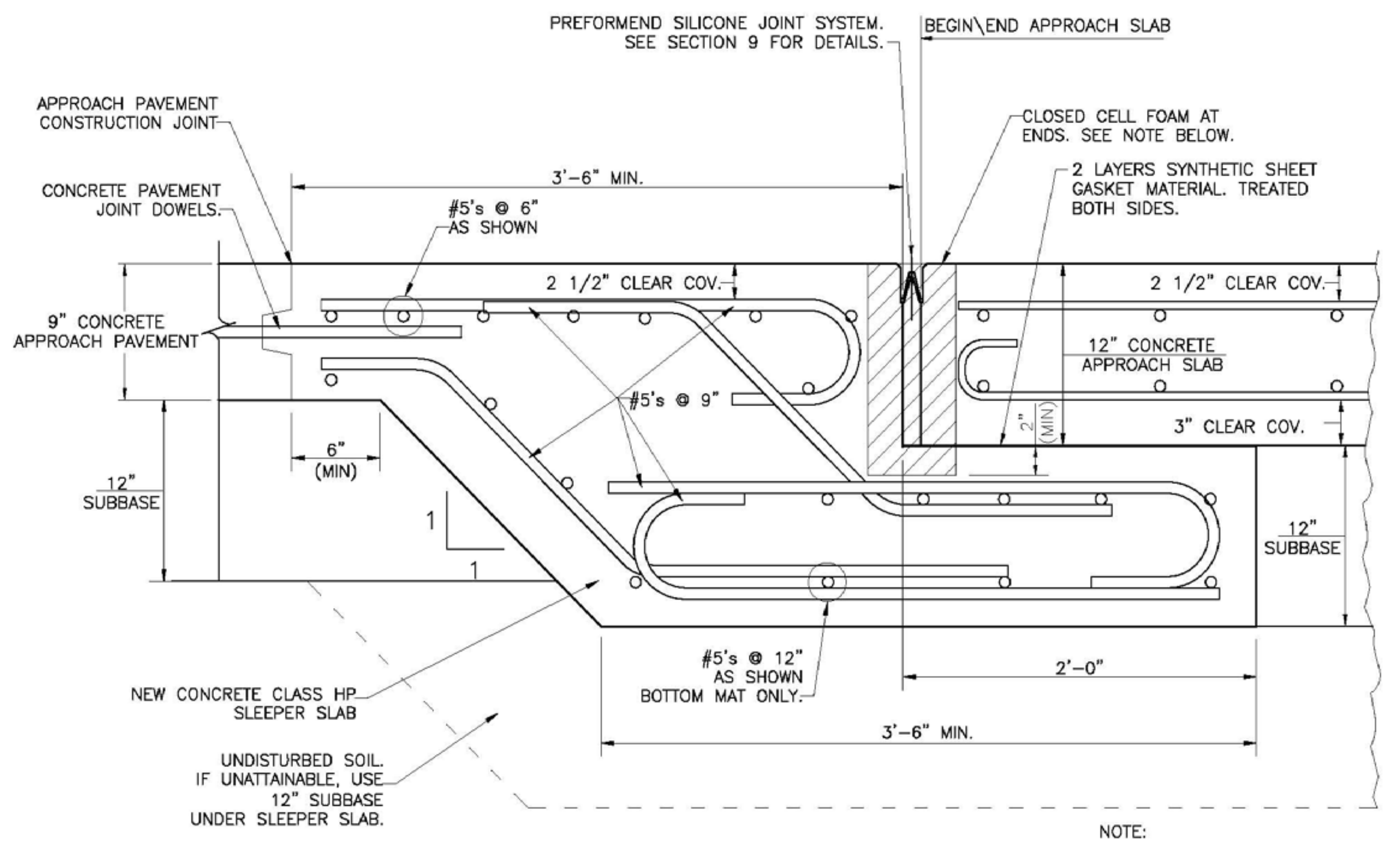

APPROACH SLAB END DETAIL – EXPANSION END.

CEMENT CONCRETE APPROACH ROADWAY
DETAIL 4.6.2.7.b
N.T.S.

NOTE:
1" x 6" CLOSED
CELL FOAM 14" LONG
SHALL BE PLACED AS SHOWN
AND BONDED TO THE CONCRETE
WITH AN APPROVED ADHESIVE
AT THE ENDS OF THE JOINT TO
PREVENT BACKFILLING OF
THE VOID UNDER THE JOINT.

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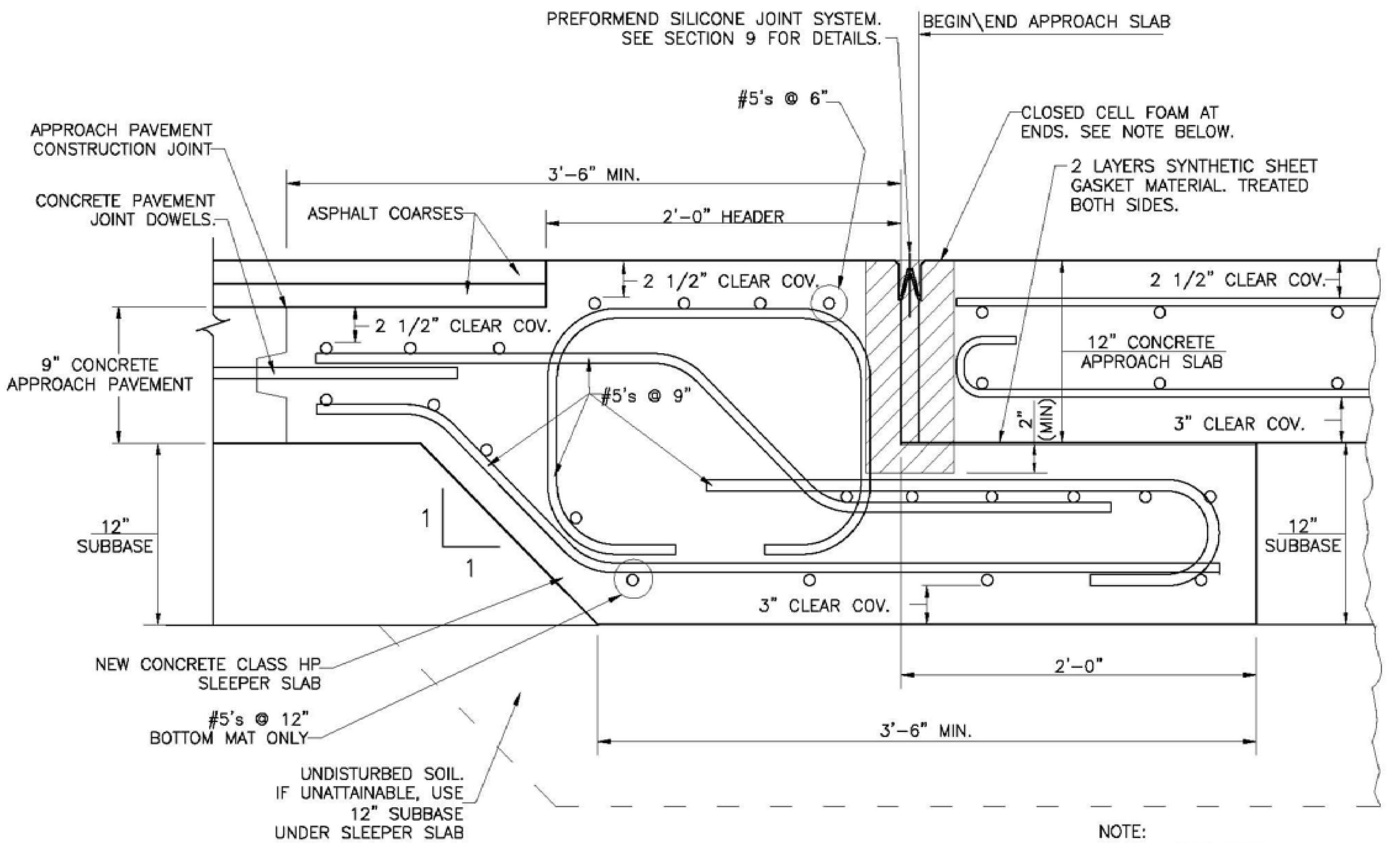

NOTE:
1" x 6" CLOSED CELL FOAM 14" LONG SHALL BE PLACED AS SHOWN AND BONDED TO THE CONCRETE WITH AN APPROVED ADHESIVE AT THE ENDS OF THE JOINT TO PREVENT BACKFILLING OF THE VOID UNDER THE JOINT.

APPROACH SLAB END DETAIL – EXPANSION END.

CEMENT CONCRETE APPROACH ROADWAY/ASPHALT WEARING SURFACE
DETAIL 4.6.2.7.c
N.T.S.

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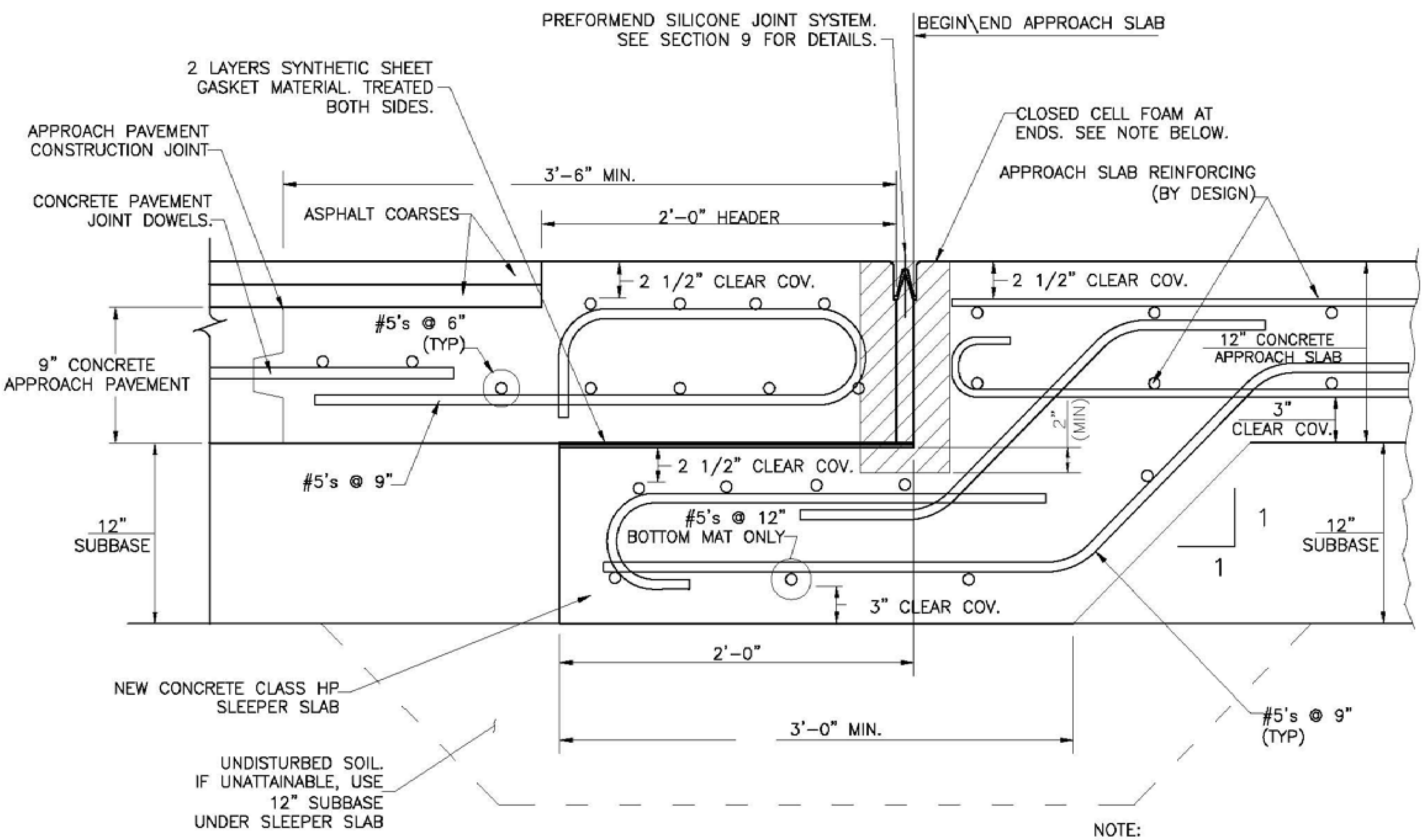


APPROACH SLAB END DETAIL – FIXED END.

CEMENT CONCRETE APPROACH ROADWAY/ASPHALT WEARING SURFACE
DETAIL 4.6.2.7.d
N.T.S.

NOTE:
1" x 6" CLOSED CELL FOAM 14" LONG SHALL BE PLACED AS SHOWN AND BONDED TO THE CONCRETE WITH AN APPROVED ADHESIVE AT THE ENDS OF THE JOINT TO PREVENT BACKFILLING OF THE VOID UNDER THE JOINT.

The following criteria are recommended for integral abutment joint treatments:

A. Expansion lengths less than 60 feet: No provisions for expansion at the ends of the approach slabs are required if asphalt approach pavements are used. When the approach pavement is rigid cement concrete, use a preformed silicone joint system between the sleeper slab and the approach pavement. See Details 4.6.2.7.d.

B. Expansion Lengths from 60 feet to 150 feet: Provisions for expansion at the ends of approach slabs will require the use of a preformed silicone joint system between the sleeper slab and the approach slab with asphalt concrete approach pavement. See Detail 4.6.2.7.a. A preformed silicone joint system between the sleeper slab and the approach slab shall also be used with rigid concrete approach pavement. See Details 4.6.2.7.b and 4.6.2.7.c.

C. Expansion length between 150 feet and 300 feet: Provision shall be made for expansion at the end of the approach slab. If at all possible, the span arrangement and interior bearing selection shall be such that approximately equal movements will occur at each abutment. See “B” above.

D. Expansion length between 300 feet to 400 feet: Lengths in this range shall be approved by the **DSD** on an individual basis. Provision for expansion shall be made at the end of each approach slab with an appropriate joint. Refer to Section 9 – Joints, for selecting the appropriate joint type between the sleeper slab and the approach slab based on the expected thermal movement.

E. Expansion lengths over 400 feet: Integral abutment bridges shall not be used with expansion lengths over 400 feet.

4.6.2.9 - SLOPE PROTECTION

Slope protection in front of integral abutments shall typically be concrete block paving or stamped concrete as described in Subsection 4.3 – Embankment and Slope Protection. See Details 4.3.1.1.a & 4.3.1.1.b. Where a bridge crosses a waterway the slope protection will be as required in the **HADR**. Bedding requirements, such as geotextile, will be specified in the **FDR**.

4.6.2.10 - FREEBOARD/SUBMERGED INLETS

Structures with reduced freeboard or submerged inlets will be subjected to a greater general and local scour and impact damage. Therefore, structure height should include at least 2 feet of freeboard above **Design High Water** elevation **(DHW)** unless more clearance is required by the **HADR**. The fixity between the superstructure and substructure on integral abutment bridges will provide adequate protection against superstructure uplift and movement.

4.6.3 - INTEGRAL ABUTMENT BRIDGE DESIGN PROCEDURES

The design of structures with integral abutments requires some modifications to the **AASHTO** Specifications. Those modifications are presented in the following subsections.

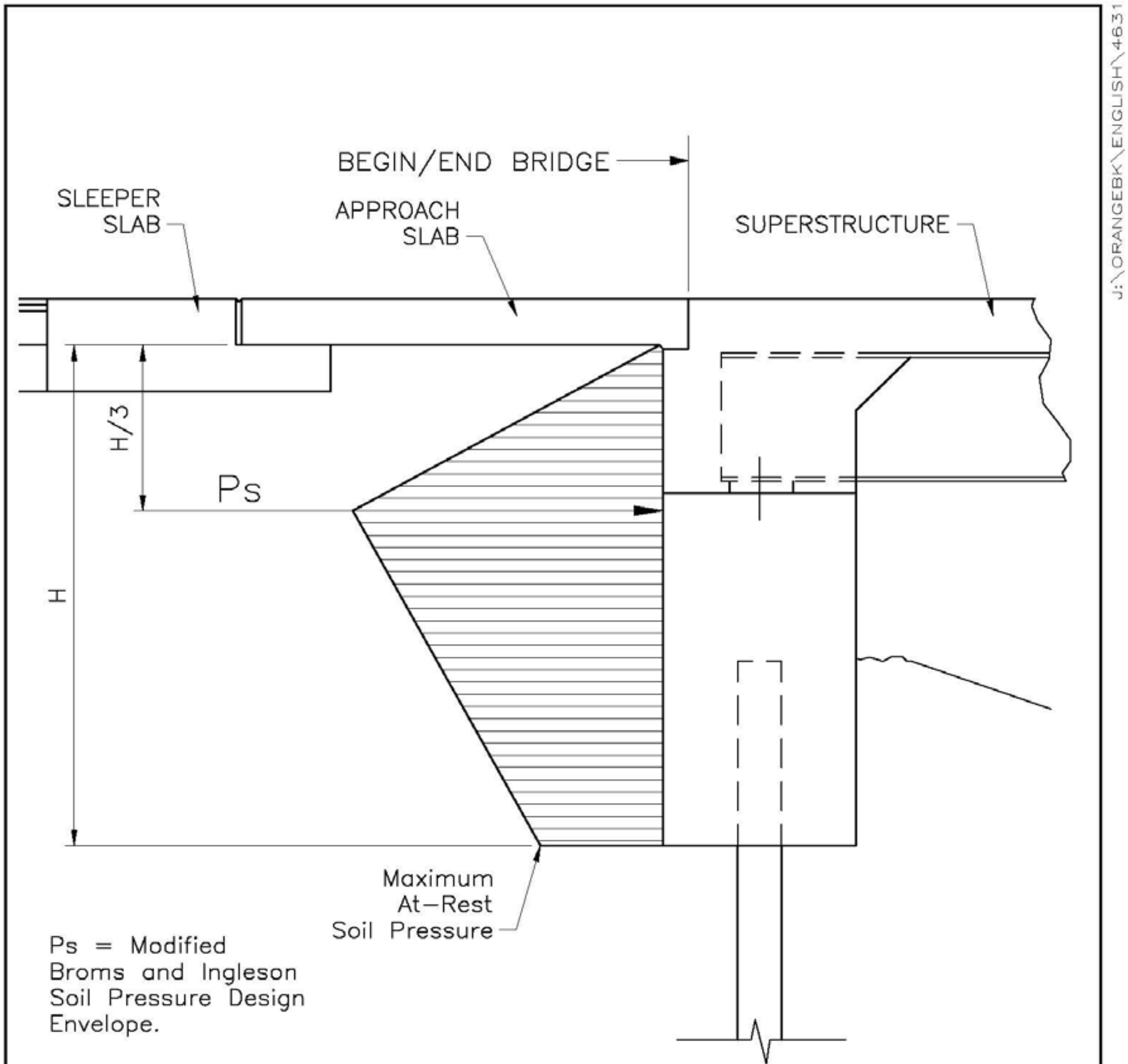
4.6.3.1 – SUBSTRUCTURES

A. Preliminary abutment pile sizes will be given in the **FDR**. The designer shall verify the size and orientation of the piles using the Rational Design Approach for Integral Abutment Bridge Piles. In general, the pile is assumed fixed in the abutment stem and fixed in the soil at some depth below the pre-augured hole. The piles experience compound bending and vertical loading and will be designed plastically in most cases. If the pile size indicated in the **FDR** must be changed, the Geotechnical Engineer shall be informed, and a supplemental **FDR** will be issued. Detailed design procedures and programs are available in the Structures Design Bureau for this type of design. Refer to Table 4.6.2.1 for typical pile types and sizes.

B. On all integral abutment stems, the stability of the stem and strength of the piles must be checked under construction loading. Assuming compacted backfill behind the stem up to the bridge seat elevation, a 2 foot surcharge on top of that to account for construction vehicle loads, and no backfill in front of the stem, The strength of the piles and stability of the pile-stem system shall be checked. The maximum allowable deflection at the top of the stem under this loading is 1.0 inch. The soil pressure envelope for this loading condition shall assume the active condition.

C. On integral abutment stems $\leq$ 10 feet high, the stem concrete and vertical reinforcing steel shall be designed for a combination of the soil pressure developed against the back of stem from either full passive pressure or the modified Broms and Ingleson Design Envelope shown on Detail 4.6.3.1, and the moments induced in the abutment from **SDL** and **LL** on the superstructure. The abutment is assumed rigidly connected to the superstructure and the piles, not allowing relative rotation or translation between the elements. Detailed design procedures and programs are available in the Structures Design Bureau for this type of design.

D. On Integral abutments > 10 feet high where the Modified Broms and Ingleson Design Envelope may produce an impractical design, i.e. too much reinforcement required, the stem shall be isolated from the backfill soil pressure as shown on Detail 4.6.2.6.b. The **FDR** will provide the **GRES** design. For projects where a **GRES** retained backfill is too costly due to site conditions, a project specific solution will be determined in the **FDR**. The stem concrete and vertical reinforcing steel shall be designed for the moment on the stem due to a combination of the lateral resistance of the Geofoam during thermal expansion and the moments induced in the abutment from **SDL** and **LL** on the superstructure.



DESIGN SOIL PRESSURE ENVELOPE
INTEGRAL ABUTMENT

DETAIL 4.6.3.1
N.T.S.

The abutment is assumed rigidly connected to the superstructure and the piles, not allowing relative rotation or translation between the elements. Detailed design procedures and programs are available in the Structures Design Bureau for this type of design. The actual Geofoam resistance and application point(s) will be given in the **FDR**. This type of configuration eliminates the need to design for passive soil pressure as described in the seismic requirements of the **AASHTO 17th Edition** Division IA Subsection 6.4.3(B).

E. Horizontal reinforcement in the abutment stem shall be designed continuously between the piles for the loading of the passive resistance during thermal expansion.

F. Wingwalls integral with the main abutment stem shall be designed as vertically cantilevered over the outside piles and horizontally cantilevered at the interface with the abutment stem. These wingwalls must be designed to support the loading due to passive soil pressure during thermal expansion as given in the **FDR**.

G. Wingwalls separate from the main abutment stem shall be designed as vertically cantilevered retaining walls resisting active soil pressure from the approach backfill.

4.6.3.2 – SUPERSTRUCTURES

A. The maximum span lengths shall be as specified in Subsection 4.6.1.1.

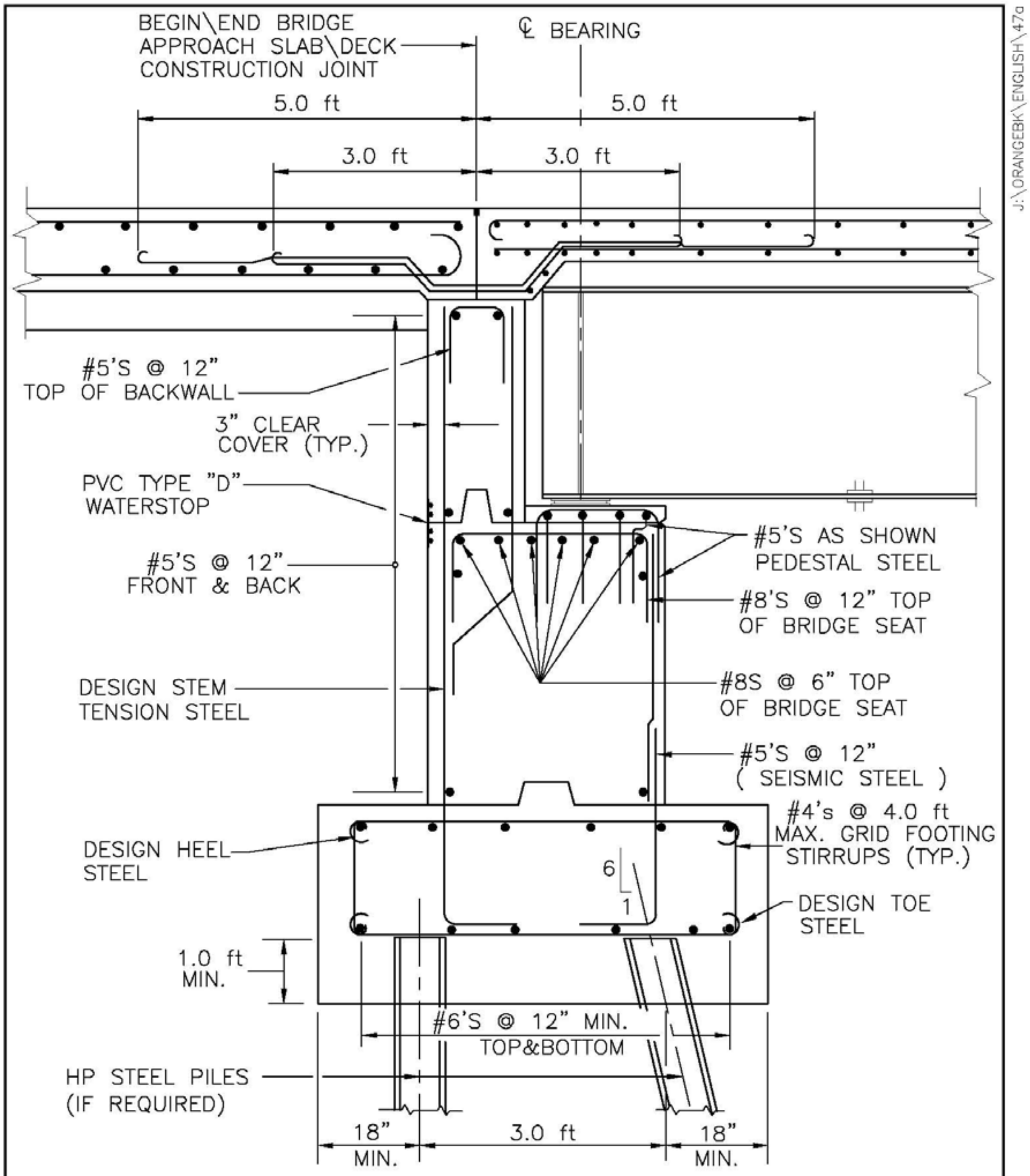
B. On traditional integral abutment bridges the superstructure main load carrying members and deck shall be designed in the normal manner assuming simple supports at the abutments and continuous over the pier(s) for positive bending within the spans and negative bending and shear at the pier. At the abutments, the superstructure main load carrying members and deck shall be designed for negative bending and shear assuming the abutment ends are fixed against rotation.

C. On rigid frame integral abutment bridges the superstructure main load carrying members and deck shall be considered simple spans until the deck ends are poured at the substructures. For **SDL** and **LL** the superstructure shall be designed for positive and negative bending and shear assuming rigid connections at all substructures. Detailed design procedures and programs are available in the Structures Design Bureau for this type of design.

D. Approach slabs shall be designed as reinforced concrete beams as detailed in Subsection 4.6.2.6 with design reinforcing in the bottom running parallel to traffic.

4.7 – JOINTLESS BRIDGE ABUTMENTS

The jointless bridge abutment consists of a concrete stem supported on a spread footing or multi-row pile cap. The superstructure sits on bearings on an abutment bridge seat or pedestals. The superstructure deck is continuous with the approach slab over an abutment backwall. The backwall is rigidly attached to the abutment stem and also supports the backfill beneath the approach slab. Expansion of the deck and approach slab over the backwall is achieved with sheet gasket material on top of the backwall used to form a bond breaker See Detail 4.7.2.5. Expansion and contraction of the roadway surface is handled at the approach slab/sleeper slab interface similar to that of the integral abutment configuration. This type of abutment eliminates the need for expansion joints on the bridge. See Detail 4.7.



TYPICAL JOINTLESS BRIDGE ABUTMENT
SECTION W/STEEL SUPERSTRUCTURE
DETAIL 4.7

4.7.1 - GUIDELINES ON USE

Jointless bridge Abutments shall only be used when the **FDR** restricts the use of integral abutments. The criteria for the use and restrictions of jointless bridge abutments are described in the following subsections.

4.7.1.1 - EXPANSION LENGTHS

The movement of the superstructure over the backwall of a jointless bridge abutment is largely attributed to thermal expansion and contraction of the superstructure. The longer the expansion span length, the larger the longitudinal movement of the superstructure. The expansion length of a jointless bridge abutment structure is equal to the abutment centerline to abutment centerline dimension for single span structures, and the abutment centerline to fixed pier centerline dimension for multi-span structures. As the superstructure expands and contracts, the deck/approach slab slide over the backwall. The backwall is loaded horizontally from the at-rest soil pressure behind it and the frictional force from the superstructure movement. Care must be taken to assure that the superstructure beams or girders do not contact the backwall during maximum thermal expansion. As long as these movements are accounted for, there is no expansion length limitation for this type of abutment.

4.7.1.2 – SOIL CONDITIONS

Site soil conditions shall be analyzed in the **FDR** from existing and new soil borings. The **FDR** (and **HADR** if spanning a waterway) will determine whether the jointless bridge abutments shall be supported on a spread footing or multi-row pile cap. The appropriate foundation type will be based on the geometry, loading, and site conditions at the structure.

4.7.1.3 - HORIZONTAL ALIGNMENT

Straight or curved beams and superstructures will be allowed. On curved structures, thermal movements of the superstructure (both longitudinal and transverse) must be considered in the design of the various elements.

4.7.1.4 - GRADE

There is no maximum grade for bridges on jointless bridge abutments. However, the direction of thermal expansion should always be uphill wherever possible.

4.7.1.5 - SKEW ANGLE

There is no skew limitation on bridges with jointless bridge abutments. However, the effects of skew must be analyzed and accounted for in the design of all of the structural components.

4.7.1.6 - UTILITIES (Refer to Subsection 1.6 - UTILITY COMMUNICATION AND COORDINATION)

Since jointless bridge abutments and backwalls do not move, all utilities may run through the stem and backwall with appropriate sleeving as necessary.

4.7.2 – DESIGN AND DETAIL CONSIDERATIONS

4.7.2.1 - FOUNDATION TYPES

All jointless bridge abutments shall be supported on a spread footing or multi-row pile cap. Steel H or **CIP** piles shall be used as recommended in the **FDR**. Exact length required shall be specified in the **FDR**. A complete **HADR** (that includes a depth of scour analysis) of the site and proposed structure is required where a structure crosses over a waterway. For these structures, the piles are designed to gain all of their capacity below the scour elevation.

4.7.2.2 - ABUTMENT STEM

The abutment stem thickness shall be determined from the geometry of the bridge seat and backwall. The bridge seat/pedestals must be deep and wide enough to provide room for the appropriate bearing with required edge and end clearances, and allow for thermal movement of the superstructure without coming in contact with the backwall. The backwall shall be a minimum of 18 inches thick. The use of corbels at the back and bottom of the backwall to reduce abutment stem thickness should be avoided. Refer to Subsection 4.5.2.

4.7.2.3 - WINGWALLS

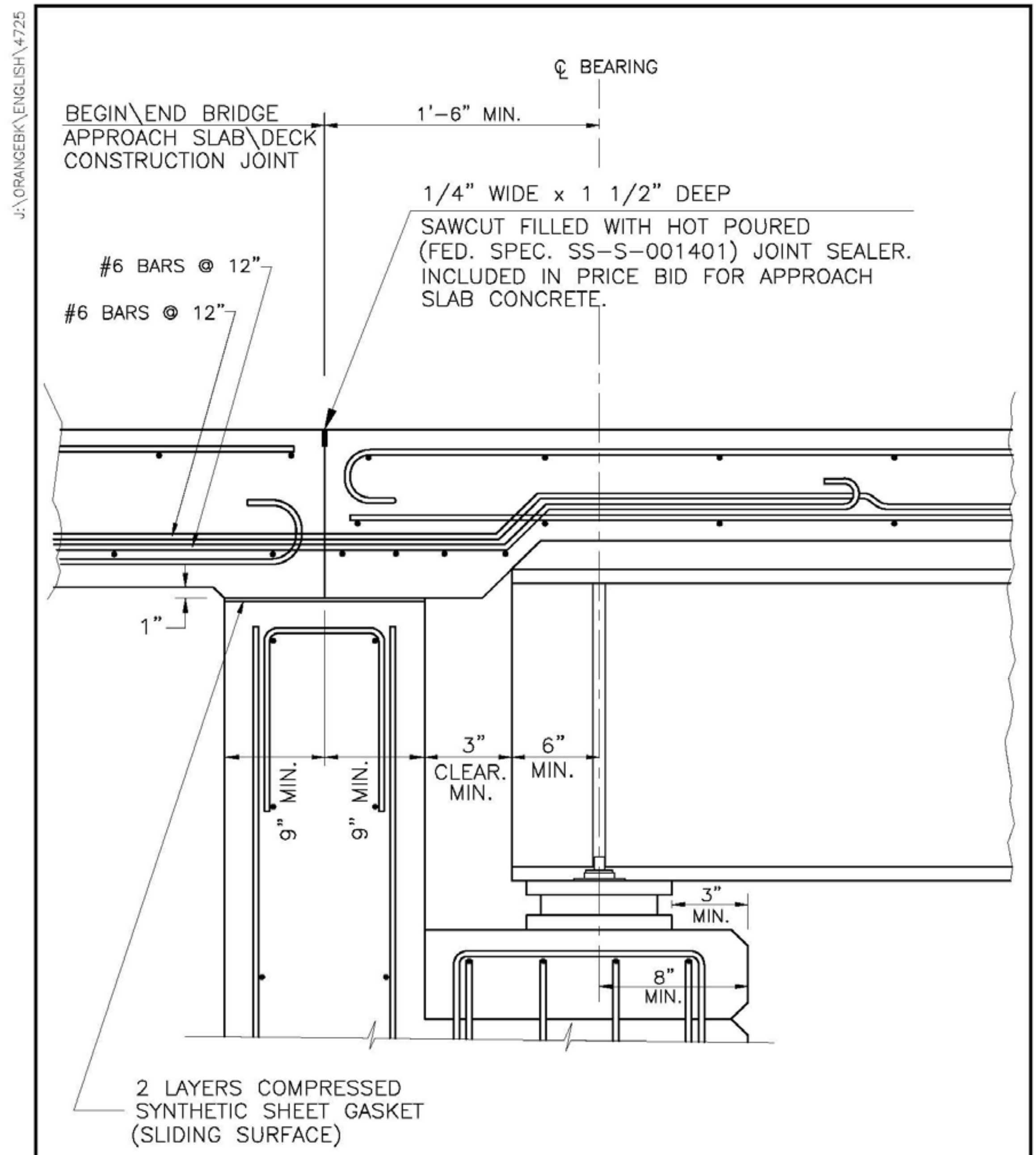
Wingwalls shall be a minimum of 18 inches thick. Wingwalls shall have a constant thickness or be tapered depending on height and length. Tapering the thickness of larger walls from end to end may result in significant concrete quantity savings. In-line wingwalls are the preferred arrangement for jointless bridge abutments. Flared walls shall be used at stream crossings and taller abutments where wingwall length may be significantly reduced by flaring. U-walls should not be used on jointless bridge abutments. U-walls are discouraged because the expanding and contracting approach slab tends to bind on the U-walls. For wingwalls cantilevered off the jointless bridge abutment stem, see Subsection 4.5.3 and Detail 4.6.2.3. For wingwalls self supported on footings or piles and connected to the abutment with a keyed construction, contraction, or expansion joint, see Subsection 4.5.4, and the details in Appendix C.

4.7.2.4 - SUPERSTRUCTURE TYPE

Steel beams, plate girders, or prestressed concrete beams may be used on bridges with jointless bridge abutments. The deck may be either cast-in-place or precast. Refer to Section 3 – Decks, for more information. Refer to Subsection 4.7.3 – Jointless Bridge Abutment Bridge Design Procedures.

4.7.2.5 - BEARINGS

When steel beams or plate girders are used in the superstructure, a bearing device shall be designed and detailed to support the superstructure on the abutment bridge seat/pedestals. Refer to Section 8 – Bearings for more information. See Detail 4.7.2.5. Prestressed concrete I-beams require individual rectangular plain rubber bearing pads placed perpendicular to the centerline of the beam. Prestressed concrete box beams require a continuous rectangular plain rubber bearing pad placed at the centerline of bearing of the beams for the full length of the bridge seat. Both types of prestressed concrete beams shall be connected to the abutment with anchor rods. Refer to Section 8 – Bearings. Refer to **NYSDOT BD** Sheets for more information.



TYPICAL JOINTLESS BRIDGE ABUTMENT SECTION
W/STEEL SUPERSTRUCTURE

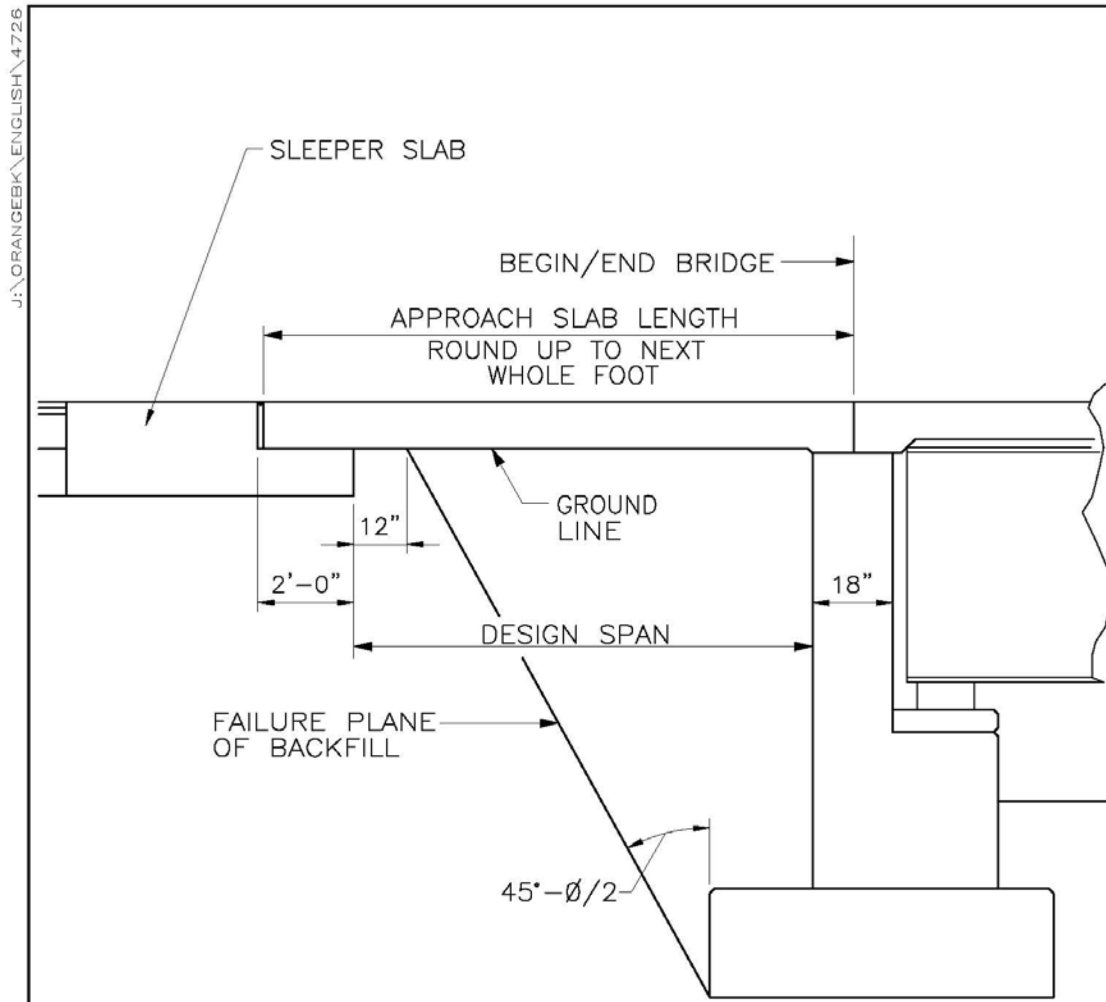
DETAIL 4.7.2.5

N.T.S.

4.7.2.6 - APPROACH SLABS

Approach slabs are required for all jointless bridge abutments. The purpose of the approach slab is to bridge the fill directly behind the abutment and provide a transition from the approach pavement to the bridge deck. Approach slab thickness shall be a minimum of 1 foot. Thickness may be greater depending on the design span length of the approach slab. Approach slab lengths vary depending on the height of the abutment and backfill treatment. In most cases the length is determined based on the intercept of the backfill active failure plane from the bottom and back of the abutment footing to the bottom of the approach slab (ground line). See Detail 4.7.2.6. The end of the approach slab shall be supported on a sleeper slab. This end of the approach slab shall be perpendicular to the centerline of the roadway and run from face-of-guiderail to face-of-guiderail. The abutment end of the approach slab shall be rigidly connected to the superstructure deck as shown in Detail 4.7.2.5. Polyethylene curing covers shall be placed on top of the subbase prior to pouring the approach slab.

This sheet will aid in allowing free thermal movement of the approach slab on the subbase material. The approach slab shall incorporate both top and bottom steel reinforcement. The top mats (transverse and longitudinal) of reinforcement shall be a minimum of #5 Bars @ 12 inch spacing in both directions. The bottom mat longitudinal reinforcement shall be designed for traffic loading (reinforcing parallel to traffic) with the design span being a simple span from the back face of the abutment to 1 foot beyond the intersection with the failure plane. The bottom mat transverse reinforcement shall be for temperature only.



Ø FOR GRANULAR MATERIAL
IS TYPICALLY 34°
REFER TO FDR FOR
ACTUAL VALUE.

APPROACH SLAB LENGTH
JOINTLESS & SEMI–INTEGRAL
BRIDGE ABUTMENTS

DETAIL 4.7.2.6
N.T.S.

Refer to the **AASHTO** specifications for the minimum requirements when the main reinforcement is parallel to traffic.

4.7.2.7 - SLEEPER SLABS

The sleeper slab is a buried concrete foundation used to support the free end of the approach slab. The end of the approach slab slides on the end of the sleeper slab. Sleeper slab reinforcement shall be as shown in Details 4.6.2.7.a through 4.6.2.7.d.

4.7.2.8 - JOINTS

A cold formed construction joint should be located between the approach slab and the superstructure deck as described in Subsection 4.7.2.6, at the centerline of the backwall. This joint will provide a controlled crack rather than allowing a random crack to develop in the roadway surface. This joint shall also define the beginning and end of bridge stationing. See Detail 4.7.2.5.

Galvanized reinforcing steel shall connect the approach slab to the superstructure deck. This reinforcement provides a positive connection between the two to keep the joint tight. This joint **must** be cold formed. The joint will be sawn and sealed as described in the Approach Slab Notes in Appendix B.

An expansion joint shall be placed at the end of the approach slab between the approach slab and the sleeper slab. The purpose of this joint is to allow for the thermal movement of the superstructure and approach slab. This is a working joint that opens and closes due to thermal expansion and contraction. The longer the span, the greater the opening and closing. The size of the joint opening shall be indicated on the plans.

The following criteria are recommended for jointless bridge abutment joint treatments:

A. Expansion lengths less than 60 feet: No provisions for expansion at the ends of the approach slabs are required if asphalt approach pavements are used. When the approach pavement is rigid cement concrete, use a preformed silicone joint system between the sleeper slab and the approach pavement. See Detail 4.6.2.7.d.

B. Expansion Lengths from 60 feet to 150 feet: Provisions for expansion at the ends of approach slabs will require the use of a preformed silicone joint system between the sleeper slab and the approach slab with asphalt concrete approach pavement. See Detail 4.6.2.7.a. A preformed silicone joint system between the sleeper slab and the approach slab shall also be used with rigid concrete approach pavement. See Details 4.6.2.7.b and 4.6.2.7.c.

C. Expansion length between 150 feet and 300 feet: Provision shall be made for expansion at the end of the approach slab. If at all possible, the span arrangement and interior bearing selection shall be such that approximately equal movements will occur at each abutment. See "B" above.

D. Expansion lengths over 300 feet: Provision for expansion shall be made at the end of each approach slab with an appropriate joint. Refer to Section 9 – Joints, for selecting the appropriate joint type based on the expected thermal movement.

E. Fixed ends: When the end of the bridge is fixed, no joint is required at the end of approach slab if the approach pavement is full-depth asphalt concrete. Where the approach pavement is rigid cement concrete, a preformed silicone joint system shall be installed between the sleeper slab and the approach pavement to allow for approach pavement expansion and contraction. See Detail 4.6.2.7.d.

4.7.2.9 - SLOPE PROTECTION

Slope protection in front of jointless bridge abutments shall typically be concrete block paving or stamped concrete as described in Subsection 4.3 – Embankment and Slope Protection. See Details 4.3.1.1.a & 4.3.1.1.b. Where a bridge crosses a waterway, the slope protection will be as required in the **HADR**. Bedding requirements, such as geotextile, will be specified in the **FDR**.

4.7.2.10 - FREEBOARD/SUBMERGED INLETS

Structures with reduced freeboard or submerged inlets will be subjected to a greater general and local scour and impact damage. Therefore, structure height should include at least 2 feet of freeboard above **DHW** unless more clearance is required by the **HADR**. Connection of the superstructure to the abutment bridge seat/pedestal through the bearings must be designed to resist uplift and/or horizontal stream loads on the superstructure as indicated in the **HADR**.

4.7.3 - JOINTLESS BRIDGE ABUTMENT BRIDGE DESIGN PROCEDURES

The design of structures with jointless bridge abutments will be as required in the **AASHTO** specifications.

4.7.3.1 – SUBSTRUCTURES

A. Where the abutment stem is supported on piles, the preliminary abutment pile sizes and pile cap dimensions will be given in the **FDR**. The designer shall verify the size and orientation of the piles checking for bearing capacity and uplift. Any uplift requirements shall be relayed to the geotechnical engineer for consideration in the **FDR**.

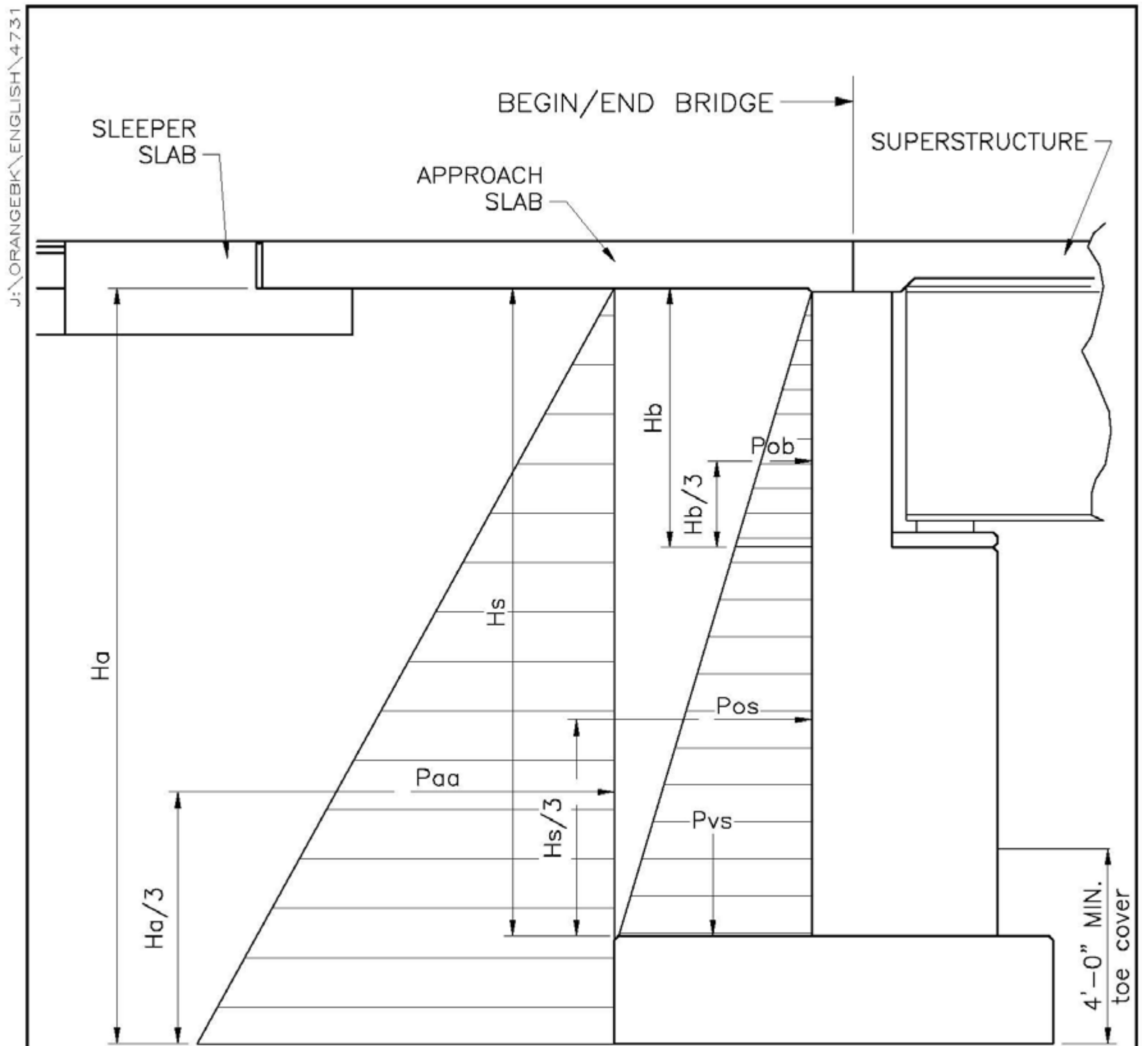
B. Where the abutment stem is supported on a spread footing, the allowable bearing pressure will be given in the **FDR**. The designer shall proportion the footing for stability (sliding and overturning) of the abutment under construction loading. See Detail 4.7.3.1. The top and bottom reinforcing shall be designed for the completed live load condition.

C. The stem concrete and vertical reinforcing steel shall be designed for a combination of the vertical and horizontal loads from the superstructure and the At-Rest soil pressure developed against the back of stem from the compacted soil on the abutment heel. See Detail 4.7.3.1.

D. Horizontal reinforcement in the abutment stem shall be for temperature only.

E. Wingwalls cantilevered off the abutment stem shall be designed as vertically cantilevered over the abutment footing and horizontally cantilevered at the interface with the abutment stem resisting the At-Rest soil pressure from the approach backfill.

F. Wingwalls supported on their own foundations shall be designed as vertically cantilevered retaining walls resisting active soil pressure from the approach backfill.



Paa = 1/2KaγHaxHa = ACTIVE SOIL PRESSURE ON ABUTMENT
Pos = 1/2KoγHsxHs = AT-REST SOIL PRESSURE ON STEM
Pob = 1/2KoγHbxHb = AT-REST SOIL PRESSURE ON BACKWALL
Pvs = γxHsxHEEL LENGTH = VERTICAL SOIL LOAD ON HEEL
γ = UNIT WT OF BACKFILL

REFER TO FDR FOR VALUES FOR γ AND Ø (FOR DETERMINING "K" VALUES).

DESIGN SOIL PRESSURES
JOINTLESS & CONVENTIONAL BRIDGE ABUTMENT
DETAIL 4.7.3.1

N.T.S.

4.7.3.2 – SUPERSTRUCTURES

A. There are no maximum span lengths for this type of structure as long as the thermal movement is accounted for at the end of the approach slabs.

B. The superstructure main load carrying members and deck shall be designed assuming simple supports at the abutments and continuous over any pier(s) for positive bending within the spans and negative bending and shear at the pier(s).

C. Approach slabs shall be designed as reinforced concrete beams as detailed in Subsection 4.7.2.6 with design reinforcing in the bottom running parallel to traffic.

4.8 – SEMI-INTEGRAL ABUTMENTS WITH CURTAIN WALL BACKWALLS

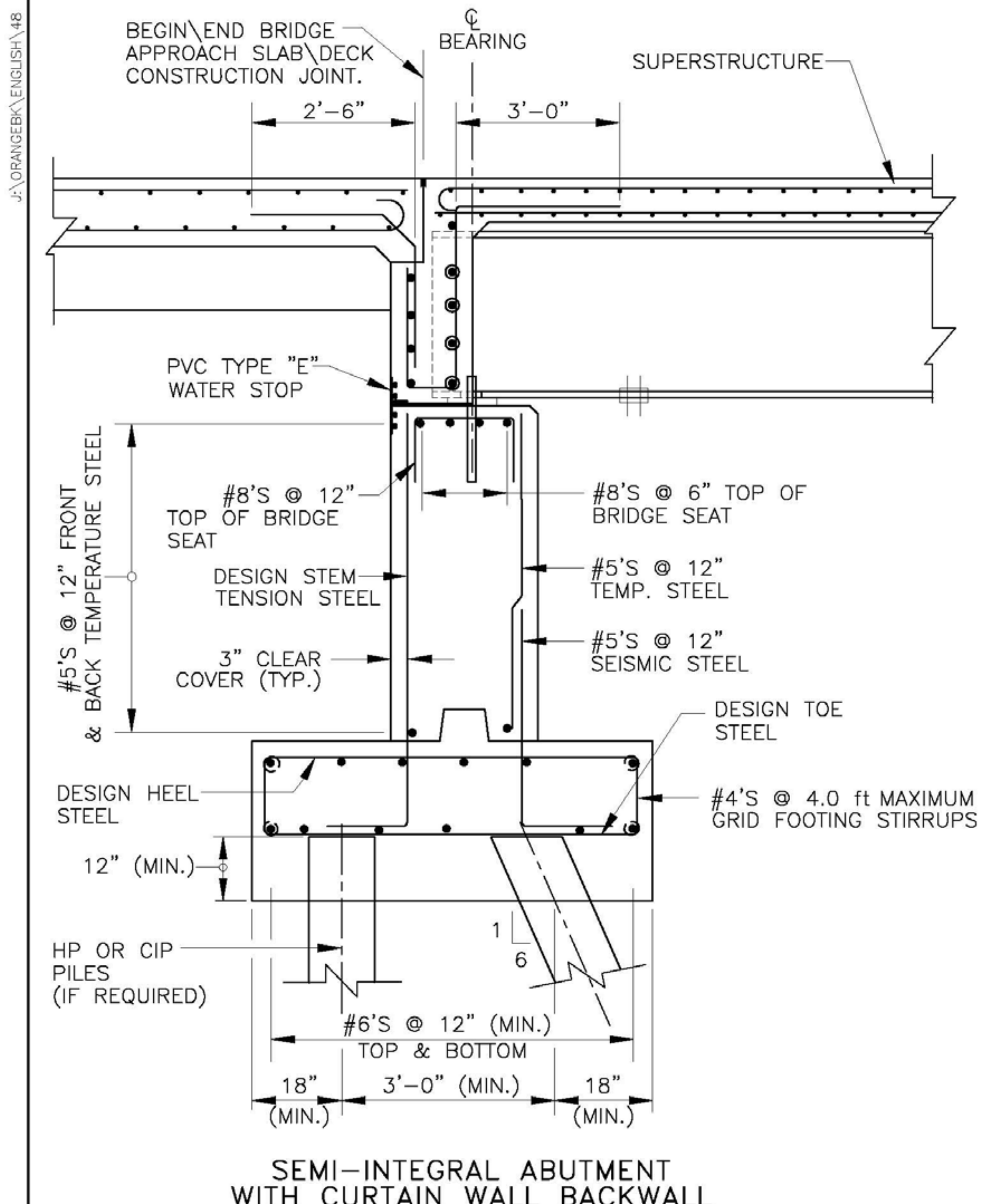
The semi-integral abutment with curtain wall backwall consists of a concrete stem supported on a spread footing or multi-row pile cap. The superstructure sits on elastomeric pads on an abutment bridge seat. The superstructure deck is continuous with the approach slab and a curtain wall acting as an abutment backwall. The curtain wall is cast with the deck around the beam ends acting as end diaphragms as well as a retaining wall for the soil behind. While the beams are supported on individual elastomeric pads on the bridge seat, the curtain wall rests on a continuous elastomeric pad on the bridge seat. All expansion and contraction involves the superstructure, approach slab, and the curtain wall over the abutment bridge seat. At expansion abutments, a ½ inch thick elastomeric pad, and two layers of compressed synthetic sheet gasket are placed between the bridge seat and the curtain wall. Expansion and contraction of the roadway surface is handled at the approach slab/sleeper slab interface similar to that of the integral abutment configuration. This type of abutment eliminates the need for expansion joints on the bridge. At fixed abutments, a ½ inch thick elastomeric pad alone is placed between the bridge seat and the curtain wall with no bond breakers. A waterstop at the back of abutment stem joins the abutment to the curtain wall at both ends to keep ground water from penetrating the bridge seat area. Anchor rods run up from the abutment stem through slotted holes in the beams to allow for translation and rotation of the superstructure. This type of structure may be appropriate when the use of a single line of flexible piles (such as with an integral abutment) is not possible due to foundation conditions. It also may be appropriate at the abutments of superstructure replacements. See Detail 4.8.

4.8.1 - GUIDELINES ON USE

The semi-integral abutment is a hybrid of the integral abutment and the jointless bridge abutment. Semi-integral abutments may be used when the **FDR** restricts the use of integral abutments and expansion span lengths are short. The criteria for the use and restrictions of semi-integral abutments are described in the following subsections.

4.8.1.1 - EXPANSION LENGTHS

The movement of the superstructure over the bridge seat of a semi-integral abutment is largely attributed to thermal expansion and contraction of the superstructure. The longer the expansion span length, the larger the longitudinal movement of the superstructure.



SEMI–INTEGRAL ABUTMENT
WITH CURTAIN WALL BACKWALL
TYPICAL SECTION

DETAIL 4.8

N.T.S.

The expansion length of a semi-integral abutment structure is equal to the abutment centerline to abutment centerline dimension for single span structures, and the abutment centerline to fixed pier centerline dimension for multi-span structures. As the superstructure expands and contracts, the deck/curtain wall/approach slab slide over the bridge seat. The bridge seat is loaded horizontally from the frictional force from the superstructure movement. The allowable expansion length is limited by the capacity of the waterstop at the back of abutment/curtain wall interface to distort without failing. In general expansion lengths should be limited to 100 feet.

4.8.1.2 – SOIL CONDITIONS

Site soil conditions shall be analyzed in the **FDR** from existing and new soil borings. The **FDR** (and **HADR** if spanning a waterway) will determine whether the semi-integral abutments shall be supported on a spread footing or multi-row pile cap. The appropriate foundation type will be based on the geometry, loading, and site conditions at the structure.

4.8.1.3 - HORIZONTAL ALIGNMENT

Only straight beams will be allowed. Curved superstructures will be allowed provided the beams are straight and continuous between the abutments. All beams shall be parallel to each other. The abutments and any intermediate piers shall also be parallel to each other.

4.8.1.4 – GRADE

The maximum vertical curve gradient between abutments shall be 5%.The maximum straight grade allowed on semi-integral abutment bridges is 10%.

4.8.1.5 - SKEW ANGLE

There is no skew limitation on semi-integral abutment bridges. However, the effects of skew must be analyzed and accounted for in the design of all of the structural components.

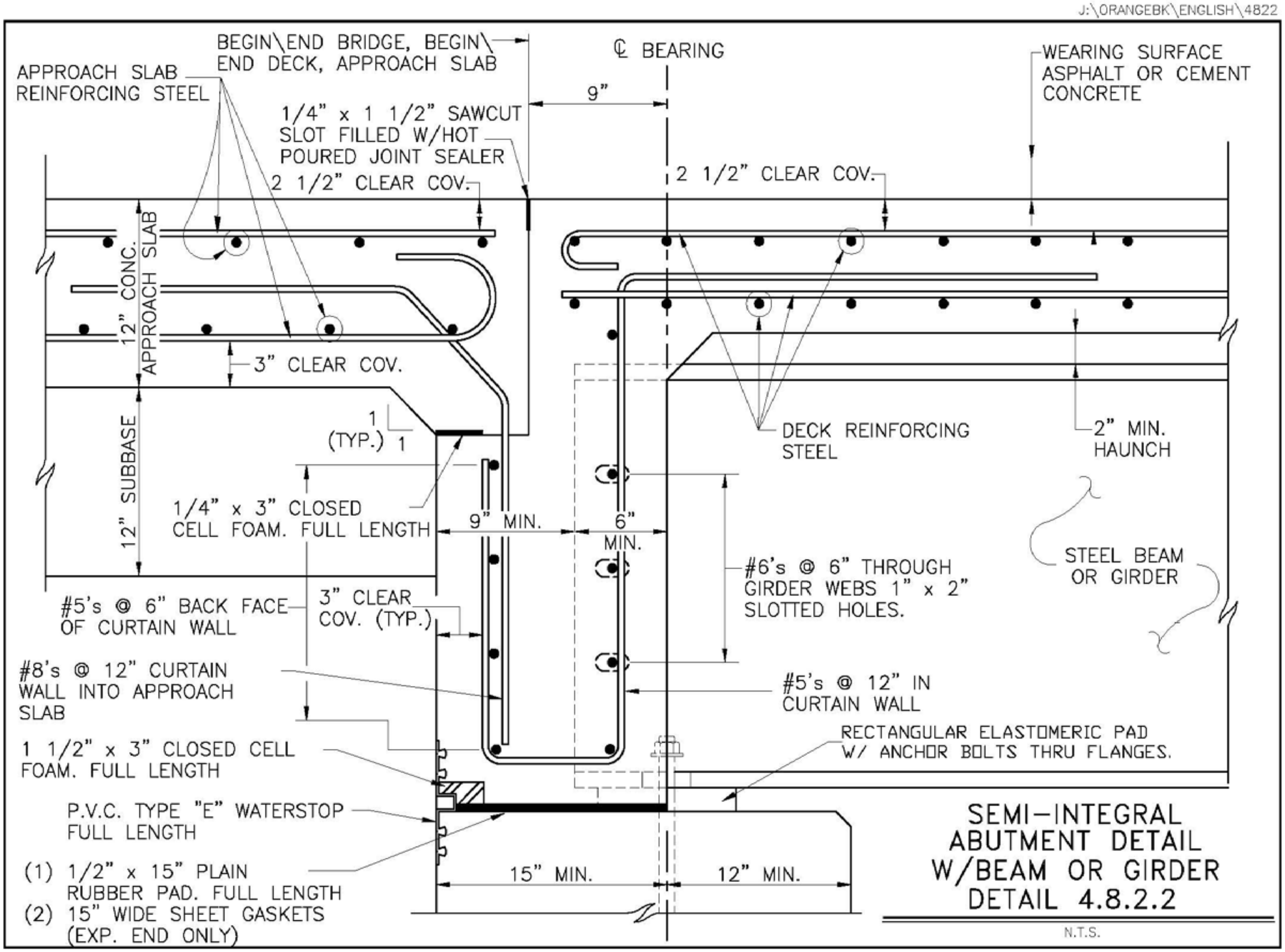
4.8.1.6 - UTILITIES (Refer to Subsection 1.6 - UTILITY COMMUNICATION AND COORDINATION)

Since the expansion span lengths of semi-integral abutment bridges is relatively short, all utilities may run through the abutment stem and curtain wall with appropriate sleeving as necessary.

4.8.2 – DESIGN AND DETAIL CONSIDERATIONS

4.8.2.1 - FOUNDATION TYPES

All semi-integral abutments shall be supported on a spread footing or multi-row pile cap. Steel H or **CIP** piles shall be used as recommended in the **FDR**. Exact length required shall be specified in the **FDR**. A complete **HADR** (that includes a depth of scour analysis) of the site and proposed structure is required where a structure crosses over a waterway. For these structures, the piles are designed to gain all of their capacity below the scour elevation.


J:\ORANGEBK\ENGLISH\4822
BEGIN\END BRIDGE, BEGIN\END DECK, APPROACH SLAB
℄ BEARING
WEARING SURFACE ASPHALT OR CEMENT CONCRETE
APPROACH SLAB REINFORCING STEEL
9"
1/4" x 1 1/2" SAWCUT SLOT FILLED W/HOT POURED JOINT SEALER
2 1/2" CLEAR COV.
2 1/2" CLEAR COV.
12" CONC. APPROACH SLAB
3" CLEAR COV.
12" SUBBASE
1 (TYP.) 1
DECK REINFORCING STEEL
2" MIN. HAUNCH
1/4" x 3" CLOSED CELL FOAM. FULL LENGTH
9" MIN.
6" MIN.
#6's @ 6" THROUGH GIRDER WEBS 1" x 2" SLOTTED HOLES.
STEEL BEAM OR GIRDER
#5's @ 6" BACK FACE OF CURTAIN WALL
3" CLEAR COV. (TYP.)
#8's @ 12" CURTAIN WALL INTO APPROACH SLAB
#5's @ 12" IN CURTAIN WALL
1 1/2" x 3" CLOSED CELL FOAM. FULL LENGTH
RECTANGULAR ELASTOMERIC PAD W/ ANCHOR BOLTS THRU FLANGES.
P.V.C. TYPE "E" WATERSTOP FULL LENGTH
(1) 1/2" x 15" PLAIN RUBBER PAD. FULL LENGTH
(2) 15" WIDE SHEET GASKETS (EXP. END ONLY)
15" MIN.
12" MIN.
SEMI-INTEGRAL ABUTMENT DETAIL W/BEAM OR GIRDER DETAIL 4.8.2.2
N.T.S.

4.8.2.2 - ABUTMENT STEM

The abutment stem thickness shall be determined from the geometry of the bridge seat. The bridge seat must be deep enough to provide room for the appropriate size elastomeric pads under the beams with required edge and end clearance, and the curtain wall. The curtain wall shall be a minimum of 15 inches thick for steel or concrete I-beam superstructures and 12 inches for concrete box-beam superstructures. The use of corbels at the back and below the bridge seat to reduce abutment stem thickness should be avoided. Refer to Subsection 4.5.2. See Detail 4.8.2.2.

4.8.2.3 – WINGWALLS

Wingwalls shall be a minimum of 18 inches thick. Wingwalls shall have a constant thickness or be tapered depending on height and length. Tapering the thickness of larger walls from end to end may result in significant concrete quantity savings. In-line wingwalls are the preferred arrangement for semi-integral abutments. Flared walls shall be used at stream crossings and taller abutments where wingwall length may be significantly reduced by flaring. U-walls shall not be used on semi-integral abutment bridges. U-walls are not allowed because they prevent the curtain wall from moving freely during thermal changes. For wingwalls cantilevered off the semi-integral abutment stem, see Subsection 4.5.3 and Detail 4.6.2.3. For wingwalls self supported on footings or piles and connected to the abutment stem with a keyed construction, contraction, or expansion joint, see Subsection 4.5.4, and the details in Appendix C. All wingwalls shall be separated from the curtain wall with a keyless expansion joint to allow for the movement of the curtain wall.

4.8.2.4 - SUPERSTRUCTURE TYPE

Steel beams, plate girders, or prestressed concrete beams may be used on bridges with semi-integral abutments. The deck may be either cast-in-place or precast. Refer to Section 3 – Decks, for more information. Refer to Subsection 4.8.3 – Semi-Integral Abutment Bridge Design Procedures.

4.8.2.5 – BEARINGS

When steel beams, plate girders or prestressed concrete I-beams are used in the superstructure, rectangular elastomeric bearing pads shall be designed and detailed to support the beams on the abutment bridge seat. Refer to Section 8 – Bearings for more information. Steel beams, plate girders and prestressed concrete I-beams shall be connected to the abutment with anchor rods. See Detail 4.8.2.2. When prestressed box beams are used, a continuous rectangular plain rubber bearing pad shall be placed at the centerline of bearing of the beams for the full length of the bridge seat. Refer to Section 8 – Bearings. Prestressed box beams shall be connected to the abutment with anchor rods. Refer to **NYSDOT BD** Sheets for more information.

4.8.2.6 - APPROACH SLABS

Approach slabs are required for all semi-integral abutments. The purpose of the approach slab is to bridge the fill directly behind the abutment and provide a transition from the approach pavement to the bridge deck. Approach slab thickness shall be a minimum of 12 inches. Thickness may be greater depending on the design span length of the approach slab. Approach slab lengths vary depending on the height of the abutment and backfill treatment. In most cases the length is determined based on the intercept of the backfill active failure plane from the bottom and back of the abutment footing to the bottom of the approach slab (ground line). See Detail 4.7.2.6. The end of the approach slab shall be supported on a sleeper slab. This end of the approach slab shall be perpendicular to the centerline of the roadway and run from face-of-guiderail to face-of-guiderail. The abutment end of the approach slab shall be rigidly connected to the superstructure deck and curtain wall as shown in Detail 4.8.2.2. Polyethylene curing covers shall be placed on top of the subbase prior to pouring the approach slab. This sheet will aid in allowing free thermal movement of the approach slab on the subbase material. The approach slab shall incorporate both top and bottom steel reinforcement. The top mats (transverse and longitudinal) of reinforcement shall be a minimum of #5 Bars @ 12 inch spacing in both directions. The bottom mat longitudinal reinforcement shall be designed for traffic loading (reinforcing parallel to traffic) with the design span being a simple span from the back face of the abutment to 1 foot beyond the intersection with the failure plane. The bottom mat transverse reinforcement shall be for temperature only. Refer to the **AASHTO** specifications for the minimum requirements when the main reinforcement is parallel to traffic.

4.8.2.7 - SLEEPER SLABS

The sleeper slab is a buried concrete foundation used to support the free end of the approach slab. The end of the approach slab slides on the end of the sleeper slab. Sleeper slab reinforcement shall be as shown in Details 4.6.2.7.a through 4.6.2.7.d.

4.8.2.8 – JOINTS

A cold formed construction joint should be located between the approach slab and the superstructure deck/curtain wall as described in Subsection 4.8.2.6, 6 inches from the back of the curtain wall. This joint will provide a controlled crack rather than allowing a random crack to develop in the roadway surface. This joint shall also define the beginning and end of bridge stationing. See Detail 4.8.2.2. Galvanized reinforcing steel shall connect the approach slab to the curtain wall/deck concrete. This reinforcement provides a positive connection between the two to keep the joint tight. This joint **must** be cold formed. The joint will be sawn and sealed as described in the Approach Slab Notes in Appendix B.

An expansion joint shall be placed at the end of the approach slab between the approach slab and the sleeper slab. The purpose of this joint is to allow for the thermal movement of the superstructure/curtain wall and approach slab. This is a working joint that opens and closes due to thermal expansion and contraction. The longer the span, the greater the opening and closing. The size of the joint opening shall be indicated on the plans.

The following criteria are recommended for semi-integral abutment bridge joint treatments:

A. Expansion lengths less than 60 feet: No provisions for expansion at the ends of the approach slabs are required if asphalt approach pavements are used. When the approach pavement is rigid cement concrete, use a preformed silicone joint system between the sleeper slab and the approach pavement. See Details 4.6.2.7.d.

B. Expansion Lengths from 60 feet to 100 feet: Provisions for expansion at the ends of approach slabs will require the use of a preformed silicone joint system between the sleeper slab and the approach slab with asphalt concrete approach pavement. See Detail 4.6.2.7.a. A preformed silicone joint system between the sleeper slab and the approach slab shall also be used with rigid concrete approach pavement. See Details 4.6.2.7.b and 4.6.2.7.c.

C. Expansion lengths over 100 feet: Expansion lengths over 100 feet are not recommended for semi-integral abutment bridges. Another abutment type should be used if the expansion length exceeds 100 feet.

D. Fixed ends: When the end of the bridge is fixed, no joint is required at the end of approach slab if the approach pavement is full-depth asphalt concrete. Where the approach pavement is rigid cement concrete, a preformed silicone joint system shall be installed between the sleeper slab and the approach pavement to allow for approach pavement expansion and contraction. See Detail 4.6.2.7.d.

4.8.2.9 - SLOPE PROTECTION

Slope protection in front of semi-integral abutments shall typically be concrete block paving or stamped concrete as described in Subsection 4.3 – Embankment and Slope Protection. See Details 4.3.1.1.a & 4.3.1.1.b. Where a bridge crosses a waterway, the slope protection will be as required in the **HADR**. Bedding requirements, such as geotextile, will be specified in the **FDR**.

4.8.2.10 - FREEBOARD/SUBMERGED INLETS

Structures with reduced freeboard or submerged inlets will be subjected to a greater general and local scour and impact damage. Therefore, structure height should include at least 2 feet of freeboard above **DHW** unless more clearance is required by the **HADR**. Since the superstructure is not rigidly connected to the abutment stem, the designer must insure that the unfactored superstructure dead loads are greater than any uplift and/or horizontal stream loads on the structure as indicated in the **HADR**. This may require the addition of concrete ballast between the superstructure beams at the ends. See details in Appendix C.

4.8.3 – SEMI-INTEGRAL ABUTMENT BRIDGE DESIGN PROCEDURES

The design of structures with semi-integral abutments will be as required in the AASHTO Standard Specifications as modified below.

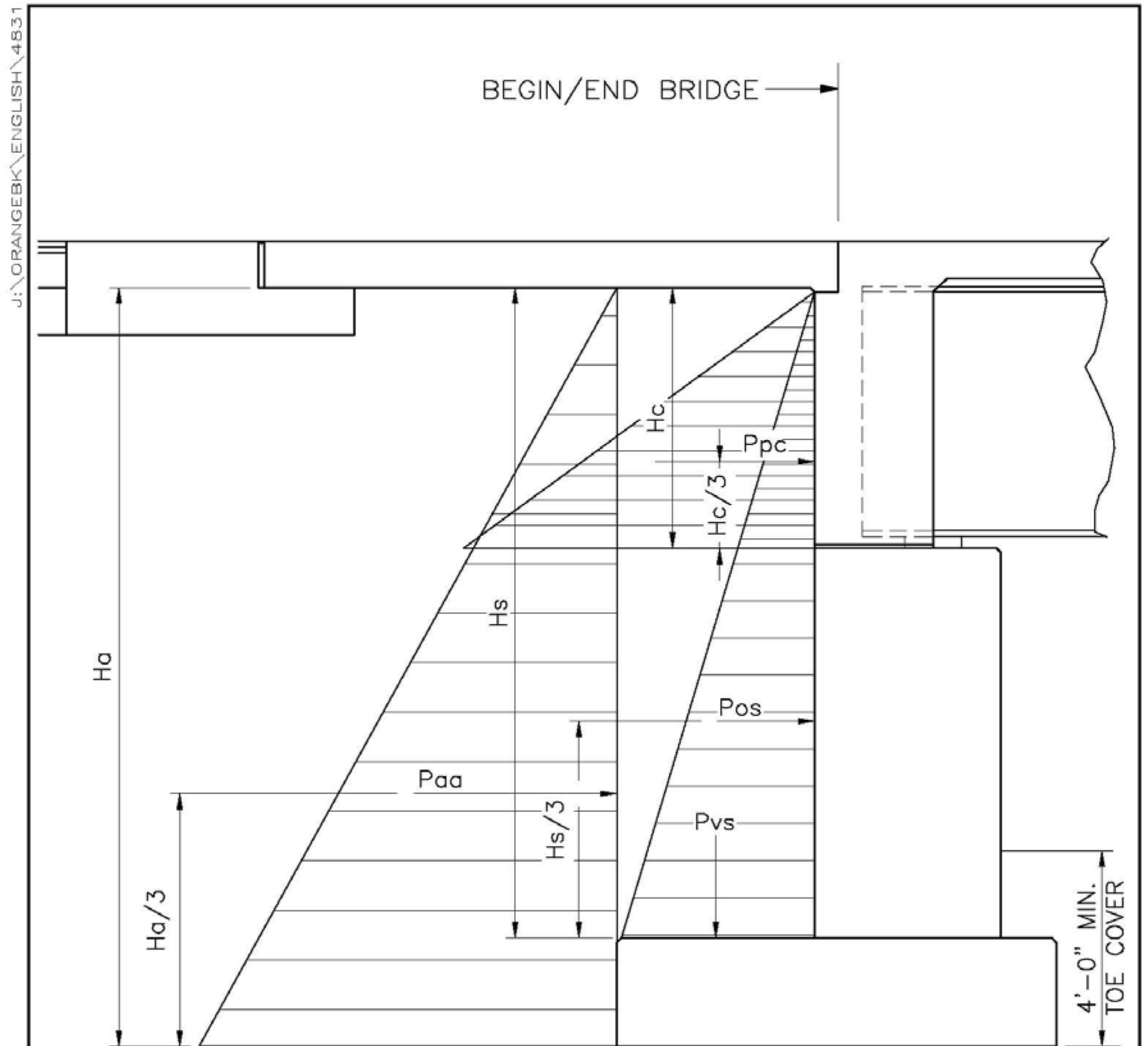
4.8.3.1 – SUBSTRUCTURES

A. Where the abutment stem is supported on piles, the preliminary abutment pile sizes and pile cap dimensions will be given in the **FDR**. The designer shall verify the size and orientation of the piles checking for bearing capacity and uplift. Any uplift requirements shall be relayed to the geotechnical engineer for consideration in the **FDR**.

B. Where the abutment stem is supported on a spread footing, the allowable bearing pressure will be given in the **FDR**. The designer shall proportion the footing for stability (sliding and overturning) of the abutment under construction loading. See Detail 4.8.3.1. The top and bottom reinforcing shall be designed for the completed live load condition.

C. The stem concrete and vertical reinforcing steel shall be designed for a combination of the vertical and horizontal loading from the superstructure and the At-Rest soil pressure developed against the back of stem from the compacted soil on the abutment heel. See Detail 4.8.3.1.

D. Horizontal reinforcement in the abutment stem shall be for temperature only.



$Paa = \frac{1}{2} Ka \gamma Ha x Ha$ = ACTIVE SOIL PRESSURE ON ABUTMENT
$Pos = \frac{1}{2} Ko \gamma Hs x Hs$ = AT-REST SOIL PRESSURE ON STEM
$Ppc = \frac{1}{2} Kp \gamma Hc x Hc$ = PASSIVE SOIL PRESSURE ON CURTAINWALL
$Pvs = \gamma x Hs x$HEEL LENGTH = VERTICAL SOIL LOAD ON HEEL
γ = UNIT WT OF BACKFILL

REFER TO FDR FOR VALUES FOR γ AND Ø (FOR DETERMINING "K" VALUES).

DESIGN SOIL PRESSURES
SEMI-INTEGRAL ABUTMENT
DETAIL 4.8.3.1

N.T.S.

E. Wingwalls cantilevered off the abutment stem shall be designed as vertically cantilevered over the abutment footing and horizontally cantilevered at the interface with the abutment stem resisting the At-Rest soil pressure from the approach backfill.

F. Wingwalls supported on their own foundations shall be designed as vertically cantilevered retaining walls resisting At-Rest soil pressure from the approach backfill.

4.8.3.2 – SUPERSTRUCTURES

A. The maximum expansion span length for semi-integral abutment bridges is 100 feet.

B. The superstructure main load carrying members and deck shall be designed assuming simple supports at the abutments and continuous over any pier(s) for positive bending within the spans and negative bending and shear at the pier(s).

C. The Curtain wall concrete and reinforcing steel shall be designed horizontally between the beams for the passive soil pressure developed from the thermal expansion of the superstructure. The Curtain wall concrete and reinforcing steel shall be designed vertically between the beams for the passive soil pressure developed from the thermal expansion of the superstructure. See Detail 4.8.3.1. The proposed height of the curtain wall shall be provided to the geotechnical engineer so that the actual soil loading and application point(s) will be given in the **FDR**.

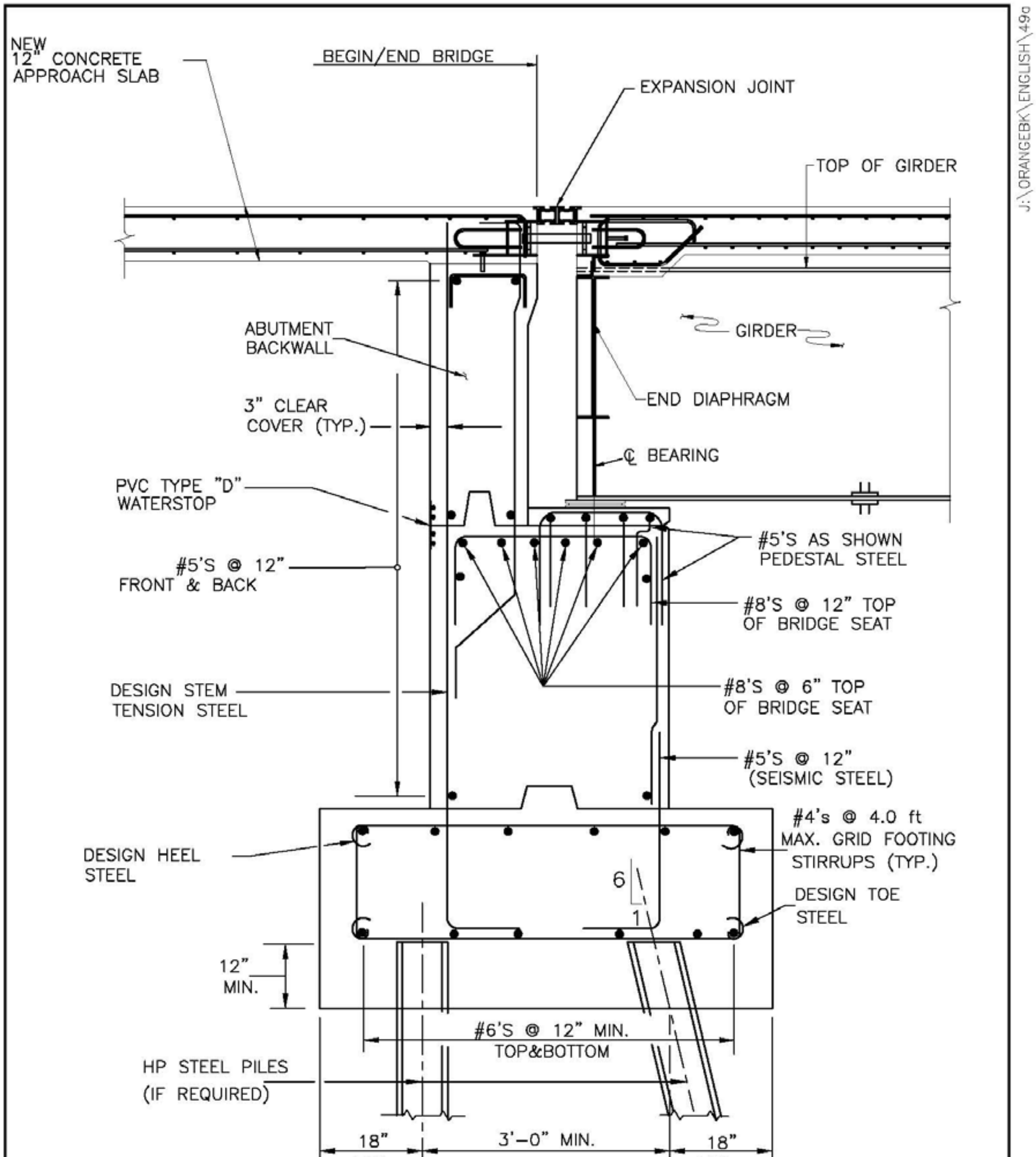
D. Approach slabs shall be designed as reinforced concrete beams as detailed in Subsection 4.8.2.6 with design reinforcing in the bottom running parallel to traffic.

4.9 – CONVENTIONAL STEM ABUTMENTS WITH BRIDGE EXPANSION JOINTS

On original Thruway structures, the conventional abutment consists of solid stem with short pedestals or a backwall with individual tall pedestals supported on a spread footing or pile cap. In both cases, the superstructure rests on bearings connected to the pedestals, and a backwall retains the soil behind the superstructure. On new structures, the conventional abutment would consist of a solid stem with a stepped bridge seat or sloped bridge seat with short pedestals. At the expansion ends of bridges, an expansion joint is mounted between the deck end and the backwall. See Detail 4.9.a. This joint allows for all thermal movement of the bridge. At the fixed ends of bridges, the deck is poured over the backwall to form a cold joint. See Detail 4.9.b.

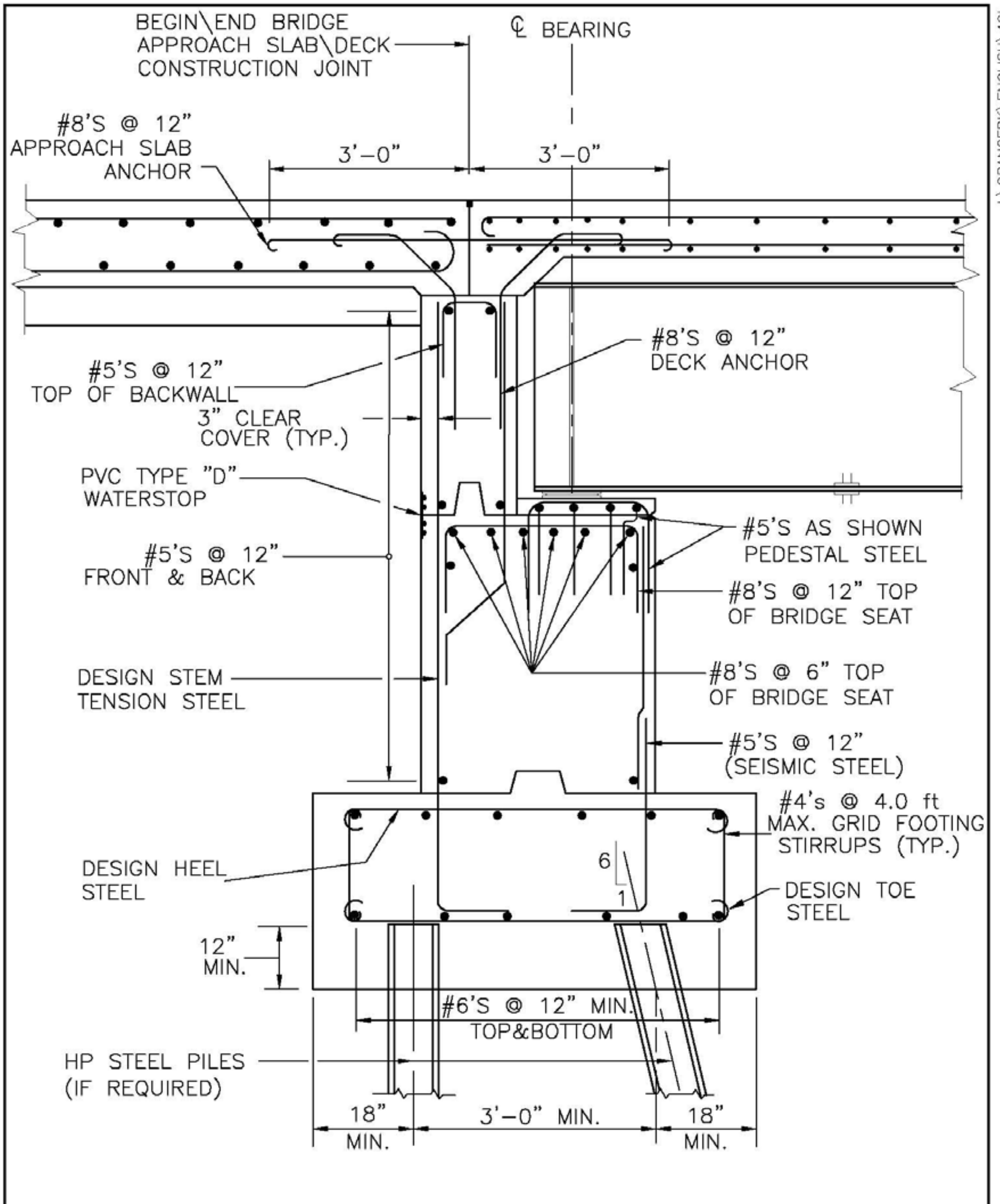
4.9.1 – GUIDELINES ON USE

Whenever possible, it is preferable to eliminate joints by use of one of the previously described abutment types. If this is not possible, the Designer should try to limit the number of joints on a bridge structure to a minimum. The criteria for the use and restrictions of conventional abutments are described in the following subsections.



CONVENTIONAL ABUTMENT SECTION
AT EXPANSION END
DETAIL 4.9.a

N.T.S.



CONVENTIONAL ABUTMENT SECTION
AT FIXED END
DETAIL 4.9.b

N.T.S.

Existing Abutments -On rehabilitation projects, abutments of this type should be modified in one of the following manners:

A. Fill in between pedestals and modify the existing abutments with jointless details. This would be done if a seismic retrofit was required on the existing structure and the existing deck and bearings needed replacement and the abutments did not need replacement. If an existing abutment with joints is being modified to be an abutment for a jointless bridge, the existing U-walls must be removed and replaced with in-line or flared wingwalls. See Subsection 4.7 - Jointless Bridges.

B. Fill in between pedestals and replace existing joint system with new system (see Section 9 - Joints, for replacement joint systems). This would be done if the existing deck, bearings and substructures did not need replacement.

New and Replacement Abutments

In general, expansion joints on new bridges should be avoided due to the maintenance problems experienced in the past. If a new abutment is required with joints, it shall be of the solid stem type with a stepped bridge seat or sloped with short pedestals. For a description of acceptable joint configurations, see Section 9 - Joints.

4.9.1.1 - EXPANSION LENGTHS

The allowable expansion lengths for this type of abutment are controlled by the type of joint system used at the abutment. Refer to Section 9 - Joints.

4.9.1.2 – SOIL CONDITIONS

Site soil conditions shall be analyzed in the **FDR** from existing and new soil borings. The **FDR** (and **HADR** if spanning a waterway) will determine whether the conventional abutments shall be supported on a spread footing or multi-row pile cap. The appropriate foundation type will be based on the geometry, loading, and site conditions at the structure.

4.9.1.3 - HORIZONTAL ALIGNMENT

Straight or curved beams and superstructures will be allowed. On curved structures, thermal movements of the superstructure (both longitudinal and transverse) must be considered in the design of the various elements.

4.9.1.4 - GRADE

There is no maximum grade for bridges on conventional abutments. However, the direction of thermal expansion should always be uphill wherever possible.

4.9.1.5 - SKEW ANGLE

The allowable skew angle for these types of abutments is controlled by the type of joint system used at the abutment. Refer to Section 9 - Joints.

4.9.1.6 - UTILITIES (Refer to Subsection 1.6 - UTILITY COMMUNICATION AND COORDINATION)

All utilities may run through the stem and backwall with appropriate sleeving as necessary.

4.9.2 – DESIGN AND DETAIL CONSIDERATIONS

4.9.2.1 - FOUNDATION TYPES

All conventional abutments shall be supported on a spread footing or multi-row pile cap. Steel H or **CIP** piles shall be used as recommended in the **FDR**. Exact length required shall be specified in the **FDR**. A complete **HADR** (that includes a depth of scour analysis) of the site and proposed structure is required where a structure crosses over a waterway. For these structures, the piles are designed to gain all of their capacity below the scour elevation.

4.9.2.2 - ABUTMENT STEM

The abutment stem thickness shall be determined from the geometry of the bridge seat and backwall. The bridge seat/pedestal must be deep and wide enough to provide room for the appropriate bearing with required edge and end clearances, and allow for thermal movement of the superstructure without coming in contact with the backwall. The backwall shall be a minimum of 18 inches thick. The use of corbels at the back and bottom of the backwall to reduce abutment stem thickness should be avoided. Refer to Subsection 4.5.2.

4.9.2.3 – WINGWALLS

Wingwalls shall be a minimum of 18 inches thick. Wingwalls shall have a constant thickness or be tapered depending on height and length. Tapering the thickness of larger walls from end to end may result in significant concrete quantity savings. In-line wingwalls are the preferred arrangement for conventional abutments. Flared walls shall be used at stream crossings and taller abutments where wingwall length may be significantly reduced by flaring. U-walls should not be used on conventional abutments. U-walls are discouraged because the expanding and contracting approach slab tends to bind on the U-walls. For wingwalls cantilevered off the conventional abutment, see Subsection 4.5.3 and Detail 4.6.2.3. For wingwalls greater self supported on footings or piles and connected to the abutment with a keyed construction, contraction, or expansion joint, see Subsection 4.5.4, and the details in Appendix C.

4.9.2.4 - SUPERSTRUCTURE TYPE

Steel beams, plate girders, or prestressed concrete beams may be used on bridges with conventional abutments. The deck may be either cast-in-place or precast. Refer to Section 3 – Decks, for more information. Refer to Subsection 4.7.3 – Conventional Abutment Bridge Design Procedures.

4.9.2.5 – BEARINGS

When steel beams or plate girders are used in the superstructure, a bearing device shall be designed and detailed to support the superstructure on the abutment bridge seat/pedestal. Refer to Section 8 – Bearings for more information. See Detail 4.9.2.5. Prestressed concrete I-beams require individual rectangular plain rubber bearing pads placed perpendicular to the centerline of the beam. Prestressed concrete box beams require a continuous rectangular plain rubber bearing pad placed at the centerline of bearing of the beams for the full length of the bridge seat. Both types of prestressed concrete beams shall be connected to the abutment with anchor rods. Refer to Section 8 – Bearings.

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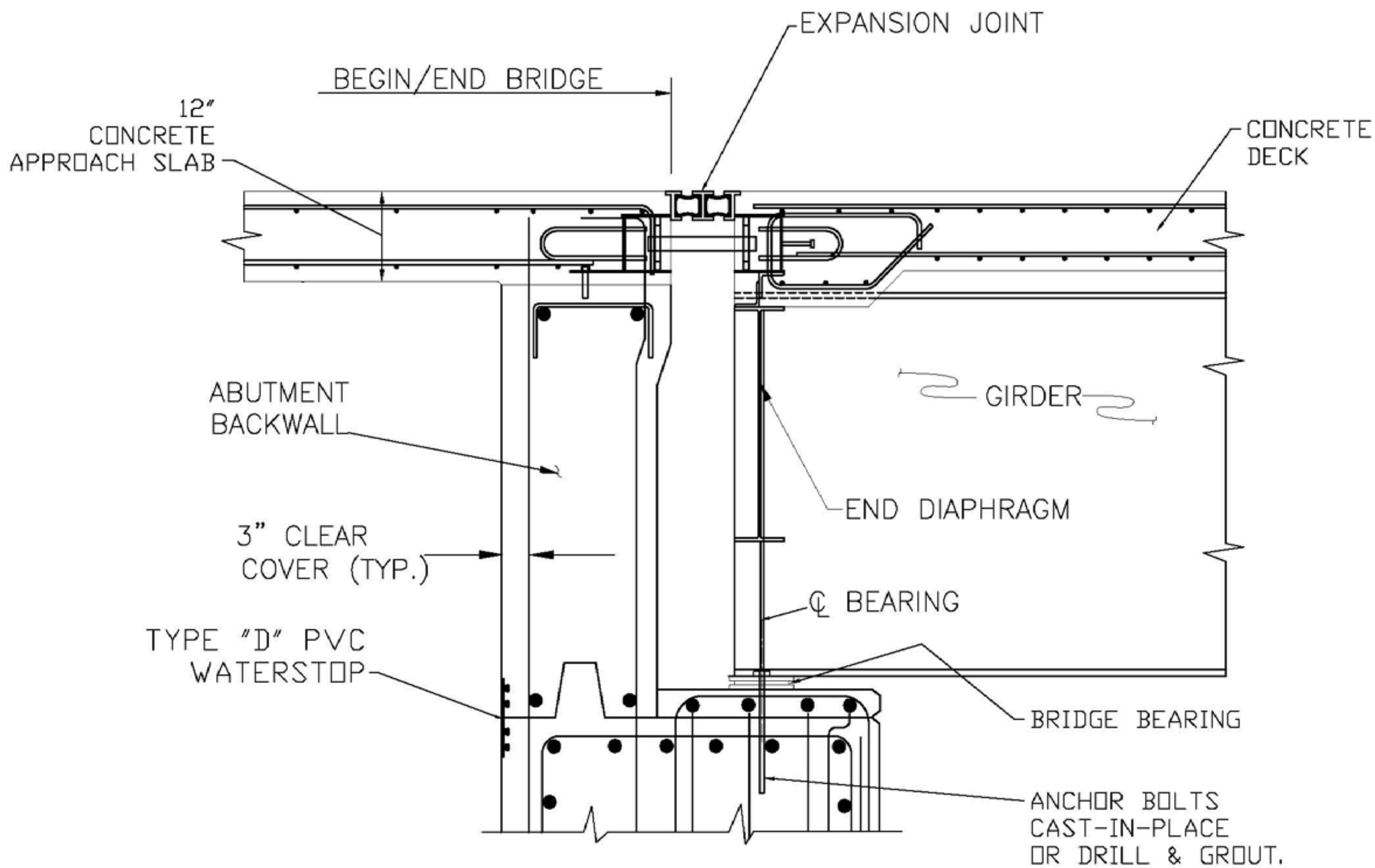


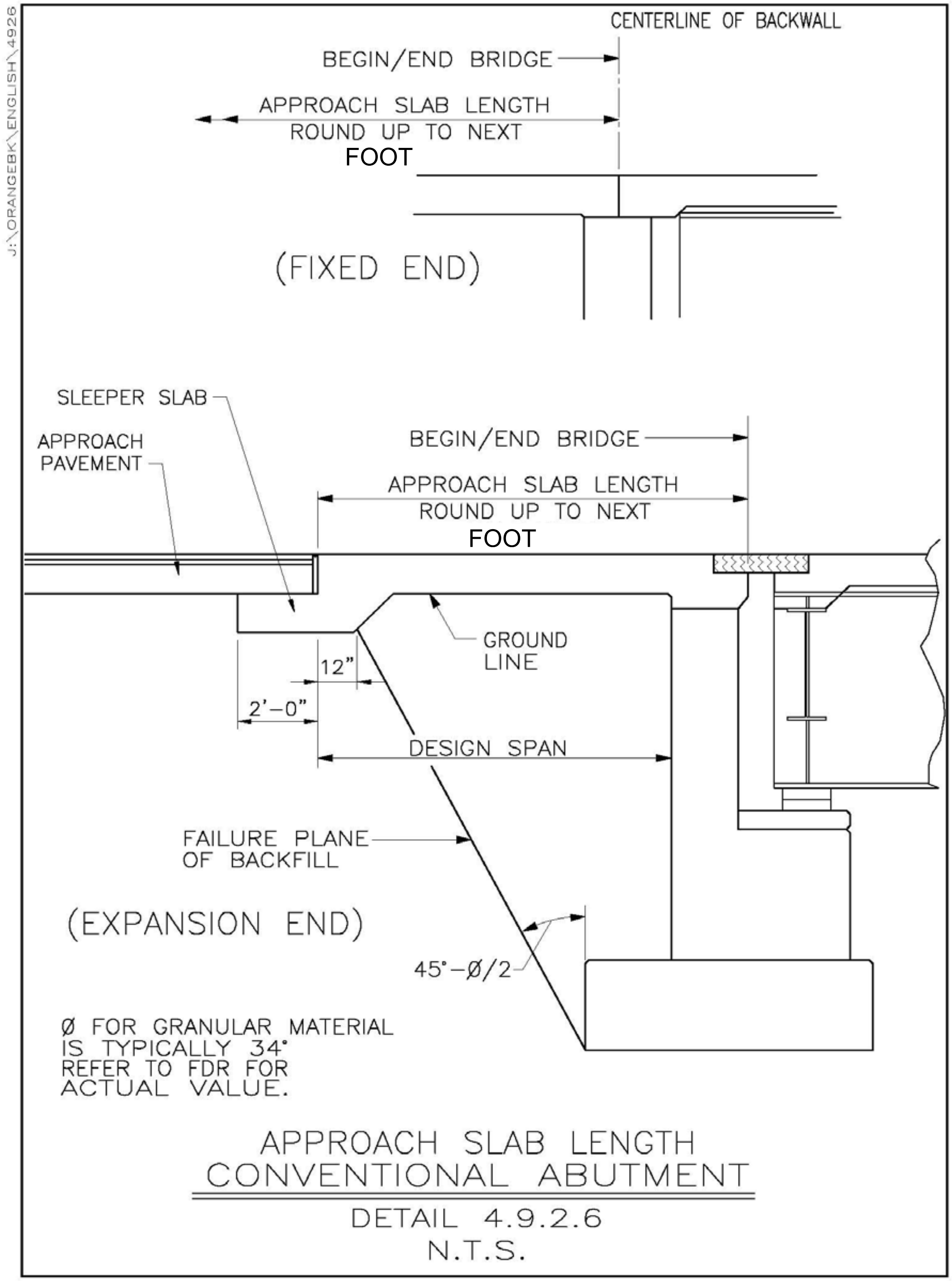
CONVENTIONAL ABUTMENT
W/STEEL SUPERSTRUCTURE
DETAIL 4.9.2.5

N.T.S.

4.9.2.6 - APPROACH SLABS

Approach slabs are required for all conventional abutments. The purpose of the approach slab is to bridge the fill directly behind the abutment and provide a transition from the approach pavement to the bridge deck. Approach slab thickness shall be a minimum of 12 inches. Thickness may be greater depending on the design span length of the approach slab. Approach slab lengths vary depending on the height of the abutment and backfill treatment. In most cases the length is determined based on the intercept of the backfill active failure plane from the bottom and back of the abutment footing to the bottom of the approach slab (ground line). See Detail 4.9.2.6. The end of the approach slab shall be supported on a sleeper slab. This end of the approach slab shall be perpendicular to the centerline of the roadway and run from face-of-guiderail to face-of-guiderail. The abutment end of the approach slab shall be rigidly connected to the abutment backwall as shown in Detail 4.9.2.5.

The approach slab shall incorporate both top and bottom steel reinforcement. The top mats (transverse and longitudinal) of reinforcement shall be a minimum of #5 Bars @ 12 inch spacing in both directions. The bottom mat longitudinal reinforcement shall be designed for traffic loading (reinforcing parallel to traffic) with the design span being a simple span from the back face of the abutment to 1 foot beyond the intersection with the failure plane. The bottom mat transverse reinforcement shall be for temperature only. Refer to the **AASHTO** Specifications for the minimum requirements when the main reinforcement is parallel to traffic.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CENTERLINE OF BACKWALL
BEGIN/END BRIDGE
APPROACH SLAB LENGTH
ROUND UP TO NEXT
FOOT
(FIXED END)
SLEEPER SLAB
APPROACH
PAVEMENT
BEGIN/END BRIDGE
APPROACH SLAB LENGTH
ROUND UP TO NEXT
FOOT
GROUND
LINE
12"
2'-0"
DESIGN SPAN
FAILURE PLANE
OF BACKFILL
(EXPANSION END)
45°-Ø/2
Ø FOR GRANULAR MATERIAL
IS TYPICALLY 34°
REFER TO FDR FOR
ACTUAL VALUE.
APPROACH SLAB LENGTH
CONVENTIONAL ABUTMENT
DETAIL 4.9.2.6
N.T.S.

4.9.2.7 - SLEEPER SLABS

The sleeper slab is a buried concrete foundation used to support the free end of the approach slab. Sleeper slab reinforcement shall be as shown in Detail 4.6.2.7.d.

4.9.2.8 – JOINTS

An expansion joint should be located between the approach slab/backwall and the superstructure deck. The purpose of this joint is to allow for the thermal movement of the superstructure on the bearings. This is a working joint that opens and closes due to thermal expansion and contraction. The longer the span, the greater the opening and closing. The size and type of joint shall be indicated on the plans. Refer to Section 9 – Joints, for selecting the appropriate joint type. This joint shall also define the beginning and end of bridge stationing. See Detail 4.9.2.5. The joint will be anchored to the backwall and deck concrete as described in Section 9 – Joints.

Where the approach slab abuts rigid concrete approach pavement, an expansion joint shall be placed at the end of the approach slab between the rigid pavement and the sleeper slab to allow for the thermal movement of the approach pavement. See Detail 4.6.2.7.d.

4.9.2.9 - SLOPE PROTECTION

Slope protection in front of conventional abutments shall typically be concrete block paving or stamped concrete as described in Subsection 4.3 – Embankment and Slope Protection. See Details 4.3.1.1.a & 4.3.1.1.b. Where a bridge crosses a waterway, the slope protection will be as required in the **HADR**. Bedding requirements, such as geotextile, will be specified in the **FDR**.

4.9.2.10 - FREEBOARD/SUBMERGED INLETS

Structures with reduced freeboard or submerged inlets will be subjected to a greater general and local scour and impact damage. Therefore, structure height should include at least 2 feet of freeboard above **DHW** unless more clearance is required by the **HADR**. Connection of the superstructure to the abutment bridge seat/pedestals through the bearings must be designed to resist uplift and/or horizontal stream loads at the structure as indicated in the **HADR**.

4.9.3 - CONVENTIONAL ABUTMENT BRIDGE DESIGN PROCEDURES

The design of structures with conventional abutments will be as required in the **AASHTO** specifications.

4.9.3.1 – SUBSTRUCTURES

A. Where the abutment stem is supported on piles, the preliminary abutment pile sizes and pile cap dimensions will be given in the **FDR**. The designer shall verify the size and orientation of the piles checking for bearing capacity and uplift. Any uplift requirements shall be relayed to the geotechnical engineer for consideration in the **FDR**.

B. Where the abutment stem is supported on a spread footing, the allowable bearing pressure will be given in the **FDR**. The designer shall proportion the footing for stability (sliding and overturning) of the abutment under construction loading. See Detail 4.7.3.1. The top and bottom reinforcing shall be designed for the completed live load condition.

C. The stem concrete and vertical reinforcing steel shall be designed for a combination of the vertical and horizontal superstructure loads and the At-Rest soil pressure developed against the back of stem from the compacted soil on the abutment heel. See Detail 4.7.3.1.

D. Horizontal reinforcement in the abutment stem shall be for temperature only.

E. Wingwalls cantilevered off the abutment stem shall be designed as vertically cantilevered over the abutment footing and horizontally cantilevered at the interface with the abutment stem resisting the At-Rest soil pressure from the approach backfill.

F. Wingwalls supported on their own foundations shall be designed as vertically cantilevered retaining walls resisting active soil pressure from the approach backfill.

4.9.3.2 – SUPERSTRUCTURES

A. There are no maximum span lengths for this type of structure as long as the thermal movement is accounted for at the expansion joint(s).

B. The superstructure main load carrying members and deck shall be designed assuming simple supports at the abutments and continuous over any pier(s) for positive bending within the spans and negative bending and shear at the pier(s).

C. Approach slabs shall be designed as reinforced concrete beams as detailed in Subsection 4.9.2.6 with design reinforcing in the bottom running parallel to traffic.

4.10 - PIERS

The following subsections present the Authority policy on the design of new piers and the replacement or retrofit of existing piers.

4.10.1 - NEW PIERS

In general, all new piers on mainline, interchange and overhead bridges shall be solid. Aesthetic surface treatments are encouraged. Piers designed with aesthetic details (alternate shapes w/ or w/o "block-outs") should be considered and are encouraged in urban areas. See Subsection 1.1 – Aesthetics. A multi-column structure should be considered when the pier is greater than 30 feet high. In this situation, the potential cost savings of concrete quantity may outweigh additional forming costs and future maintenance costs. Due to safety concerns, column piers are more likely to be considered on mainline crossings over rural routes and non-navigable waterways than on overhead structures, mainline structures over urban routes, railroads or navigable waterways. Piers shall be designed using elastic criteria unless a plastic design is approved by the **DSD**. Generally, plastic design is reserved for taller pier stems (> 30 feet) and some piers designed with aesthetic details. Superstructures will connect with the pier in one of two ways:

1. On conventional continuous bridges, the superstructure is supported on the pier bridge-seat/pedestals with a single line of bearings. Expansion joints in the superstructure at the pier, which would require two lines of bearings should be avoided. In most cases the bearings will be a fixed type directing thermal movement to the ends of the bridge. See Detail 4.10.1.a. On longer bridges with multiple piers,

expansion bearings may be used at one or more piers. Refer to Section 8 – Bearings, for more information.

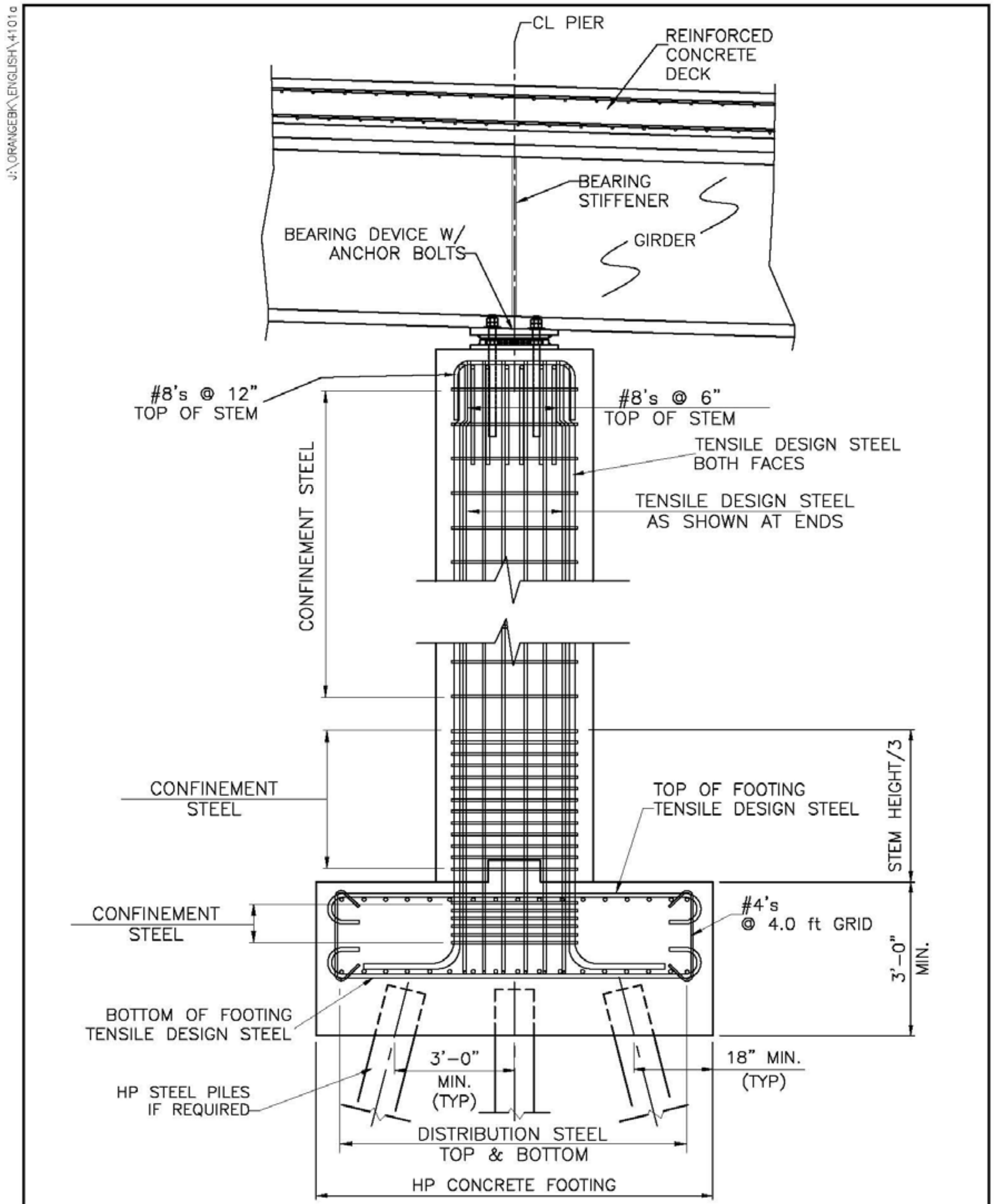
2. On rigid frame multi-span bridges, the girder ends will rest on mortar pads on the pier bridge seat. Vertical reinforcing steel will run up through the girder bottom flanges from the pier stem. Horizontal reinforcing steel will run through the girder webs and the deck concrete will be poured monolithically with the top of pier and girder ends forming a rigid connection. See Detail 4.10.1.b.

4.10.1.1 - PIER LOADING

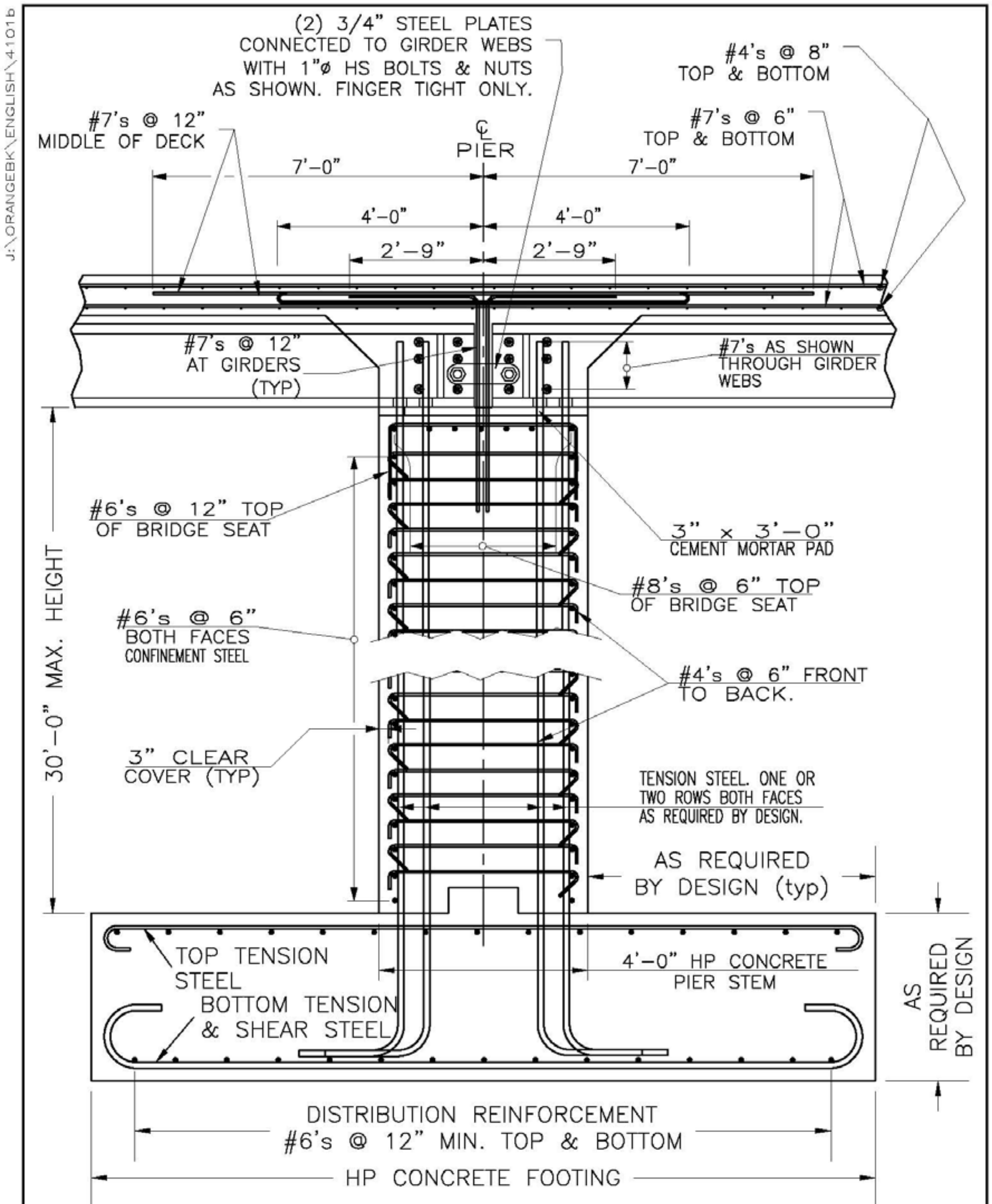
The loading conditions to consider in the design of bridge piers are described below.

Extreme Event 1 Loading (Seismic)

Although all applicable group loadings should be investigated, under most circumstances, **AASHTO 17th Edition** Group VII loading and **AASHTO LRFD 4th Edition** Extreme Event 1 loading will control the design of the various pier components. Under this loading, earthquake loads shall be applied to the pier in addition to the load combinations described in Table 3.22.1A in the **AASHTO 17th Edition** and Table 3.4.1-1 in the **AASHTO LRFD 4th Edition**. The distribution of seismic loading to the pier(s) will be dependent on the type of bridge structure used. On conventional, jointless, and semi-integral abutment bridges, the seismic forces will be assumed absorbed by the fixed pier alone. These structures shall be analyzed with Seisab as fixed at the pier base and pinned at the top.



DETAIL 4.10.1.a
TYPICAL CONVENTIONAL PIER
FOR CONTINUOUS STRUCTURE
N.T.S.



DETAIL 4.10.1.b
TYPICAL INTEGRAL PIER
FOR RIGID FRAME STRUCTURE
N.T.S.

On integral abutment and rigid frame bridges with abutment stem heights ≤ 10 feet, the seismic forces will be distributed between the fixed pier and integral abutments. These structures shall be analyzed with Seisab with all nodes modeled as fixed with the exception of the abutment bases which will be modeled as pinned. The ratio of distribution will be dependent on the relative stiffness differences between the substructure units. On integral abutment and rigid frame bridges with abutment stem heights > 10 feet where the approach backfill has been isolated from the abutment stem (see Detail 4.6.2.6.b) as described in Subsection 4.6.3.1(C), a majority of the seismic forces will be absorbed by the fixed pier, with a small portion distributed to the abutments due to the pile fixity in the soil. The proportion of distribution will depend on the relative stiffness between the pier stem and the abutment piles. These structures shall be analyzed with Seisab with all nodes fixed.

In most cases, live loading from the superstructure shall not be considered when analyzing a new pier during a seismic event. However, some structures based on their functional importance and projected traffic loading at peak hours may warrant the addition of some part or all of this live loading in the analysis of the substructure during a seismic event. The addition of any live loading in the analysis will be only as directed by the **DSD**. The designer is reminded that under the **AASHTO 17th Edition** Group VII loading, allowable stresses may be increased by 33% for the steel and concrete used in the pier.

Extreme Event 2 Loading

In addition to earthquakes, other temporary loads classified as extreme events shall be considered when designing a new pier. These loads are: ice loading, vehicular collision, and vessel collision. Where applicable, new piers shall be analyzed for each of these loads exclusive to each other and in place of the seismic force in **AASHTO 17th Edition** Group VII loading. In addition to this Group VII loading as modified, 50% of the live loads from the superstructure shall be included for each of the applicable event analysis. In the **AASHTO LRFD 4th Edition**, Extreme Event 2 loading shall be investigated in addition to Extreme Event 1 loading. For piers in navigable waterways, the applicable loads are ice loading and vessel collision. New piers in navigable waterways shall be protected from these loads with an approved fender system. The vessel collision loading requirements are dependent on the expected types of vessel traffic at a specific location. Refer to the current AASHTO specifications for the recommended design criteria.

The only Applicable load for piers in non-navigable waterways is ice loading. Refer to the current **AASHTO** specifications for the recommended design criteria.

For Piers adjacent to vehicle roadways the only applicable load is a vehicle collision. In the **AASHTO 17th Edition**, the vehicle collision load is defined as a horizontal force of 400 kips applied at 6 feet above the compacted fill grade at the pier at any angle to the pier. In the **AASHTO LRFD 4th Edition**, the vehicle collision load is defined as a horizontal force of 400 kips applied at 4 feet above the compacted fill grade at the pier at any angle to the pier. This load is applied either entirely to a single column or distributed over the full width of the solid stem (whichever is applicable). This vehicle load need not be applied to the pier if it is protected from a collision by the use of an approved concrete barrier system. This concrete barrier system shall be a minimum of 4'-4" high if it is located 10 feet or less from the face of pier and 3'-6" high if it is located greater than 10 feet from the face of pier. The designer shall analyze both options; designing the pier to resist the vehicle collision load or provide an approved concrete barrier system, and choose the more economical solution.

For piers adjacent to railroads the only applicable load is the same vehicle collision load applied adjacent to vehicle roadways. This vehicle load need not be applied if the pier is protected from a train collision by the use of an approved crash wall system. Refer to Subsection 1.9 and current railroad owner specifications for crash wall geometry and placement requirements.

Rigid Frame Loading

On rigid frame type bridges, as described in Subsection 4.6 – Integral Abutments, the tops of piers are poured monolithically and therefore rigid with the superstructure. In this configuration, superimposed dead load and live load moments are distributed into the pier stem. The amount of distribution depends on the relative stiffness differences between the superstructure and substructures (pier(s) and abutments). As is described under Extreme Events 1&2 Loading, rigid frame piers shall also be designed/checked for those various external loads. However, based on experience, **AASHTO 17th Edition** Group IA loading will usually control the design due to the superstructure moment distribution into the pier(s). An exception to this would be in the case of a combination of seismic and live loading as described in Extreme Event 1 Loading above. As is stated there, the addition of any live loading in the Group VII loading analysis will be only as directed by the **DSD**. The designer is reminded that under Group IA loading, allowable stresses may be increased by 50% for the steel and concrete used in the pier.

4.10.1.2 - PIER DESIGN PROCEDURES

All new piers shall be designed using current **AASHTO** requirements as modified by this Manual. All applicable group loading conditions shall be analyzed with specific consideration to the loading conditions described above.

A. Where the pier stem is supported on piles, the preliminary pile sizes and pile cap dimensions will be given in the **FDR**. The designer shall verify the size and orientation of the piles checking for bearing capacity and uplift. Any uplift requirements shall be relayed to the geotechnical engineer for consideration in the **FDR**.

B. Where the pier stem is supported on a spread footing, the allowable bearing pressure will be given in the **FDR**. The designer shall proportion the footing for stability (sliding and overturning) of the pier under construction loading. The top and bottom reinforcing shall be designed for the completed live load condition.

C. The stem concrete and vertical reinforcing steel shall be designed for the moments, shears, and vertical loads as described in Subsection 4.10.1.1 above.

D. Horizontal reinforcement in the pier stem shall be for seismic confinement as described in

AASHTO 17th Edition Division IA Subsection 6.6.2 and **AASHTO LRFD 4th Edition** Subsection 5.10.11.

4.10.1.3 - PIER PROTECTION

Waterways

Piers in navigable waterways shall be protected from vessel impact and ice loading with an approved fender system. Pier nosing as described in 4.10.1.3.1 below, shall be incorporated into the fender system and designed to deflect and/or break up ice flow. Heavy stone fill, piles, rock anchors, or any other system that is designed to protect the pier foundation from being undermined shall be used to protect against scour. A hydraulic analysis will be required at these locations. Recommendations in the **HADR** shall be incorporated into the pier design. Piers in non-navigable waterways shall be similarly protected excluding the fender system.

Travel Lanes

The use of an approved guiderail system may be required at new piers adjacent to vehicular traffic as described in Subsection 1.7. If the pier design incorporates the use of the concrete barrier system as described in Subsection 4.10.1.1, standard transition/termination details shall be used.

Railroad Tracks

The use of an approved crash wall system may be required at new piers adjacent to railroad tracks as described in subsection 1.9. As stated in Extreme Event 2 Loading, refer to the railroad owner specifications when determining crash wall geometry and placement requirements.

4.10.1.3.1 - PIER NOSING

For stream bridges, a recommendation shall be displayed on the Preliminary Plans regarding the need and type of ice breaker for pier nosing (if required). This information can be found in the **HADR**. If required, the ice breaker shall consist of a steel angle or other device secured to the concrete by means of suitable anchors. All structural steel and studs for pier nosing shall be hot dipped galvanized. The Concrete Anchor Stud Note in Appendix B shall be included in the plans when pier nosing is used.

4.10.2 - EXISTING PIERS

Existing piers on mainline, interchange and overhead bridges are typically multi-column piers integral with a concrete or steel cap beam. Fracture-critical details shall be eliminated on all major rehabilitation projects. Steel cap beams could be replaced with a concrete pier cap or an entire new solid pier. An existing pier shall be evaluated at the time of project scoping to determine if it is economical to rehabilitate or replace. Refer to Subsection 4.10.1 - New Piers. An existing pier can remain in place if it is structurally sound (minimal repairs), geometrically compatible with the rehabilitated structure (height, location and skew), and able to carry the revised loading from the superstructure. Epoxy injection of cracks in substructure units as a means of repair is not done on Thruway owned or maintained structures. Damaged concrete shall be repaired using an approved concrete repair detail. Refer to Thruway Standard Detail sheets.

4.10.2.1 - EXISTING PIER LOADING

Existing piers that do not meet **AASHTO** requirements shall be modified or replaced to meet those requirements, when included in the project scope. Modification of the existing pier shall consider reorientation of the bearings (placing expansion bearings at the pier), and/or bearing replacement (with seismic isolation bearings) to reduce the design forces. All loading criteria as described in Subsection 4.10.1.1 shall apply when analyzing an existing pier for reconstruction, retrofit or replacement.

4.10.2.2 – EXISTING PIER PROTECTION

Existing piers shall be protected from the various loads in the same manner as new piers. An inspection and evaluation of the existing protection and load carrying capacity of in-place structures and fill materials shall precede this design work.

4.10.2.3 – EXISTING PIER REPAIRS

On most bridge rehabilitation projects, existing piers will require at least some amount of concrete repair. Pier columns and caps within the splash zone and/or below superstructure expansion joints typically show moderate to heavy delamination of the concrete and degraded reinforcing steel. The amount and locations of these areas will vary from project to project. For this reason it is important for designers to perform 100% hands-on inspections of all pier members looking for spalled areas and exposed reinforcing steel, and sounding the concrete to identify hollow areas. This information should then be mapped out on sketches of the pier elements for analysis as described below.

Typically these areas can be repaired using Class D repairs that remove the concrete in the affected area to a minimum depth of 1½ inches behind the reinforcing steel. This depth will vary if the unsound concrete extends to a depth beyond the 1½ inch minimum. Heavily corroded reinforcing steel is then replaced and new concrete is cast to complete the repair. Refer to the Thruway Structures Standard Sheet "Class D Concrete Repair Details" in ProjectWise for more information on repair procedures. If the designer suspects that the integrity of the concrete below these depths is compromised he may elect to have cores taken in selected areas to gather further information.

Pier Cap Evaluation and Repair Procedure Guidelines

After the site investigation and mapping as described above, the designer must evaluate the pier for repair considering the stability and strength of the pier during construction assuming that all repairs are completed concurrently. If the designer determines through structural evaluation that the pier cap will not be capable of supporting the existing loads during repair procedures, he may have to consider repairing the cap in sections with shoring supporting the cap. The designer must consider that the dead loads from the pier cap and the dead and live loads from the superstructure will be present during the repair procedures in his analysis. If a repair/shoring process is required by analysis of the pier cap, these specific details and procedures must be detailed on the plans. If the deteriorated condition of the pier cap prevents shoring to its underside, the designer should detail a shoring system that will completely support the superstructure during the repairs. These details too should be shown on the plans. The plans should also indicate all assumed loads and instruction given that the contractor may design an alternate shoring system at no additional cost to the Authority.

Pier Columns Evaluation and Repair Procedure Guidelines

As with pier caps, pier columns must also be evaluated for stability and strength during repair procedure determination. If the designer determines through structural evaluation that a pier column will not be capable of supporting the existing loads during repair procedures assuming that all repairs are completed concurrently, he may have to consider repairing the column with shoring under the pier cap. If the moment connections between the pier cap and columns are deteriorated or if the pier cap cannot support shoring due to its condition, the designer will have to consider shoring the superstructure separately as described above. To reiterate, if a repair/shoring process is required by analysis of the various pier elements, these specific details and procedures must be detailed on the plans with assumed loads and the option for the contractor to design his own shoring system at no additional cost to the Authority.

4.11 - CONCRETE SEALANTS

Authority policy is to apply a concrete sealant to all concrete substructure elements. The type of sealant used depends upon where it is to be used, as detailed in the following subsections. Refer to the Structures Estimate Template in ProjectWise for approved sealant item numbers.

4.11.1 - SOLID COLOR PROTECTIVE CONCRETE SEALER

Solid Color Protective Concrete Sealer shall be used on all exposed surfaces of substructure concrete (excluding underside of pier caps) on all existing overhead substructures and mainline substructures that previously had this type of sealer applied.

This type of sealer shall also be used on new structures when requested by the **DBE**.

4.11.2 - CLEAR PENETRATING SEALER

Clear penetrating sealer shall be used on all exposed surfaces of substructure concrete (excluding underside of pier caps) on all new overhead and mainline substructures and existing substructures that previously had this type of sealer applied.

4.12 - CONCRETE JOINTS

There are several reasons to place joints in a concrete mass. There are also several types of joints commonly used. The following subsections detail those joints and their uses. See Appendix C for joint details.

4.12.1 - TYPES OF JOINTS

Construction Joint:

A. Reinforcement goes through joint.

B. Joint has a shear key.

C. Joint has a Type "D" water stop.

Contraction Joint:

A. Reinforcement does not go through joint.

B. Joint has a shear key.

C. Joint has a Type "D" water stop.

Expansion Joint:

A. Reinforcement does not go through joint.

B. Joint has a shear key.

C. Joint has a Type "E" water stop.

D. Joint has a layer of premolded bituminous joint filler or closed cell form.

* - The Type "D" water stop may be omitted from the construction and contraction joints if leakage is unlikely or where staining due to leakage would not be objectionable.

** - On integral abutments, the expansion joint used between the abutment and wingwall (separate foundations) would not have a shear key.

4.12.2 - APPLICATION OF JOINTS

The following subsections detail the locations where the different types of concrete joints are used.

4.12.2.1 - HORIZONTAL JOINTS

All horizontal joints in substructure concrete are construction joints unless otherwise specified.

4.12.2.2 - VERTICAL JOINTS

Short stem abutment with in-line wingwall (footings on soil):

A. Contraction joint.

B. Joint does not extend through footing.

Short stem abutment with in-line wingwall (on rock or piles):

A. Construction joint.

B. Joint does not extend through footing.

Short stem abutment with flared wingwall (length ≤ 6 feet):

A. Construction joint.

B. Joint does not extend through footing.

Short stem abutment with flared wingwall (length > 6 feet):

A. Contraction joint.

B. Joint does not extend through footing.

Short stem abutment with U-walls:

A. Construction joint.

B. Joint does not extend through footing.

Integral abutment with in-line wingwall (length ≤ 6 feet):

A. Construction joint.

B. Wingwall suspended from abutment.

C. No footing.

Integral abutment with in-line wingwall (length > 6 feet):

A. Expansion joint - no shear key.

B. Wingwall on independent foundation.

Integral abutment with flared wingwall (length ≤ 6 feet):

A. Construction joint.

B. Wingwall suspended from abutment.

C. No footing.

Integral abutment with flared wingwall (length > 6 feet):

A. Expansion joint - no shear key.

B. Wingwall on independent foundation.

4.12.2.3 - VERTICAL JOINTS AT OTHER LOCATIONS

Construction joints

A. At 30 foot intervals in the abutment and pier stem if staining through shrinkage cracks would be objectionable.

B. Joint does not extend through footing

Contraction Joints

A. At 30 foot intervals in wingwalls and retaining walls.

B. Joint does not extend through footing

Expansion Joints

A. At 90 foot intervals in wingwalls and retaining walls

B. Joint extends through footing

C. No expansion joints are allowed in abutment or pier stems unless there is a complete separation of the superstructure above it.