C6.5 Abutments

C6.5.1.1.1 Integral

20 October 2014 ~ Integral abutment limits for horizontally curved steel girder bridges
Integral abutment policy has been adjusted based on the latest ISU research (see report reference below). The purpose of the research was to investigate the behavior of horizontally curved bridges with integral abutment and semi-integral abutment bridges with a specific interest in the response to changing temperatures. This was done by monitoring and evaluating the behavior of six in-service, horizontally curved, steel-girder bridges with integral and semi-integral abutments. The six bridges are located at the intersection of I-80, I-35, and I-235 on the northeast side of Des Moines, also known as the northeast mix-master. The long-term objective of this effort is to establish guidelines for the use of integral abutments with curved girder bridges. In light of the monitored in-service bridge behavior and resulting discussion between IDOT and ISU, the integral abutment limits were extended to horizontally curved steel girder bridges with minimum radii of 900 feet and a maximum bridge width of 44 feet from gutter line to gutter line. These same parameters are applicable to horizontally curved girder bridges with semi-integrals abutments. The limits with respect to semi-integral abutments were not included in the main body of the BDM since these abutments types have not yet been addressed in the manual.

Greimann, L.F., B.P. Phares, Y. Deng, G. Shryack and J. Hoffman. *Field Monitoring of Curved Girder Bridges with Integral Abutments*, Final Report, FHWA Pooled Fund Study TPF-5(169), Iowa State University InTrans Project 08-323, January 2014. (Available online at https://intrans.iastate.edu/app/uploads/2018/03/curved_girder_integral_abutments_w_cvr.pdf).

16 August 2007 ~ Parameter study and discussion
In order to adjust integral abutment policy to LRFD and the latest ISU research (Abendroth and Greimann 2005) a parameter study was conducted to determine the effects of bridge length, end span length, skew, and prebore depth. For PPCB bridges the study specifically worked with the new A-D and BTB-BTD beams with an end span of maximum beam length. For CWPG members the basic condition was taken to be an end span of 150 feet or the maximum end span that would result in a pile structural resistance at Structural Resistance Level – 1, the LRFD equivalent to a 6 ksi axial stress under service load design.

In LRFD there is a single check for combined forces (axial load and bending) rather than the two (stability and yield) in service load design. The parameter study included the LRFD combined forces check and a ductility check (Abendroth and Greimann 2005). Generally the LRFD combined forces check gave results less conservative but similar to those from the stability and yield checks in service load design. A different, more conservative way of evaluating the effects of pile skew (similar to Abendroth and Greimann 2005), however, gave results essentially the same as those for past parameter studies.

With the latest ISU recommendations for ductility, the ductility check generally will not control the design, but use of the recommended seismic plate ratios requires that several H-pile shapes be avoided. Ratios for flange plates $b_f/2t_f$ above 11.0 do not work for Grade 50 steel, and the policy recommendation is to set an upper limit of 10.5, but either limit results in the same list of acceptable H-piles: HP 10x57, HP 12x74, HP 12x84, HP 14x102, and HP 14x117. The HP 14x102 shape should be avoided because it generally is not readily available.

Because of the less conservative biaxial bending and ductility checks, bridges with minimal skew may have greater lengths than present policy allows. Limits other than bridge length may be appropriate, however. Considering the type and performance of present pavement joints, the maximum bridge length for zero skew was set for approximately 1.55 inches maximum movement each way, assuming that the bridge is fixed at mid-length. At the maximum bridge length the pavement joints should be of the CF-3 type [OD SRP PV-101 and RK-20]. At shorter bridge lengths the CF-2 or CF-1 joints should be used within the guidelines on the standard road plan [OD SRP RK-20].

In general, the parameter study verified the previous study conducted for the service load design manual. The information for the Bridge Design Manual tables, however, was modified to better fit the LRFD format.

Reference
Abendroth, R.E. and Greimann, L.F. (2005) *Field Testing of Integral Abutments, Final Report HR-399.* Center for Transportation Research and Education (CTRE), Iowa State University, Ames, Iowa. Available online at <https://iowadot.gov/research/reports/Year/2005/fullreports/hr399.pd>.

01 January 2016 (Updated 01 July 2016) ~ Integral abutment piles with shallow bedrock requiring rock coring

Rock coring for integral abutment bridges shall be required for Case 1 and 2:

1. Short bridges of 130 feet or less without prebore per BDM 6.5.1.1.1 which have less than 10 feet of soil between the bottom of abutment footing and top of bedrock.
2. All bridges with 10 feet of prebore and 15 feet or less of soil between the bottom of abutment footing and top of bedrock.

Check with the Chief Structural Engineer regarding the feasibility of using integral abutments under these situations.

In the text below the term rock socket is specifically reserved for the bottom 3 feet of the rock core that is filled with concrete in order to lock the piles in place. Any rock coring above the bottom 3 feet is referred to as cored rock prebore. Any prebore of the soil above the top of bedrock is referred to as soil prebore.

Case 1: Because installation of piling requires rock coring these short bridges will be constructed with 10 feet of prebore as is typical for bridges longer than 130 feet. The soil prebore and any cored rock prebore extending to a depth of 10 feet below the bottom of abutment footing shall have a minimum diameter 4 inches greater than the maximum cross-sectional dimension of the pile. The rock socket shall have a minimum diameter 2 inches greater than the maximum cross-sectional dimension of the pile. [Example: For HP10 steel piles specify a minimum of 18 inches for the diameter of prebored holes in soil and rock and a minimum of 16 inches for the diameter of rock sockets.] The 10 feet deep prebored hole shall be filled with bentonite slurry.

Case 2: The minimum depth of prebore is 10 feet. The maximum depth of prebore is 15 feet per BDM 6.5.4.1.1. In general, it is advisable to set the depth of prebore to the top of bedrock when the depth of bedrock is between 10 and 15 feet, otherwise 10 feet shall be used. The soil prebore and any cored rock prebore extending to a depth of 10 feet below the bottom of abutment footing shall have a minimum diameter 4 inches greater than the maximum cross-sectional dimension of the pile. Any soil prebore below 10 feet shall also have a minimum diameter 4 inches greater than the maximum cross-sectional dimension of the pile. The rock socket shall have a minimum diameter 2 inches greater than the maximum cross-sectional dimension of the pile. [Example: For HP10 steel piles specify a minimum of 18 inches for the diameter of prebored holes in soil and rock and a minimum of 16 inches for the diameter of rock sockets.] The prebored hole shall be filled with bentonite slurry.

Where rock coring is required the minimum depth of the rock socket shall be 3 feet into rock. The piles shall be seated by driving the pile to the target driving resistance. A minimum of 3 feet of Class C structural concrete shall be placed in the bottom of the rock socket to lock the base of the piles in place. The concrete may be placed before or after the piles are seated. The contractor shall make a reasonable effort to clean the bottom of the socket before inserting pile or placing concrete. If piles are seated after the concrete is placed, then a retarder may be required to ensure the concrete remains plastic while the piles are driven. The contractor shall brace the piles in the correct position until the concrete achieves 4 ksi compressive strength. The required unsupported pile height, typically 10 feet to 15 feet below the bottom of the abutment footing, shall be maintained by filling the prebored hole with bentonite slurry according to IDOT SS 2501.03, Q. If necessary, the designer may require the contractor to place saturated sand from the top of the concrete in the rock socket to the bottom of the unsupported pile height.

When rock coring is required the designer shall generally indicate the following on the Longitudinal Section Along Centerline Approach Roadway of the Situation Plan sheet:

- Type, size, and length of pile
- Bottom of abutment footing elevation
- Bottom of prebored hole elevation, diameter of prebored hole, and length filled with bentonite slurry
- Anticipated top of rock elevation
- Minimum bottom of rock socket elevation, socket diameter, and length filled with concrete

Note E184 Prebored Holes in the General Notes shall reference rock coring notes on the abutment sheets. For example:

> THE BRIDGE CONTRACTOR SHALL PREBORE HOLES FOR ABUTMENT PILES. HOLES SHALL BE BORED TO THE ELEVATIONS SHOWN ON THE "LONGITUDINAL SECTION ALONG CENTERLINE APPROACH ROADWAY" ON DESIGN SHEET _____. SEE ABUTMENT PILING NOTES ON DESIGN SHEET _____ FOR SPECIAL ROCK CORING REQUIREMENTS.

The following notes or similar ones shall be included with the abutment plan sheets:

> THE _____ INCH DIAMETER PREBORED HOLE IN THE SOIL SHALL EXTEND FROM THE BOTTOM OF THE ABUTMENT FOOTING TO THE TOP OF THE _____ STRATUM AS SHOWN ON THE SOIL PROFILE SHEET WHICH IS ANTICIPATED AT ELEVATION _____. THE _____ INCH DIAMETER ROCK SOCKET SHALL BE CORED TO ELEVATION _____ OR LOWER SUCH THAT THE BOTTOM OF THE SOCKET SHALL BE AT LEAST 3 FEET INTO THE _____ STRATUM. THE CONTRACTOR SHALL CLEAN THE BOTTOM OF THE ROCK SOCKET BEFORE INSERTING PILES OR PLACING CONCRETE.
>
> SEAT THE PILING IN THE BEDROCK BY DRIVING IT TO THE TARGET DRIVING RESISTANCE. THE NUMBER OF HAMMER BLOWS SHALL BE LIMITED TO PREVENT DAMAGE TO THE PILING. THE SOCKET SHALL BE BACKFILLED WITH 3 FEET OF CLASS "C" STRUCTURAL CONCRETE BEFORE OR AFTER DRIVING PILES. CONCRETE PLACED BEFORE PILES ARE DRIVEN SHALL REMAIN PLASTIC UNTIL PILE DRIVING IS COMPLETE. RETARDER MAY BE REQUIRED AS DIRECTED BY THE ENGINEER. PILES SHALL BE BRACED IN THE CORRECT POSITION UNTIL CONCRETE HAS REACHED 4 KSI COMPRESSIVE STRENGTH.
>
> IF BEDROCK IS NOT ENCOUNTERED ABOVE ELEVATION _____, THEN NO ADDITIONAL PREDRILLING IN THE SOIL IS REQUIRED AND NO "CORING ROCK SOCKET" QUANTITY SHALL BE MEASURED FOR PAYMENT. THE LENGTH OF PREDRILLING SHALL BE MEASURED AND PAID FOR AT THE PRICE BID FOR "PREBORED HOLES".
>
> PREDRILLED HOLES SHALL BE FILLED WITH BENTONITE ABOVE ELEVATION _____.

When rock coring is specified designers shall include bid item 2599-999909 CORING ROCK SOCKET with units of linear feet. Designers shall include bid item 2501-6335010 PREBORED HOLES to account for predrilling in soil and placement of bentonite slurry. A rock coring bid item reference note similar to the following shall be included:

> THE QUANTITY OF "CORING ROCK SOCKET" IS BASED ON THE ANTICIPATED TOP OF ROCK ELEVATION AND CORING _____ FEET INTO THE ROCK FOR EACH PILE IN THE _____ ABUTMENT. THE NUMBER OF LINEAL FEET OF ROCK CORING WILL BE MEASURED IN THE FIELD AND PAID FOR AT THE CONTRACT UNIT PRICE BID PER LINEAL FOOT. THE PRICE BID INCLUDES THE COST OF PREDRILLING _____ INCH DIAMETER ROCK SOCKETS A MINIMUM _____ FEET INTO _____. THIS ITEM INCLUDES THE COST OF DISPOSAL OF EXCAVATED MATERIAL AND FURNISHING AND PLACING CLASS "C" STRUCTURAL CONCRETE TO BACKFILL THE SOCKET TO THE REQUIRED ELEVATION. [IT ALSO INCLUDES THE COST OF BACKFILLING THE HOLES FROM THE TOP OF SOCKET CONCRETE TO THE BOTTOM OF PREBORE ELEVATION WITH SATURATED SAND MEETING THE REQUIREMENTS OF STANDARD SPECIFICATION 4110, GRADATION 1.] IF ABUTMENT PILES CAN BE DRIVEN TO A MINIMUM _____ FEET OF LENGTH, NO MEASUREMENT OR PAYMENT WILL BE MADE FOR "CORING ROCK SOCKET".

LRFD Integral Abutment Example

Given: Four-span PPCB bridge, 105-120-120-105-foot spans, 450-foot length, 20-degree skew
Five-BTC cross section, beam spacing 9'-3
Integral abutments
Soils Design Unit recommendation: H-piles end bearing on rock
Total abutment factored vertical load (includes IM) = $\Sigma\eta_i\gamma_i P_i$ = 1200 kips
The thermal origin is assumed to be at the center of the bridge.

Select HP 10x57 for integral abutments for a PPCB bridge [BDM 6.5.1.1.1]. Nominal structural resistance for an HP 10x57 at SRL-2, maximum in end bearing: P_n = 365 kips [BDM Table 6.2.6.1-1]. Note, however, that this maximum may not be permissible based on integral abutment limits, which may be less than SRL-2 [BDM Table 6.5.1.1.1-1].

Check maximum bridge length. Interpolate for 20-degree skew [BDM Table 6.5.1.1.1-1].

L_{max} = 525 + [(20-15)/(30-15)](475-525) = 508 feet; 508 feet > 450 feet, OK

Check integral abutment limit on nominal structural resistance.

Table 6.5.1.1.1-1 indicates that interpolation will not lead to 365-kip resistance, but shorter-than-maximum end span will permit some increase in extrapolated value.

Try 10-foot prebore with interpolation for skew; extrapolate for resistance with 120-foot end span.

P_n = 324 + [(20-15)/(30-15)](243-324) = 297 kips

Increase P_n for shorter-than-maximum end span.

P_n = (120/105)(297) = 339 kips, which is close to 365 kips. (Using a 15-foot prebore would permit the full 365 kips but, as the next step shows, the additional prebore would not reduce the number of piles.)

Determine number of piles

Number of piles, n = $\Sigma\eta_i\gamma_i P_i/\varphi_c P_n$ = 1200/(0.6*339) = 5.9, use 6

Check minimum: 5 beams require 5 piles, OK; maximum pile spacing is 8 feet, use 6.

Factored load per pile, P_u = 1200/6 = 200 kips

This completes the structural check for the integral abutment. However, the complete design requires contract length and driving target for the piles. See "LRFD Pile Design Examples ~ 2013" for additional steps to complete a typical pile design considering site soil classification and construction control method.

2011 ~ Abutment Backfilling at MSE Walls

During construction of the I-235 overpasses the Soils Design Unit and Construction and Materials Bureau decided not to place the abutment backfill sand with the flooding method given on standard sheets [OBS SS 1007D, 1007E] when the abutment was near an MSE wall. The primary reason was that the flooding water did not flow through the abutment subdrain. In 2011 the question of backfill flooding again was asked for the Wesley Parkway Bridge over I-29. The decision reached for the bridge and for standard practice was that when the abutment is within the MSE reinforced zone, flooding should not be used. The usual geotextile fabric, porous backfill, and abutment subdrain should be placed to divert deicer chemicals from the MSE wall straps. The abutment backfill should be the same material as placed for the MSE wall, and it should be placed in lifts and compacted in the same way as the MSE wall backfill material. Site constraints may dictate that the abutment subdrain be tied into the MSE wall subdrain. The designer will need to include a note on the plans that prohibits flooding of the backfill.

C6.5.1.1.1 Stub

2011 ~ Abutment Backfilling at MSE Walls
During construction of the I-235 overpasses the Soils Design Unit and Construction and Materials Bureau decided not to place the abutment backfill sand with the flooding method given on standard sheets [OBS SS 1007D, 1007E] when the abutment was near an MSE wall. The primary reason was that the flooding water did not flow through the abutment subdrain. In 2011 the question of backfill flooding again was asked for the Wesley Parkway Bridge over I-29. The decision reached for the bridge and for standard practice was that when the abutment is within the MSE reinforced zone, flooding should not be used. The usual geotextile fabric, porous backfill, and abutment subdrain should be placed to divert deicer chemicals from the MSE wall straps. The abutment backfill should be the same material as placed for the MSE wall, and it should be placed in lifts and compacted in the same way as the MSE wall backfill material. Site constraints may dictate that the abutment subdrain be tied into the MSE wall subdrain. The designer will need to include a note on the plans that prohibits flooding of the backfill.

C6.5.4.1 Integral abutments

C6.5.4.1.1 Analysis and design

April 2013 ~ Guidelines for mass concrete
Previous guidelines for mass concrete have been misinterpreted due to aesthetic shapes and unusual configurations. The revised statement in this article

> "… when the least dimension of any element exceeds 4.5 feet".

is intended to identify the large volumes of concrete that need to be considered mass concrete. There may be cross sections along a tapered abutment that meet the above definition of mass concrete and those that do not, but the sections that meet the definition classify the abutment as mass concrete, and therefore require the mass concrete provisions.

C6.5.4.2 Stub abutments

C6.5.4.2.1 Analysis and design

April 2013 ~ Guidelines for mass concrete
Previous guidelines for mass concrete have been misinterpreted due to aesthetic shapes and unusual configurations. The revised statement in this article

> "… when the least dimension of any element exceeds 4.5 feet".

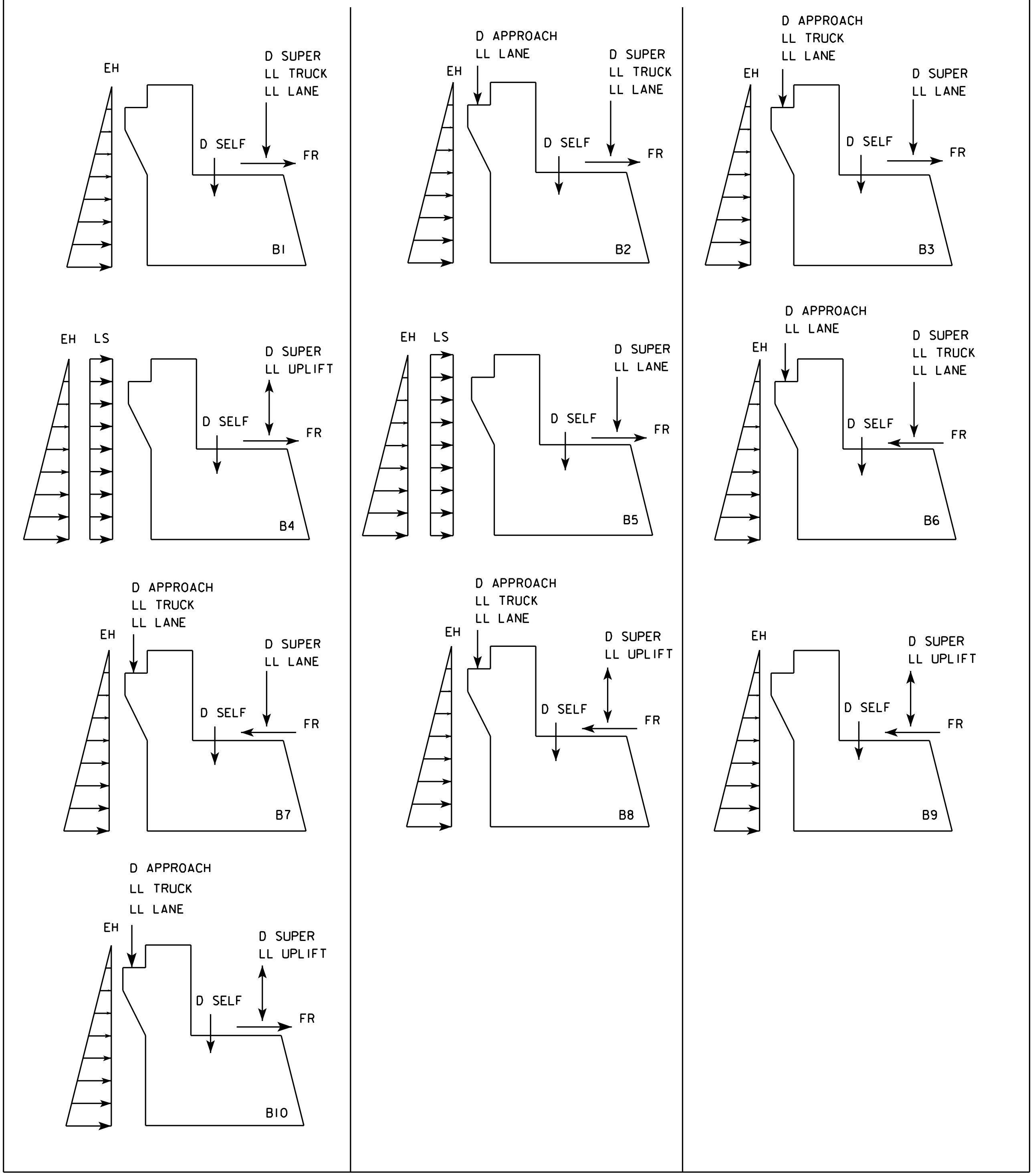
is intended to identify the large volumes of concrete that need to be considered mass concrete. There may be cross sections along a tapered abutment that meet the above definition of mass concrete and those that do not, but the sections that meet the definition classify the abutment as mass concrete, and therefore require the mass concrete provisions.

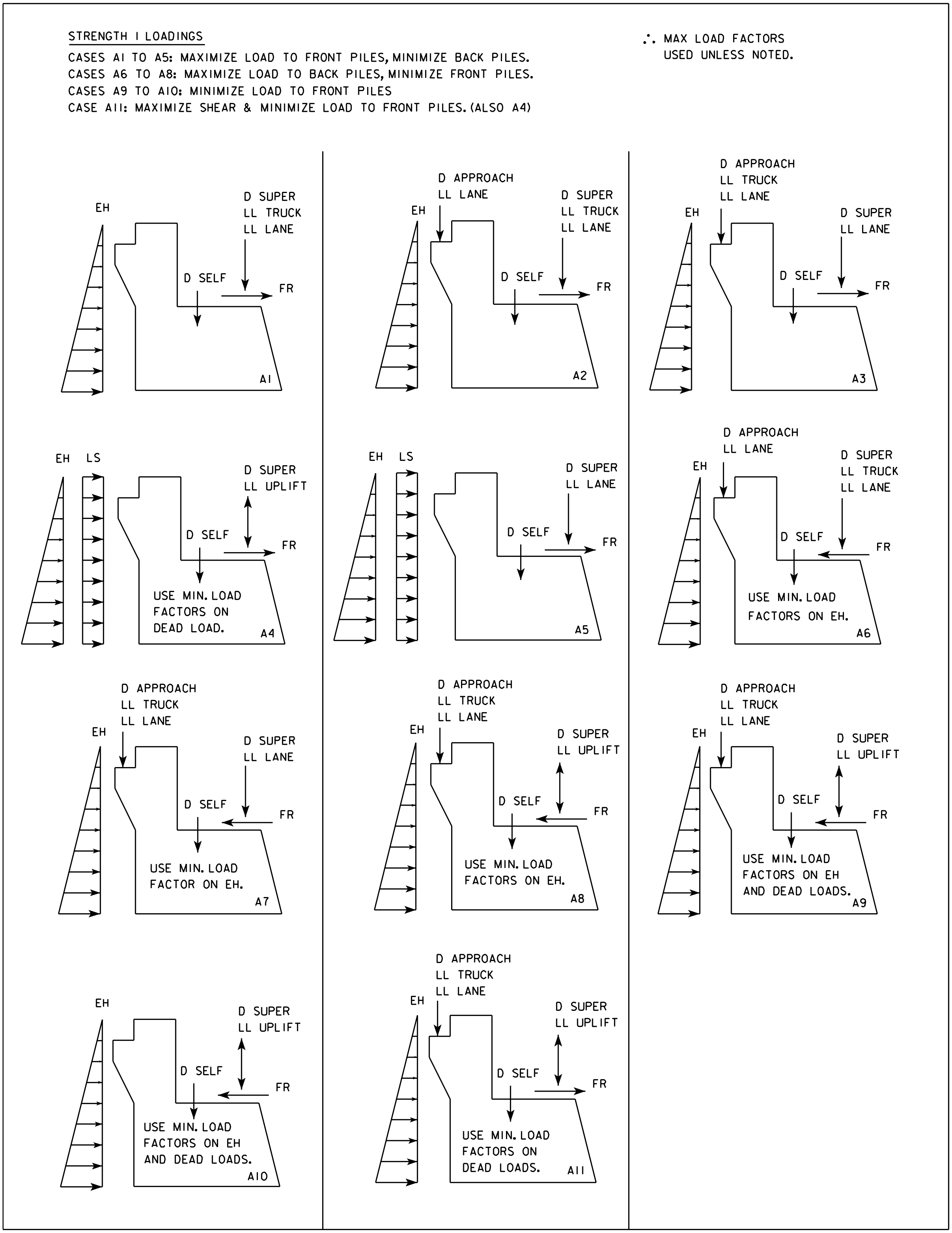
The following two figures for stub abutment load cases illustrate the typical cases that the designer should consider. The cases shown are not necessarily all the cases to be considered for a specific bridge, and the designer should be on the alert for load cases to add or remove based on the bridge under design.

SERVICE I LOADINGS

CASES B1 TO B5: MAXIMIZE LOADS TO FRONT PILES, MINIMIZE BACK PILES.
CASES B6 TO B8: MAXIMIZE LOADS TO BACK PILES, MINIMIZE FRONT PILES.
CASE B9: MINIMIZE LOADS TO FRONT PILES.
CASE B10: MAXIMIZE SHEAR & MINIMIZE LOADS TO FRONT PILES. (ALSO B4)

∴ LOAD FACTORS ARE 1.00




STRENGTH I LOADINGS
CASES A1 TO A5: MAXIMIZE LOAD TO FRONT PILES, MINIMIZE BACK PILES.
CASES A6 TO A8: MAXIMIZE LOAD TO BACK PILES, MINIMIZE FRONT PILES.
CASES A9 TO A10: MINIMIZE LOAD TO FRONT PILES
CASE A11: MAXIMIZE SHEAR & MINIMIZE LOAD TO FRONT PILES. (ALSO A4)
∴ MAX LOAD FACTORS USED UNLESS NOTED.
EH
D SUPER
LL TRUCK
LL LANE
D SELF
FR
A1
D APPROACH
LL LANE
EH
D SUPER
LL TRUCK
LL LANE
D SELF
FR
A2
D APPROACH
LL TRUCK
LL LANE
EH
D SUPER
LL LANE
D SELF
FR
A3
EH LS
D SUPER
LL UPLIFT
D SELF
FR
USE MIN. LOAD FACTORS ON DEAD LOAD.
A4
EH LS
D SUPER
LL LANE
D SELF
FR
A5
D APPROACH
LL LANE
EH
D SUPER
LL TRUCK
LL LANE
D SELF
FR
USE MIN. LOAD FACTORS ON EH.
A6
D APPROACH
LL TRUCK
LL LANE
EH
D SUPER
LL LANE
D SELF
FR
USE MIN. LOAD FACTOR ON EH.
A7
D APPROACH
LL TRUCK
LL LANE
EH
D SUPER
LL UPLIFT
D SELF
FR
USE MIN. LOAD FACTORS ON EH.
A8
D APPROACH
LL TRUCK
LL LANE
EH
D SUPER
LL UPLIFT
D SELF
FR
USE MIN. LOAD FACTORS ON EH AND DEAD LOADS.
A9
EH
D SUPER
LL UPLIFT
D SELF
FR
USE MIN. LOAD FACTORS ON EH AND DEAD LOADS.
A10
D APPROACH
LL TRUCK
LL LANE
EH
D SUPER
LL UPLIFT
D SELF
FR
USE MIN. LOAD FACTORS ON DEAD LOADS.
A11