

Integral Abutment Pile Selection

Integral abutment bridges eliminate the need for joints in bridge decks and thereby provide better protection for the superstructure and substructure from water and salt damage. Integral abutments are the preferred abutment type and the Department continues to strive to increase the number of structures eligible for integral design. Structures not eligible for integral abutments may still eliminate joints in the bridge with other options such as semi-integral abutments and alternate pile details.

The behavior and displacement capacity of integral abutment piles is a function of the soil-structure interaction that occurs with the soil embedded pile and the frame action that exists between the superstructure and abutment piles. The superstructure stiffness affects the rotational restraint, or fixity, of the pile head at the abutment and subsequently the moment developed in the pile as the superstructure expands or contracts, displacing the pile head laterally. Previously IDOT has implemented research resulting in expanded applicability of integral abutments with established prescriptive expansion length limits for the various available pile sizes. The prescriptive expansion length limits were derived from the displacement capacity of the piles for various anticipated soil conditions and superstructure stiffnesses anticipated to envelope most scenarios. This allows for a “no-analysis” policy intended to expedite integral abutment design by avoiding the need for designers to assess the capacity of piles for combined flexure and axial loads through frame analysis models which include soil-structure interaction.

The 2015 AASHTO LRFD interims introduced improvements increasing the structural capacity of concrete filled metal shell piles. These improvements have resulted in increased expansion length limits of metal shell piles for integral abutments. In addition, IDOT introduced a superstructure stiffness correction factor to better align pile behavior and superstructure stiffness and economize pile selections for superstructures that are smaller and more flexible. These improvements, including background information pertaining to IDOT’s integral abutment policy, are discussed in further detail in the following sections. Calculations for the correction factors presented herein have been programmed into an Excel spreadsheet titled “Integral Abutment Pile Selection” available on the IDOT website.

Placing piles directly beneath the superstructure beams or girders is considered the most efficient method of load transfer between the superstructure and abutment piles. As such, IDOT's "no analysis" policy requires the piles also be designed for axial load in a manner that results with an arrangement of one pile placed under each girder line. For integral abutments, it is permissible for the maximum pile spacing along the centerline of the pile cap to exceed 8 ft. Note that if the number of required piles exceeds the number of beams, an additional pile shall be placed between each beam so that the standard reinforcement schemes depicted on the base sheets are still valid.

Design Thermal Movement

The thermal range for Illinois, presented in AASHTO LRFD 3.12.2, is -20 °F to 120 °F. This range, coupled with the largest single gap joint opening permitted by AASHTO of 4 inches and the force effect due to uniform temperature, TU, of 1.2 yields a total contributing expansion length of 305'. Therefore integral structure lengths up to 610' are permitted.

Substructure components are typically detailed and built in construction for bridge geometry corresponding to a base or "installation" temperature of 50 °F. Expansion joints benefit from having the ability to adjust the opening of the joints to accommodate the ambient temperature at the time of installation as described in Article 520.04 of the IDOT Standard Specifications for Road and Bridge Construction. Conversely, it can be difficult, if not impossible, to make adjustments in construction for expansion or contraction of longitudinal superstructure elements of beam/slab type bridges that may occur prior to, or as the subject components are installed. This occurs due to the temperature of the longitudinal superstructure elements simply being different than the 50 °F base temperature assumed for establishing the layout of the substructure units.

Structures in Illinois tend to be built in the warmer months and it is anticipated that the average temperature is approximately 70 °F when superstructures and integral abutments become "locked together". Conversely, it is not unusual for portions of Illinois to experience short durations of sustained temperatures in the 0 to -5 °F range in the winter. As such and in lieu of the temperature range established by AASHTO for Illinois, the Bureau of Bridges and Structures has continued to use an 80 °F temperature range from "normal installation" for the study of

integral abutment piling for contraction, as well as expansion, realizing it is likely conservative for the latter scenario. It is worth noting that letting dates for projects can be easily moved, making it difficult to predict during the design phase the time of year and anticipated ambient air temperature likely to exist when a structure becomes integral.

Pile Orientation and Capacity

The HP pile orientation shall be with the web perpendicular to the centerline of roadway (i.e., weak axis bending). A single orientation was chosen for the HP's, regardless of skew, as the dominant direction of displacement is generally parallel to the longitudinal axis of the structure. Secondly, the dominant flexural demand is also generally about the weak axis with the weak axis flexural capacity being relatively unaffected by the axial load on the pile (when considering that the axial load will be less than or equal to the maximum geotechnical axial capacity of the pile). It is assumed that the superstructure is simply supported at the abutment although a certain amount of frame action exists between the superstructure and abutment piles. Assuming a simply supported condition at the abutment greatly simplifies the superstructure design effort and is consistent with the Department's load rating of bridge inventory with the AASHTO Bridge Rating software. Therefore, the weak axis of the piles shall be aligned with the primary bending axis of the superstructure in an effort to increase flexibility and simulate the assumed simply supported boundary condition as much as possible.

A fixed connection is detailed between the superstructure diaphragm and piles which requires the movement of the superstructure to be accommodated through flexure and combined bending and axial loads on the piles. AASHTO (2017) 6.15.1 indicates that "piles shall be designed as structural members capable of safely supporting all imposed loads" while AASHTO (2017) 6.15.3.2 indicates that piles subjected to axial load and flexure shall be designed according to equations in AASHTO LRFD 6.9.2.2.

The equations in AASHTO LRFD 6.9.2.2 are intended to estimate member capacity for limit states governed by excessive bending within the member (i.e., away from "bracing" points) accompanied by sideways deflection and/or twisting (i.e., lateral-torsional buckling). The AASHTO code implies that the soil surrounding fully embedded piles is sufficient to prevent Euler buckling and there are numerous research papers suggesting that soil embedment is sufficient to prevent lateral-torsional buckling. Given that the upper portion of IAB piles will generally be

installed in competent cohesive embankment material having a minimum Q_u of 1.0 tsf, this limit state is considered negligible for integral abutment piles.

A second limit state discussed in the “Guide to Stability Design Criteria for Metal Structures” by Theodore Galambos is the in-plane or local cross-sectional strength of the member. This limit state is considered to be more applicable for integral abutment piles given that the maximum bending moment in the pile typically occurs right at the abutment cap. Galambos provides the following equations for checking the ultimate cross sectional moment capacity of I-shaped members modified for the effect of axial compression:

$$\frac{P}{P_y} + 0.85 \left(\frac{M_o}{M_p} \right) \leq 1.0 \text{ (strong-axis bending)}$$

$$\left(\frac{P}{P_y} \right)^2 + 0.84 \left(\frac{M_o}{M_p} \right) \leq 1.0 \text{ (weak-axis bending)}$$

Where:

P = applied axial load

P_y = axial load at full yield

M_o = applied moment

M_p = plastic bending moment

$M_o \leq M_p$

The above local cross-sectional strength equations assume that slenderness and local buckling of the flanges is not a concern. A factored version of these equations exists in Appendix H of the 3rd Edition of the AISC code as shown below:

$$\left(\frac{M_{ux}}{\phi_b M_{px}} \right)^\zeta + \left(\frac{M_{uy}}{\phi_b M_{py}} \right)^\zeta \leq 1.0 \quad \phi_b = 0.9 \quad (\text{AISC Eqn. A-H3-1})$$

$$\zeta = 1.6 - \frac{\frac{P_u}{P_y}}{2 \left[\ln \left(\frac{P_u}{P_y} \right) \right]} \quad (\text{AISC Eqn. A-H3-3})$$

$$M'_{px} = 1.2 \times M_{px} \left[1 - \left(\frac{P_u}{P_y} \right) \right] \leq M_{px} \quad (\text{AISC Eqn. A-H3-5})$$

$$M'_{py} = 1.2 \times M_{py} \left[1 - \left(\frac{P_u}{P_y} \right)^2 \right] \leq M_{py} \quad (\text{AISC Eqn. A-H3-6})$$

The above equations are noted in AISC as being a considerable liberalization over those contained within the specification and mirrored in the AASHTO code. Acknowledging the statistical and probability basis of LRFD design, it is noted that there are different load and resistance factors between the AISC and AASHTO codes for similar loads and strength checks. One difference between the two codes is that the resistance factor for flexural resistance is 0.9 in AISC and 1.0 in AASHTO. Similarly, the resistance factor for axial compression is 0.85 in AISC and 0.7 for the axial resistance of HP's in the AASHTO code.

AASHTO LRFD 10.7.1.5 indicates that long-term durability of the pile (corrosion and deterioration) shall be taken into consideration and is discussed in further detail in AASHTO LRFD 10.7.5. It's been long suspected that gaps exist beneath the abutments due to normal consolidation and long term settlement of the embankments which allow air and water to come in contact with the piles. With the elimination of the concrete encasement, IDOT desires to maintain some corrosion protection of the piles or allowance for long term section loss due to corrosion. To address potential corrosion, a hybridized version of the AASHTO and AISC codes is used in assessing pile capacity by using resistance factors of 0.9 for flexure (AISC) and 0.7 for compression (AASHTO) to account for long term section loss. These resistance factors are also intended to account for additional eccentric loads that may be induced into the piles as a result of the structure being exposed to a larger temperature range due to the temperature at the time of construction, potential presence of long term shrinkage and driving tolerances for the piles. According to Article 512.12 of the IDOT Standard Specifications for Road and Bridge Construction, piles are permitted to be driven out of plan position by as much as 6 inches. It is anticipated that the above resistance factors are likely

conservative but were chosen for current use in lieu of performing statistical calibration and until future research is completed.

In the past, the flexural capacity of concrete-filled metal shell piles was computed using the ACI 318-05 code as it was much more liberal than the design provisions in the AASHTO code. The 2015 Interim Revisions to the AASHTO code introduced significant revisions for calculating the combined compression and flexural capacity of concrete-filled steel tubes considering composite action resulting in improved capacities. Combined flexural and axial capacity of metal shell piles for IAB's is now assessed using the interim revisions coupled with the use of an increased reinforcement cage (see metal shell piling base sheet) inside the metal shell pile and increased yield strength for the metal shell material (50 ksi). Since the reinforcement cage is explicitly relied upon for assessment of the structural capacity of the metal shell pile, a reduction in metal shell thickness of 0.06 in. is taken into account for potential corrosion as suggested by AASHTO LRFD 5.13.4.5.2.

Base Permissible Expansion Length

To assess displacement capacity and force demands on the abutment piles, 3-dimensional finite element analysis models were assembled with the following parameters:

- 63 in. plate girder with 1/2 in. webs, 1 in. x 14 in. flanges, and ≈ 136.75 ft spans
- 6 girders spaced at 6 ft. centers
- 36 ft. wide, 8 in. thick concrete deck
- 3'-8" thick pile cap and concrete diaphragm
- 3'-6" tall pile cap beneath the bottom of the superstructure beam
- Plates were used to model the deck, pile cap, concrete diaphragm, and wing walls.
- Beam elements were used to model the superstructure girders and piles. Inelastic beam elements were used for the pile segments just below the abutment cap. Rigid links were provided between the superstructure girders and deck to capture composite action.
- $\alpha_{\text{Steel}} = 6.5e^{-6} / ^\circ\text{F}$, $\alpha_{\text{Concrete}} = 5.5e^{-6} / ^\circ\text{F}$, $\Delta_{\text{Temperature}} = +/- 80 ^\circ\text{F}$
- 1 ft. thick "dog-ear" style wing walls. The lengths were sized assuming soil is allowed to wrap around to the front side with a maximum length of 10 ft.
- Roller supports at the piers.

- Abutment piles were placed beneath each girder and were modeled to extend 2 ft into the pile cap.
- Steel superstructures were modeled for 0, 15, 30, and 45 degree skews.
- P-y soil springs were modeled along the length of the pile assuming soil with a Q_u of 1.5 tsf.
- P-y soil springs for the abutment backfill were modeled along the back of abutment assuming an internal friction angle of 35 degrees and placed at an angle of 15 degrees from the axis perpendicular to the abutments for skews exceeding 15 degrees to account for wall friction.

Figure 1 shows the results of the analysis models and permissible effective expansion lengths that correspond with the previously discussed methods for computing the combined axial load and bending pile capacities. As the intent of the analysis models is to assess superstructure stiffness effects on the various piles, the pile capacities are assessed and permissible effective expansion lengths are computed assuming each pile is loaded to its maximum factored geotechnical axial capacity and not necessarily the vertical reactions that correspond with the superstructure parameters.

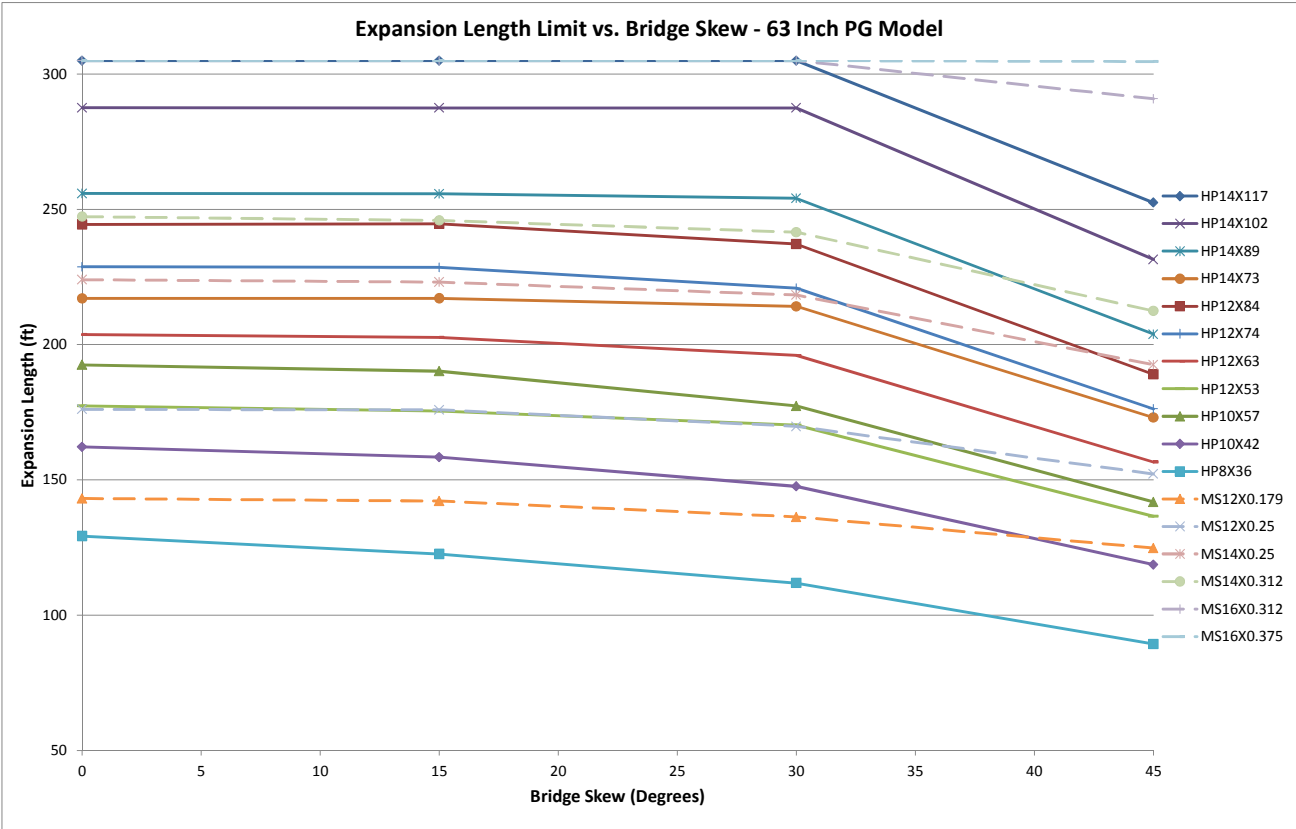


Figure 1. 63-Inch Plate Girder Results

The piles in Figure 1 with expansion lengths truncated at 305 ft. do not necessarily indicate the expansion length that corresponds to the pile capacity but rather the limits of the analysis governed by the limitations of the strip seal expansion joint at the ends of the bridge approach slabs.

Superstructure Stiffness Expansion Length Correction Factor

Superstructure stiffness is viewed as one of the largest factors that affect permissible expansion lengths for a given pile. The superstructure properties used to generate the results in Figure 1 were chosen from an example structure anticipated to result in a superstructure stiffness and permissible expansion lengths that are likely conservative for the majority of “garden variety” structures. To investigate the effects of varying superstructure stiffnesses, a limited number of piles have been analyzed using the same finite element model previously described for the 63-inch plate girder with the following alternate superstructure modifications:

- W30x124 Beam, 68.4 ft. Spans
- 72" PPC Bulb-T, 110 ft. Spans
- 36" PPC I-Beam, 59.5 ft. Spans

Analysis results from the alternate superstructure properties have been analyzed against those in Figure 1 that were generated using the base superstructure properties corresponding to the 63-inch plate girder. Following is a procedure for adjusting the permissible expansion lengths in Figure 1 for alternate superstructure properties in an effort to better economize and align pile options for the superstructure stiffness of any bridge under consideration.

Figure 2 provides a qualitative depiction of the movement that occurs at an integral abutment due to thermal contraction. This movement can be summarized with the following equation:

$$\alpha_{\text{eff}} L_{\text{exp}} \Delta_T - \frac{V_p L_{\text{exp}}}{AE_{\text{eff}}} = \Delta_p + \Delta_{\theta}$$

Equivalently, thermal contraction of the superstructure minus elastic lengthening of the superstructure due to the abutment resistance equals the lateral pile displacement (Δ_p) plus the lateral movement that occurs due to rotation of the pile and superstructure (Δ_{θ}). The above equation can be rearranged as follows to solve for the expansion length.

$$L_{\text{exp}} = \frac{\Delta_p + \Delta_{\theta}}{\alpha_{\text{eff}} \Delta_T - \frac{V_p}{AE_{\text{eff}}}}$$

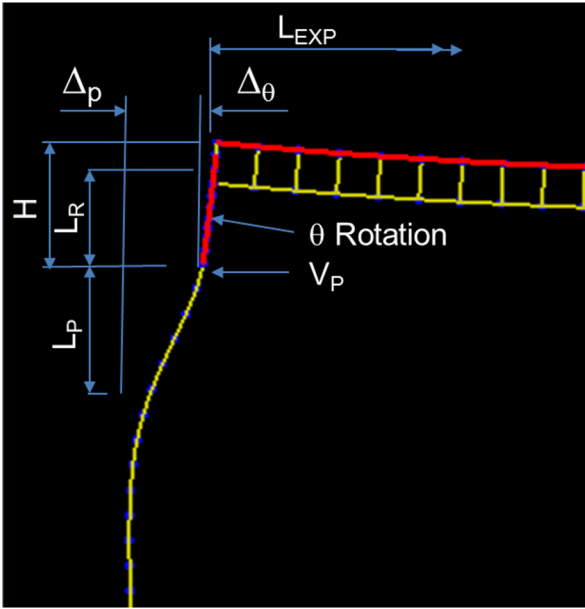


Figure 2. Illustration of Integral Abutment Movement

Where:

- L_{exp} = permissible expansion length for a given pile
- Δ_p = lateral displacement of the pile that corresponds with the maximum moment capacity of the pile
- Δ_θ = lateral displacement over the height from the bottom of the abutment cap to the mid-thickness of the deck that occurs due to rotation of the superstructure and pile
= $H \cdot \theta$
- H = height from the bottom of the abutment cap to the mid-thickness of the deck (in.)
- θ = rotation of the superstructure and pile
- α_{eff} = effective thermal coefficient for the superstructure
- Δ_T = temperature range over which thermal contraction is presumed to occur (taken as 80°F for the current study)
- V_p = shear force at the top of the pile that corresponds with the lateral stiffness and maximum moment capacity of the pile
- AE_{eff} = effective cross-sectional stiffness of the superstructure

The rotational stiffness of the superstructure at the abutment and the pile have a significant effect on Δ_p and Δ_θ . This rotational stiffness may be estimated using the following relationships:

$$\begin{aligned}
 k_\theta &= \text{total rotational stiffness at the abutment (k*ft/rad.)} \\
 &= k_{\theta-p} + k_{\theta-s} \\
 k_{\theta-p} &= \text{rotational stiffness of the pile (k*ft/rad.)} \\
 &= \frac{EI_p}{144 L_p} \\
 EI_p &= \text{flexural stiffness of the pile (k*in.²) (Note that for HP sections, the weak axis moment of inertia shall be used.)} \\
 L_p &= \text{approximate fixity depth of the pile for soil with } Q_u \text{ equal to 1.5 tsf (ft)} \\
 &= 2.2 \ln(EI_p) - 24 \\
 k_{\theta-s} &= \text{rotational stiffness of the superstructure (k*ft/rad.)} \\
 &= \frac{2 E_n I_{ne} s_p}{L_e s_s} \text{ (for simple spans)} \\
 &= \frac{E_n \left(\frac{3 I_{ne}}{L_e} + \frac{2 I_{na}}{L_a} \right) s_p}{72 \left(2 + \frac{I_{na} L_e}{I_{ne} L_a} \right) s_s} \text{ (for continuous spans)} \\
 s_s &= \text{superstructure beam spacing perpendicular to centerline of structure (ft)} \\
 s_p &= \text{pile spacing perpendicular to centerline of structure (ft)} \\
 I_{ne}, I_{na} &= \text{short term composite moment of inertia for the end span (} I_{ne} \text{) and adjacent interior span (} I_{na} \text{) superstructure beam using the width of the deck tributary to the beam (in.⁴)} \\
 L_e, L_a &= \text{length of the end span (} L_e \text{) and adjacent interior span (} L_a \text{) (ft) (Note: } L_a \text{ shall be set to a small value, such as 0.01 ft, for 2-span structures)} \\
 E_n &= \text{modulus of elasticity used to calculate } I_{ne} \text{ and } I_{na} \text{ (ksi)}
 \end{aligned}$$

The above formula for calculating the rotational stiffness of the pile models the pile as a cantilever with a concentrated moment applied to the free end. The general form of this equation (EI/L) can be found in most structural analysis text books. The expression for the fixity depth of the pile, L_p , acknowledges that that this hypothetical parameter varies according to pile stiffness and was derived from the results of analysis models for the 63-in. plate girder according to the depth at which there is an inflection point in the bending moment for the pile. The fixity depths are

anticipated to nominally fluctuate for a given pile as superstructure stiffness changes. However, the proposed estimated depths are considered suitable for the purposes of scaling the effects of superstructure stiffness on permissible expansion lengths.

The formulas for calculating the rotational stiffness of the superstructure assume a simply supported structure with a concentrated moment applied at the abutment and adjusts the stiffness for the ratio of the pile to superstructure beam spacing. The equation provided for the simple span condition can also be found in most structural analysis text books. The equation for the continuous span condition was derived using the "slope deflection" method of analysis for a simply supported continuous beam with the end of the adjacent span restrained for flexure but free to deflect vertically as shown in Figure 3.

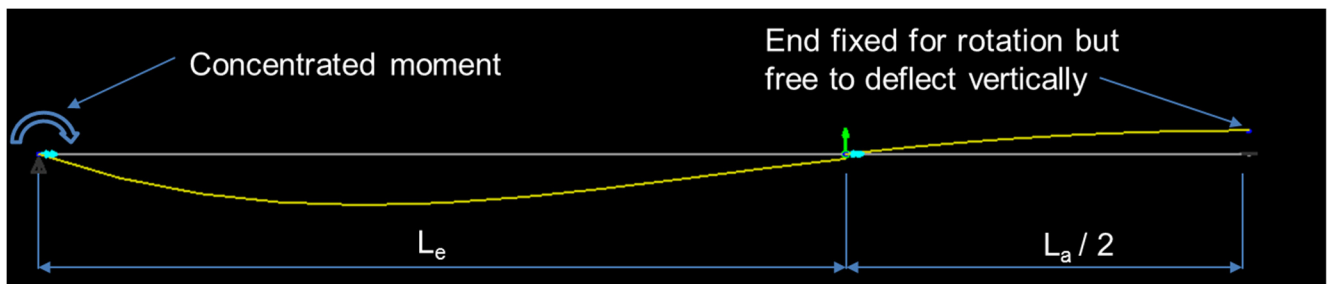


Figure 3. Continuous span model used for estimating superstructure rotational stiffness

Span configurations will affect the location of the inflection point in the adjacent span (i.e., it will not always occur at $L_a/2$ as shown in Figure 3). Estimated stiffnesses using the derived formula have been checked against values obtained from software analysis for several varying span configurations. The estimated rotational stiffness are generally within a small percentage of the values obtained from software analysis indicating that the estimated values are reasonably accurate.

In addition for steel structures, beam sections often change within the negative moment region. To investigate this impact, additional analysis was conducted with the moment of inertia of the negative moment region sections modeled as either 0.5 or 2.0 times the value in the positive moment region. The estimated rotational stiffness values were calculated using the stiffness in the positive moment regions. The comparative analysis indicated that when the moment of inertia of the negative

moment region is less than the positive moment region, there is improved agreement between the values obtained from software analysis and the estimated values. This is considered favorable as the short term composite moment of inertia in the positive moment region is generally anticipated to be greater than the negative moment of inertia considering either the noncomposite steel section or the cracked composite moment of inertia. Therefore, it is deemed acceptable to use section properties in the positive moment regions of the end and adjacent spans for estimating the superstructure rotational stiffness.

The effective cross-sectional stiffness of the superstructure (AE_{eff}) is used to account for elastic lengthening of the structure and may be calculated as follows:

$AE_{\text{eff}} = AE_{\text{eff-s}}$ adjusted for the ratio of the pile to superstructure beam spacing

$$= \frac{AE_{\text{eff-s}} s_p}{s_s}$$

$AE_{\text{eff-s}} = AE$ of the composite superstructure beam adjacent to the abutment (single and 2-span continuous structures) ($k \cdot \text{in.}^2$)

$$= \frac{(L_e + \lambda L_a) AE_e AE_a}{AE_a L_e + AE_e \lambda L_a} \text{ (continuous structures with more than 2 spans) } (k \cdot \text{in.}^2)$$

$AE_e = AE$ of the composite superstructure beam in the end span ($k \cdot \text{in.}^2$)

$AE_a = AE$ of the composite superstructure beam in the adjacent interior span ($k \cdot \text{in.}^2$)

$\lambda =$ span length factor

$= 0.5$ for 3-span structures

$= 1.0$ for structures with more than 3 spans

The above equation for continuous structures was derived using the axial load deformation relationship for members with variable cross-sectional areas that can be readily found in most mechanics of materials textbooks. Similar to the discussion for the rotational stiffness of the superstructure, it is acknowledged that steel structures often utilize larger sections in the negative moment region. The potential impact of larger negative moment sections was assessed with a series of previously designed structures having larger beam sections in the negative moment regions. There was generally less than a 5% difference in $AE_{\text{eff-s}}$ values between assuming the positive moment region properties over the entire span length and including the properties of the

larger beams in the negative moment region. This small difference is due to the inverse relationship involved in calculating axial stiffness of members with variable cross sections and connected in series (i.e., end to end). This difference is deemed negligible considering potential effects of deck cracking, lateral stiffness of intermediate piers, resistance of expansion bearings, etc. As such, and similar to the rotational stiffness of the superstructure, it is recommended that $AE_{\text{eff-s}}$ only be calculated using the superstructure properties in the positive moment regions.

Effective coefficient of thermal expansion (α_{eff}) is an intermediate coefficient of dissimilar materials working together (i.e., steel and concrete) and is calculated according to the cross-sectional stiffness of the individual elements. α_{eff} may be calculated as indicated below:

$$\alpha_{\text{eff}} = \frac{\alpha_{\text{Concrete}} AE_{\text{Concrete}} + \alpha_{\text{Steel}} AE_{\text{Steel}}}{AE_{\text{eff-s}}}$$

$\alpha_{\text{Concrete}} =$ coefficient of thermal expansion for concrete ($5.5e^{-6} / ^\circ\text{F}$)
 $\alpha_{\text{Steel}} =$ coefficient of thermal expansion for steel ($6.5e^{-6} / ^\circ\text{F}$)
 $AE_{\text{Concrete}} =$ AE of the concrete slab tributary to the superstructure beam ($\text{k}\cdot\text{in.}^2$)
 $AE_{\text{Steel}} =$ $AE_{\text{eff-s}} - AE_{\text{Concrete}}$ ($\text{k}\cdot\text{in.}^2$) (accounts for variable steel cross sections that may exist in the end and adjacent spans for continuous structures)

The lateral pile displacement, Δ_p , is difficult to predict with simple equations due to the non-linear resistance of the soil as well as the effects of the superstructure. However, the following formula has been derived in an effort to qualitatively predict the effects that the superstructure has on the relationship between pile moment and lateral pile displacement. The following formula models the pile as a “fixed-fixed” member of length L_p with a reduction in flexure at the top of the pile that is a function of the total rotational stiffness at the abutment and assumes that member “ L_R ” shown in Figure 2 is a rigid link.

$$\Delta_p = \frac{M_p \left(1 + \frac{4 EI_p}{144 k_\theta L_p} \right)}{k_p \left(\frac{L_p}{2} - \frac{4 L_R EI_p}{144 k_\theta L_p} \right)}$$

$M_p =$ moment capacity of pile ($\text{k}\cdot\text{ft.}$)

$k_p =$ lateral stiffness of the pile for a “fixed-fixed” condition ($\text{k}/\text{in.}$)

$$= \frac{12 E I_p}{(12 L_P)^3}$$

L_R = vertical distance from the bottom of the abutment cap to the centroid of the composite superstructure at the abutment (ft)

In addition, the estimated lateral movement that occurs due to rotation of the pile and superstructure, Δ_θ , can be further refined as a function of the pile moment and total rotational stiffness at the abutment.

$$\Delta_\theta = \frac{M_p H}{k_\theta}$$

Using the assorted variables described herein, regression analysis was performed in Excel to develop the following relationship to adjust permissible expansion lengths for a given pile for various superstructure properties:

ELCF = expansion length correction factor
 = $0.9077 \times 0.9967^{R_{ps}} \times 4.345^{R_p} \times 0.9874^{R_\theta} \times 0.2674^{R_a} \times 0.9752^{R_{ea}}$

R_{ps} = pile stiffness factor

$$= \frac{E I_p}{1168700}$$

R_p = Δ_p ratio

$$= \frac{\left[\left(1 + \frac{4 E I_p}{144 k_\theta L_P} \right) \right]}{\left[\left(\frac{L_P}{2} - \frac{4 L_R E I_p}{144 k_\theta L_P} \right) \right]_{alt}} \bigg/ \frac{\left[\left(1 + \frac{4 E I_p}{144 k_\theta L_P} \right) \right]}{\left[\left(\frac{L_P}{2} - \frac{4 L_R E I_p}{144 k_\theta L_P} \right) \right]_{base}}$$

R_θ = Δ_θ ratio

$$= \frac{\left[\frac{H}{k_\theta} \right]_{alt}}{\left[\frac{H}{k_\theta} \right]_{base}}$$

$$R_a = \alpha_{\text{eff}} \text{ ratio}$$

$$= \frac{[\alpha_{\text{eff}}]_{\text{alt}}}{[\alpha_{\text{eff}}]_{\text{base}}}$$

$$R_{\text{ea}} = AE_{\text{eff}} \text{ ratio}$$

$$= \frac{[AE_{\text{eff}}]_{\text{alt}}}{[AE_{\text{eff}}]_{\text{base}}}$$

base = properties related to the 63-in. plate girder model

alt = properties related to an alternate superstructure configuration

For the R_p and R_θ ratios, M_p and k_p are considered constant for a given pile and cancel out of the equations for Δ_p and Δ_θ .

The width of some analysis models were also increased to investigate potential effects of varying bridge widths. The impact to the biaxial bending demands on the piles was generally small and deemed not significant enough to develop additional policy at this time considering all other variables involved.

Soil Modification Factors

Abutments are often constructed on top of manmade embankments which are typically required by IDOT policy to consist of compacted cohesive soils having a minimum Q_u of 1.0 tsf. As such, IDOT chose to assume for the aforementioned analysis models that the upper portion of the piles subjected to significant bending and lateral displacement would be installed in material having a Q_u of 1.5 tsf. Assuming a Q_u of 1.5 tsf was anticipated to envelope a significant amount of soil properties typically encountered within the embankment at a nominal depth below the pile cap and should generate results that are conservative for weaker soils. Through time it has become apparent that a modest number of structures exist in which the soil strengths at shallow depths are comprised of soils having a Q_u greater than 1.5 tsf and/or contain granular soil layers. Rather than simply discount these structures from being eligible for integral abutments, additional correction factors have been developed.

“Pushover” analysis models have been used to assess the impact of soils strengths other than 1.5 tsf, and up to a maximum of 3.0 tsf, on various pile sizes. Increased soil strength results in

increased pile stiffness and a decrease in lateral displacement of the pile corresponding to the pile flexural capacity, “ M_p ”. Analysis suggests that there is approximately a 15% decrease in the displacement capacity of the piles for each 0.5 tsf increase in Q_u . As such, the permissible expansion lengths shown in Figure 1 can be reduced by the following modification factor to adjust for the effect of soils with a Q_u greater than 1.5 tsf:

$$M_{\text{pile}} = 1.45 - 0.3 \times Q_u$$

Analysis indicates that the above equation produces conservative results for soils with a Q_u less than 1.5 tsf.

The above equation only addresses the effect of the stiffer soil on the pile itself. However, as soil stiffness increases, a larger lateral force is required to achieve a pile displacement that corresponds to the pile’s moment capacity. IDOT’s standard integral abutment reinforcement is based on a design moment at the base of the superstructure that is a function of the pile moment plus flexure caused by the lateral pile force acting over the height of the cap for displacement demands corresponding to soil with a Q_u of 1.5 tsf. As such, the following expression and reduction factor was developed for the permissible expansion lengths shown in Figure 1 to ensure that the pile demands from the stiffer soil conditions do not exceed the assumptions used in standardizing the abutment reinforcement. The following equation is more restrictive than the equation shown above for the piles. This equation does not apply for soils with a Q_u less than 1.5 tsf.

$$M_{\text{abut}} = 1.5 / Q_u$$

For soils with a Q_u other than 1.5 tsf, the formula shown for M_{pile} can also be used to provide a reasonable estimate of the lateral stiffness of a given pile relative to its lateral stiffness for soils with a Q_u equal to 1.5 tsf. To obtain the relative lateral stiffness, a pile stiffness modifier (M), the reciprocal of the equation shown for M_{pile} should be used.

$$M = 1 / (1.45 - 0.3Q_u)$$

It is recommended that a weighted average of the soil strengths within a depth of 10 ft. (considered the “critical pile depth”) below the abutment cap be used when assessing the

previously mentioned modification factors. Below a depth of 10 ft., pushover analysis models suggest increased soil stiffness has minimal effect on the force demands on the pile for the magnitude of displacements considered when the average Q_u within the critical pile depth is greater than or equal to 1.5 tsf. Conversely, when the average Q_u within the critical pile depth is less than 1.5 tsf, pushover analysis models suggest increased soil stiffness below 10 ft. may be influential on the pile response. However, the generally conservative results for the above “ M_{pile} ” equation for soils with a Q_u less than 1.5 tsf should envelope these effects in such scenarios.

While it is anticipated that the upper portion of integral abutment piles will generally be installed in embankment material consisting of cohesive soils, designers may occasionally encounter soil profiles with a combination of cohesive and granular soils within the critical pile depth. The following expression should be used for converting granular soil layers to equivalent cohesive soils for the purpose of evaluating soils within the critical pile depth.

$$Q_u = 0.75 \ln(N) + 0.7$$

N is the SPT blow count recorded in the soil boring logs. This expression was derived by conducting a series of lateral load pile analysis for combinations of granular and cohesive soils. The above equation is intended only for the purpose of trying to equate the lateral stiffness of shallow granular soil layers and is not intended to be used for assessing the strength of granular soil layers.

Average soil strengths within the critical pile depth of 3.0 tsf have generally been considered by the Department as an upper limit for using integral abutments. Beyond 3.0 tsf, piles are anticipated to encounter significant resistance to lateral deflection from thermal superstructure movement. There are however some instances in which it may be acceptable to use integral abutments with soils having a Q_u exceeding 3.0 tsf. Such scenarios will generally include significantly different soil strengths at each abutment. As an example, if the average soil strengths at the abutments were 0.8 and 4.0 tsf, the abutment with 4.0 tsf soil is anticipated to be fairly rigid and exhibit little lateral movement with most of the expansion superstructure movement occurring at the abutment with the weaker soil. When the average soil strengths at an abutment exceed 3.0 tsf and the expansion length of structure tributary to the subject abutment is less than 20% of the overall structure length, integral abutments may still be

considered. The 20% is based upon engineering judgement acknowledging the variability that may exist when calculating the expansion length tributary to an abutment using relative stiffness and the above pile stiffness modifier (M) equation. Note that additionally the designer shall investigate pile driving stresses for piles driven in material with a Q_u greater than 3.0 tsf to assure that these are acceptable and the tributary expansion length at the abutment with the weaker soil shall still be limited to 305'. Any pile in material with a Q_u less than 1.0 tsf shall be investigated for combined bending and axial loads according to AASHTO LRFD 6.9.2.2 (HP's) and 6.9.6.3 (metal shell piles) considering the pile to be unbraced.

When the average soil strengths at an abutment exceed 3.0 tsf and do not satisfy the above 20% criteria, the Department offers two additional options to achieve a jointless structure. The first option is the semi-integral abutment which replaces the connection details between the abutment cap and the diaphragm with an elastomeric bearing to isolate the pile from the bending moments. Semi-integral structures are limited to the same structure lengths as integral structures but the skews may exceed 45 degrees if necessary; however, wider cap widths may be required. Details are provided in the Abutment Section. The second option would involve precoring the piles at the abutments in the 10-foot critical pile depth and backfilling with bentonite having a Q_u value of 1.0 tsf to increase pile flexibility. The bentonite typically comes in a pellet form and would be placed after the piles are driven. The contractor should add water according to the manufacturer recommendations to achieve the Q_u of 1.0 tsf. Note that the old practice of backfilling pre-cored holes with loose sand shall not be permitted because cyclic pile movement progressively consolidates the sand and reduces pile flexibility.

MSE abutment walls are permitted on integral structures when determined to be the most feasible option provided the following requirements are met. The pile movement shall be isolated from the reinforced soil embankment for the entire depth of the select backfill by placing a pile encasement over each pile. The pile encasement may be either a corrugated metal pipe or a HDPE pipe and shall be sized to provide a minimum space between the pile and the encasement of 3 inches or as required by design. The void between the encasement and the pile shall be filled with bentonite as previously described. The preferred MSE integral abutment wall orientation is straight and parallel to the roadway below. "Wrap around" MSE integral abutment walls are permitted up to a 90 degree "U" shaped configuration when necessary. Acute angles are not permitted. The reinforced MSE soil may only extend to the bottom of the

abutment cap. Parallel “Dogear” wingwalls from the abutment cap shall be used to control the remainder of the soil slope.

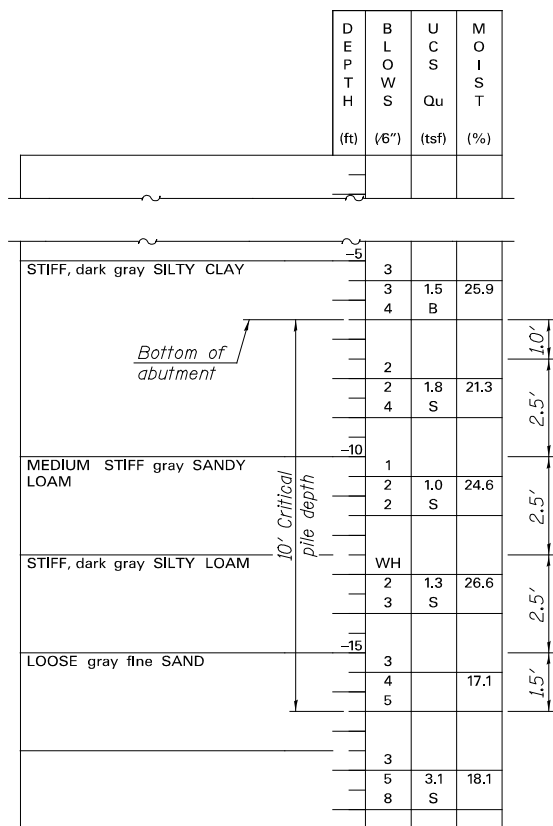
End Span Length Restrictions

Live load that causes downward deflection in the end span typically increases flexural demand on the abutment piles for the thermal expansion scenario while decreasing the flexural demand for the contraction scenario. For the thermal loading condition, superstructure contraction generally controls the flexural demand on the piles. As such, analysis used to generate the results in Figure 1 assumed contraction controlled with live load placement to create the maximum vertical live load reaction at the abutment.

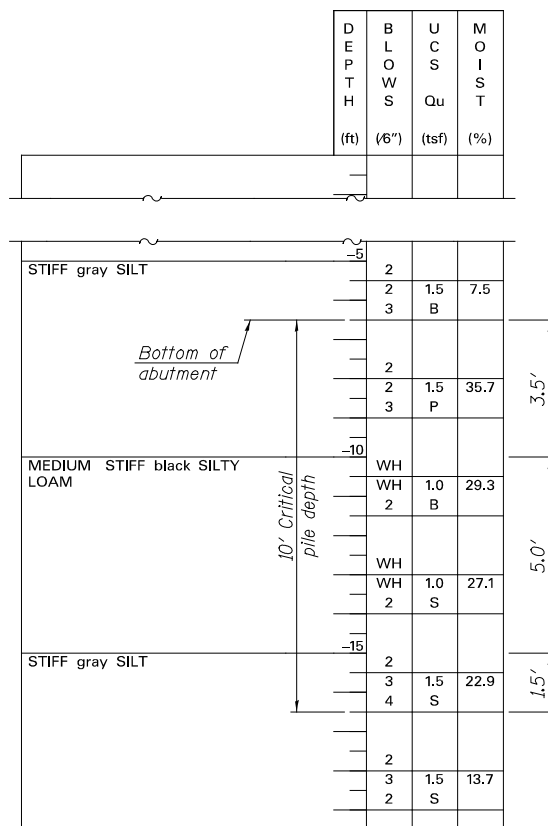
Select structures with longer end spans have been analyzed and scenarios identified where live load rotations in the end span suggest larger piles should be used at the abutments than would be otherwise specified for typical structures. As such, use of the pile selection procedure detailed herein is limited to simple span structures having a maximum length of 170 ft and continuous span structures with a maximum end span length of 200 ft. In addition, abutments adjacent to spans of 150 ft or greater shall use 14 or 16-inch metal shell piles or HP 12 x74 piles and larger.

Pile Selection Example 1

The structure is a continuous 450 ft. long structure consisting of 6 – 75 ft. spans with a zero-degree skew. The superstructure consists of 5 - W36x150 beams at a 7 ft. spacing with an 8 inch thick deck. The structure is the same width throughout and thus expected to have the same number of piles at each abutment. The following example determines the effective expansion length for the structure and indicates acceptable piles.



West Abutment Boring B-1



East Abutment Boring B-2

Determine the average Q_u for the critical pile depth at each abutment.

$$Q_{u-west} = \frac{(1.0)(1.5) + (2.5)(1.8) + (2.5)(1.0) + (2.5)(1.3) + (1.5)[0.75\ln(9) + 0.7]}{10}$$

$$= 1.53 \text{ (say 1.5 tsf)}$$

$$Q_{u-east} = \frac{(3.5)(1.5) + (5.0)(1.0) + (1.5)(1.5)}{10} = 1.25 \text{ tsf}$$

Determine the pile stiffness modifier for the east abutment since it has an average Q_u that is not equal to 1.5 tsf.

$$M_{east} = \frac{1}{1.45 - 0.3(1.25)} = 0.93$$

Assume 6 beam lines in the structure with a pile placed beneath each beam and calculate the centroid of stiffness from the west abutment.

$$\Sigma_{\text{Stiff. W. Abut.}} = \frac{(6 \text{ piles})(0 \text{ ft}) + (6 \text{ piles})(0.93)(450 \text{ ft})}{(6 \text{ piles}) + (6 \text{ piles})(0.93)} \approx 217 \text{ ft}$$

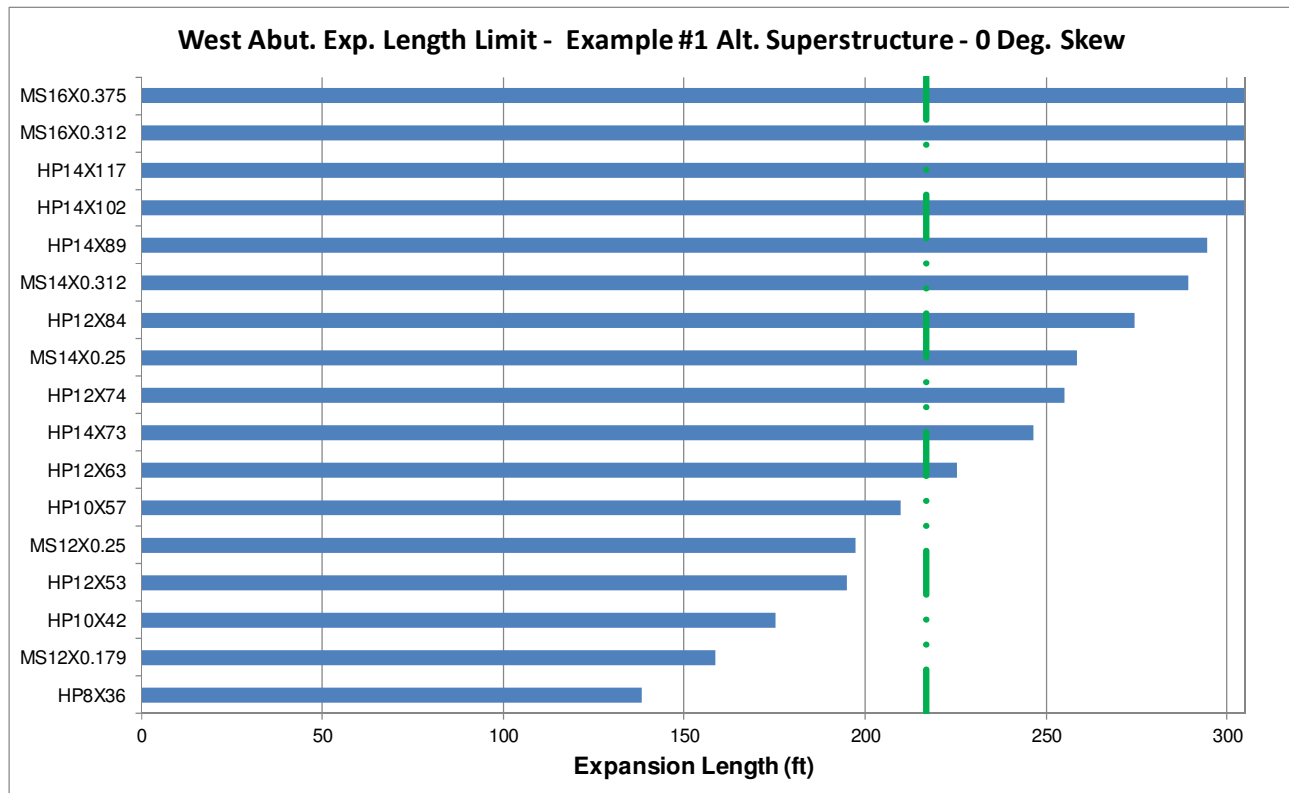
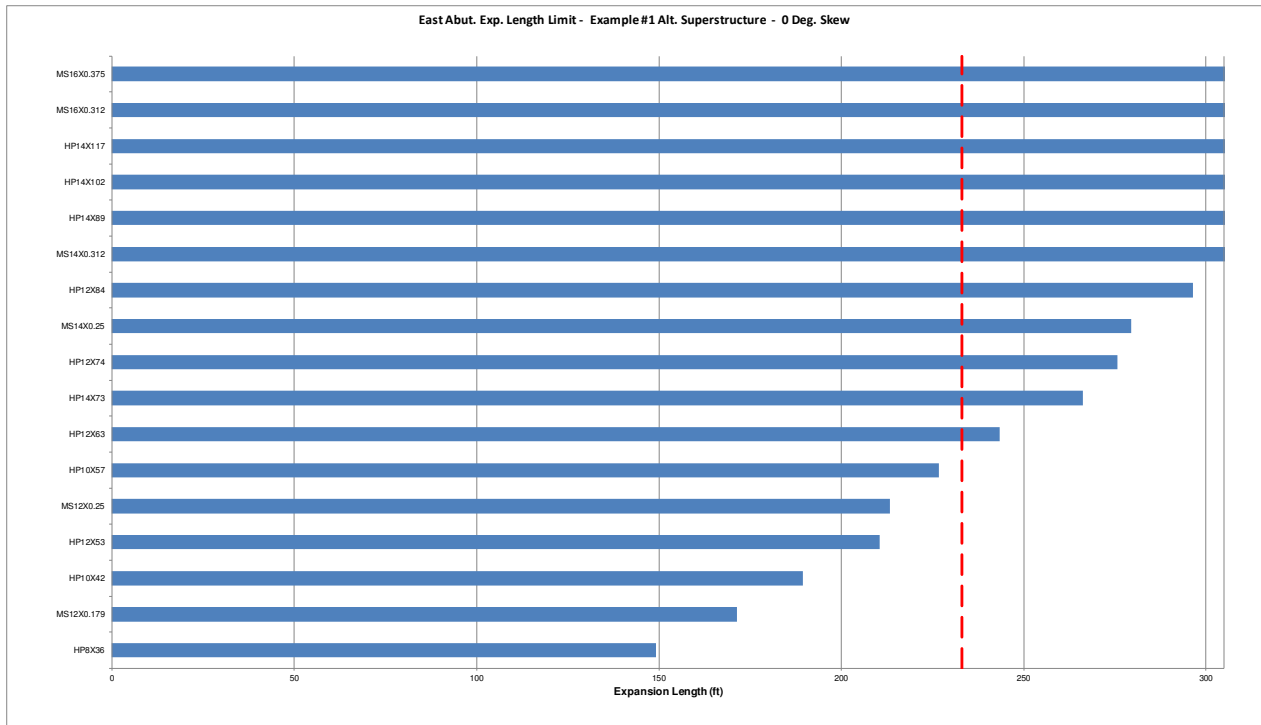
The distance from the centroid of stiffness to the East Abutment is

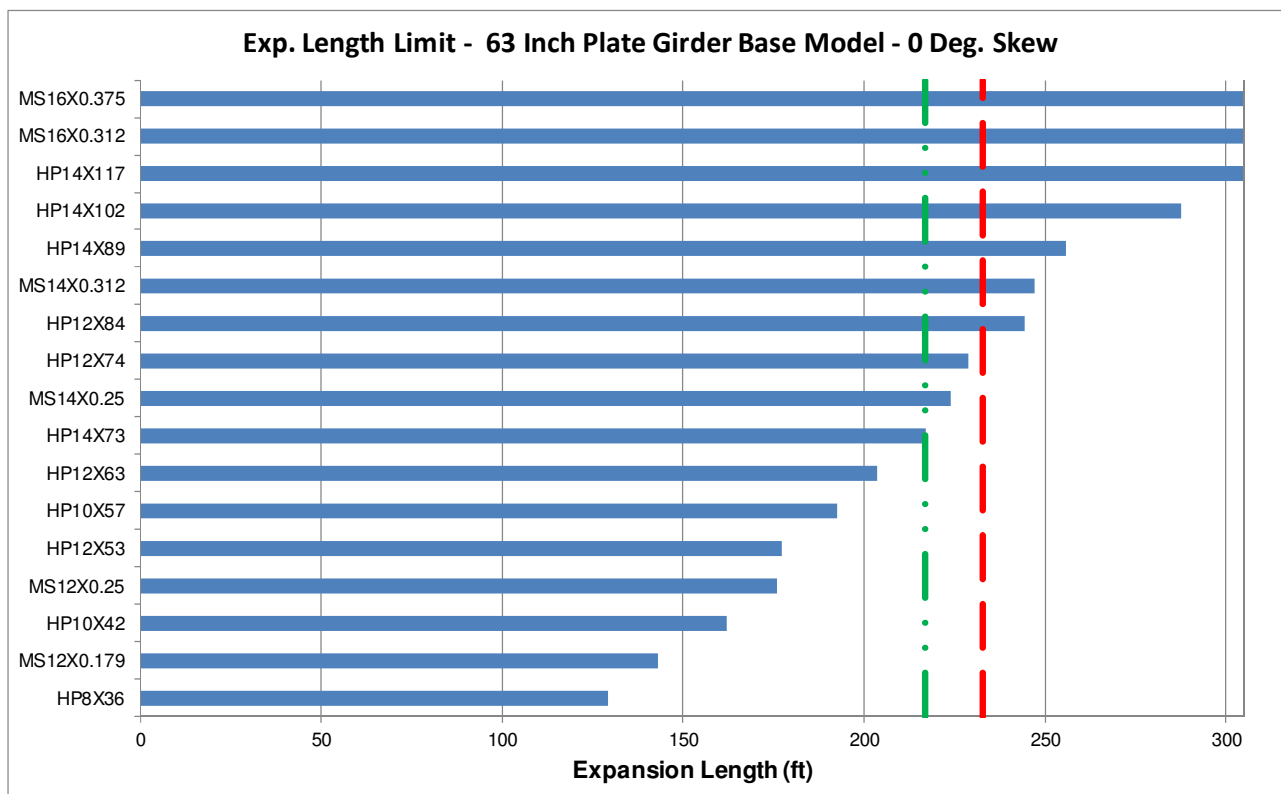
$$450 - 217 = 233 \text{ ft.}$$

The soil strength correction factor at the east abutment for the displacement capacity and permissible expansion length of the pile is the reciprocal of “M_{east}” calculated above, or 1.08.

The table below shows the base model expansion length factors for each pile type as well as the various correction factors. The superstructure stiffness correction factors have been calculated for each pile using the previously described procedure and the alternate superstructure properties for the example. Also shown are pile selection graphs for each abutment with the correction factors incorporated. Piles whose lengths exceed the tributary expansion length are suitable for use. For comparison, the tributary expansion lengths are also plotted on a graph of permissible pile expansion lengths for the base case model that assumes a Q_u of 1.5 tsf.

	BASE MODEL EXP. LENGTH (FT)	SUPERSTRUCTURE STIFFNESS CORRECTION FACTOR	EAST ABUTMENT		WEST ABUTMENT	
			SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT
HP14X117	305	1.18	1.08	1.27	1	1.18
HP14X102	288	1.16	1.08	1.26	1	1.16
HP14X89	256	1.15	1.08	1.24	1	1.15
HP14X73	217	1.14	1.08	1.23	1	1.14
HP12X84	244	1.12	1.08	1.21	1	1.12
HP12X74	229	1.12	1.08	1.20	1	1.12
HP12X63	204	1.11	1.08	1.19	1	1.11
HP12X53	177	1.10	1.08	1.19	1	1.10
HP10X57	193	1.09	1.08	1.18	1	1.09
HP10X42	162	1.08	1.08	1.17	1	1.08
HP8X36	129	1.07	1.08	1.16	1	1.07
MS12X0.25	176	1.12	1.08	1.21	1	1.12
MS14X0.25	224	1.16	1.08	1.25	1	1.16
MS14X0.312	247	1.17	1.08	1.26	1	1.17
MS16X0.312	305	1.22	1.08	1.31	1	1.22
MS16X0.375	305	1.24	1.08	1.34	1	1.24





Pile Selection Example 2

Use the same geometric configuration from Example 1 except that the average Q_u within the critical pile depth at the east abutment is increased from 1.25 to 2.5 tsf. The following example determines the effective expansion length for the structure and indicates acceptable piles.

Determine the pile stiffness modifier for the east abutment since it has an average Q_u that is greater than 1.5 tsf.

$$M_{\text{east}} = \frac{1}{1.45 - 0.3(2.5)} = 1.43$$

Assume 6 beam lines in the structure with a pile placed beneath each beam and calculate the centroid of stiffness from the west abutment.

$$\Sigma_{\text{Stiff. W. Abut.}} = \frac{(6 \text{ piles})(0 \text{ ft}) + (6 \text{ piles})(1.43)(450 \text{ ft})}{(6 \text{ piles}) + (6 \text{ piles})(1.43)} \approx 265 \text{ ft}$$

The distance from the centroid of stiffness to the East Abutment is

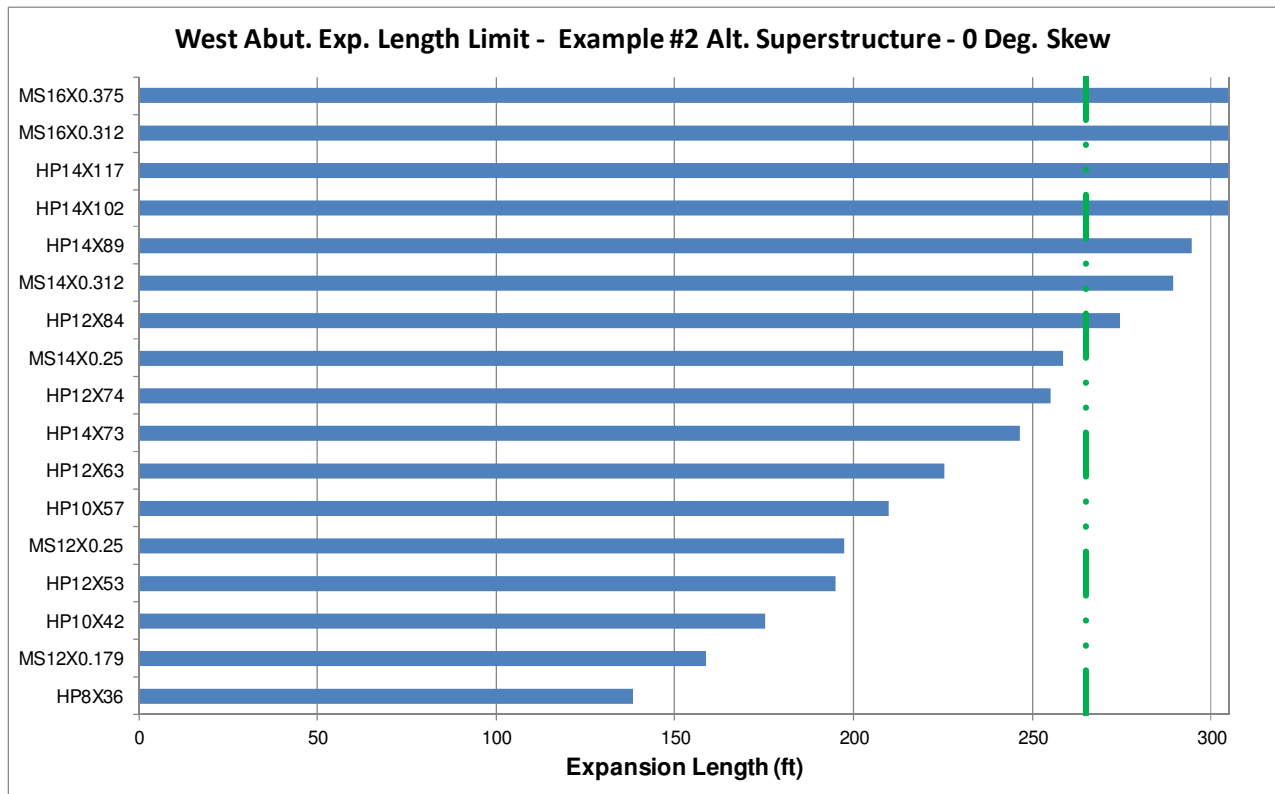
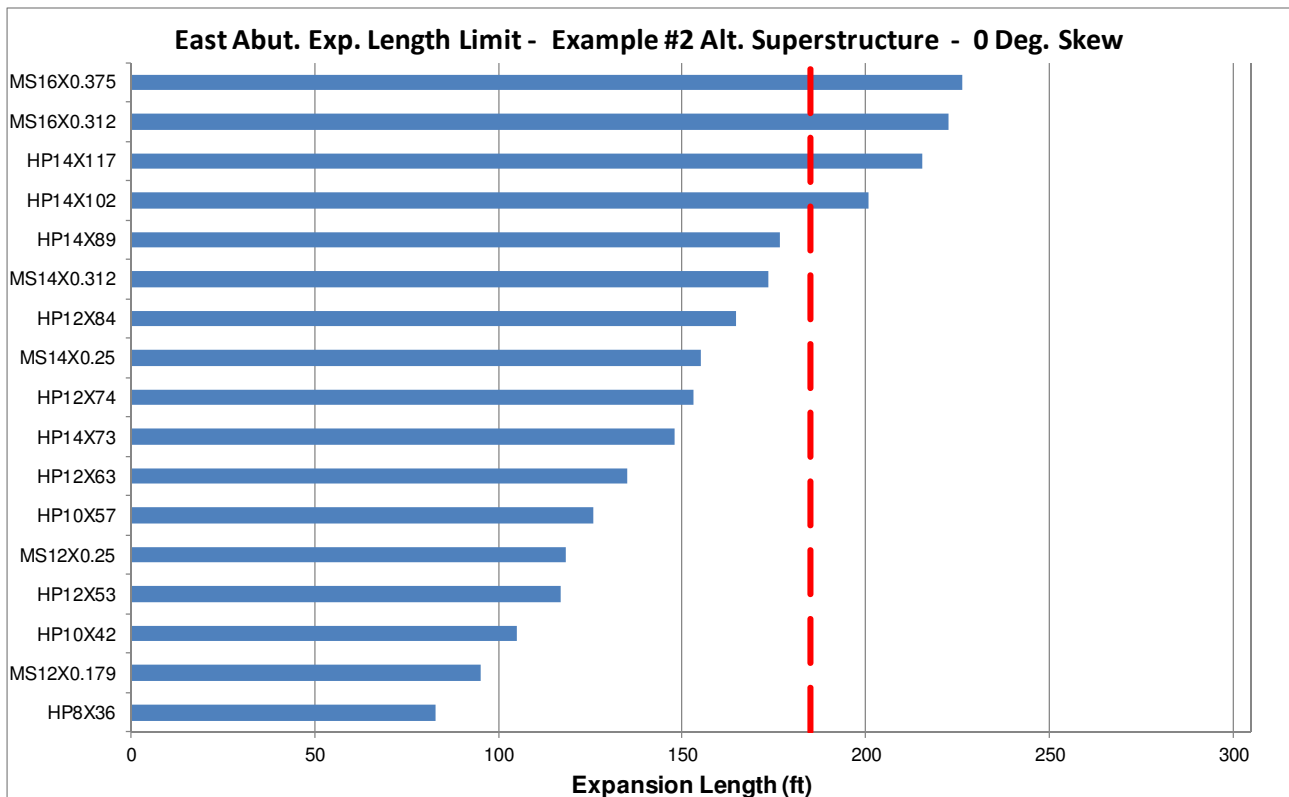
$$450 - 265 = 185 \text{ ft}$$

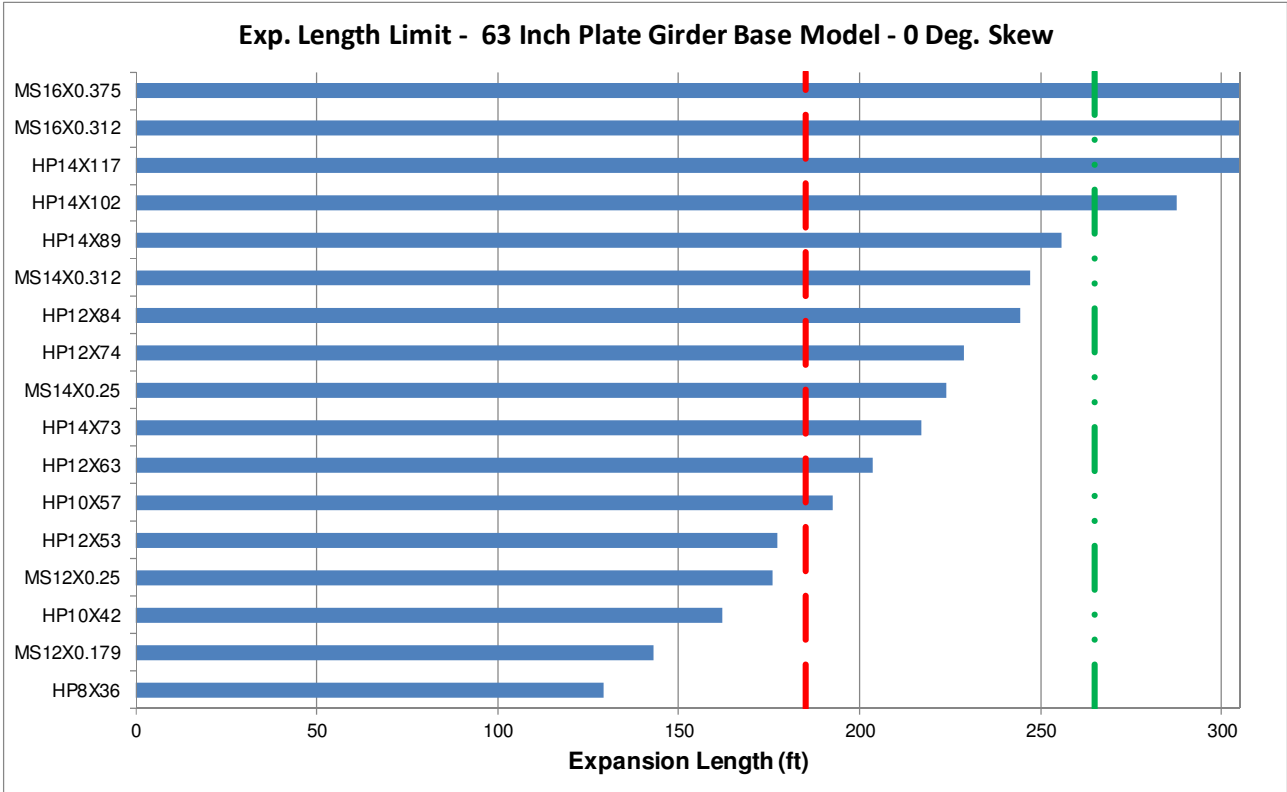
The following soil strength correction factor must be applied at the east abutment for the displacement capacity and permissible expansion length of the pile since the average Q_u for the abutment is greater than 1.5 tsf.

$$\frac{1.5}{2.5} = 0.6$$

Similar to Example 1, the following tables and graph show the various correction factors and corresponding expansion lengths for each pile.

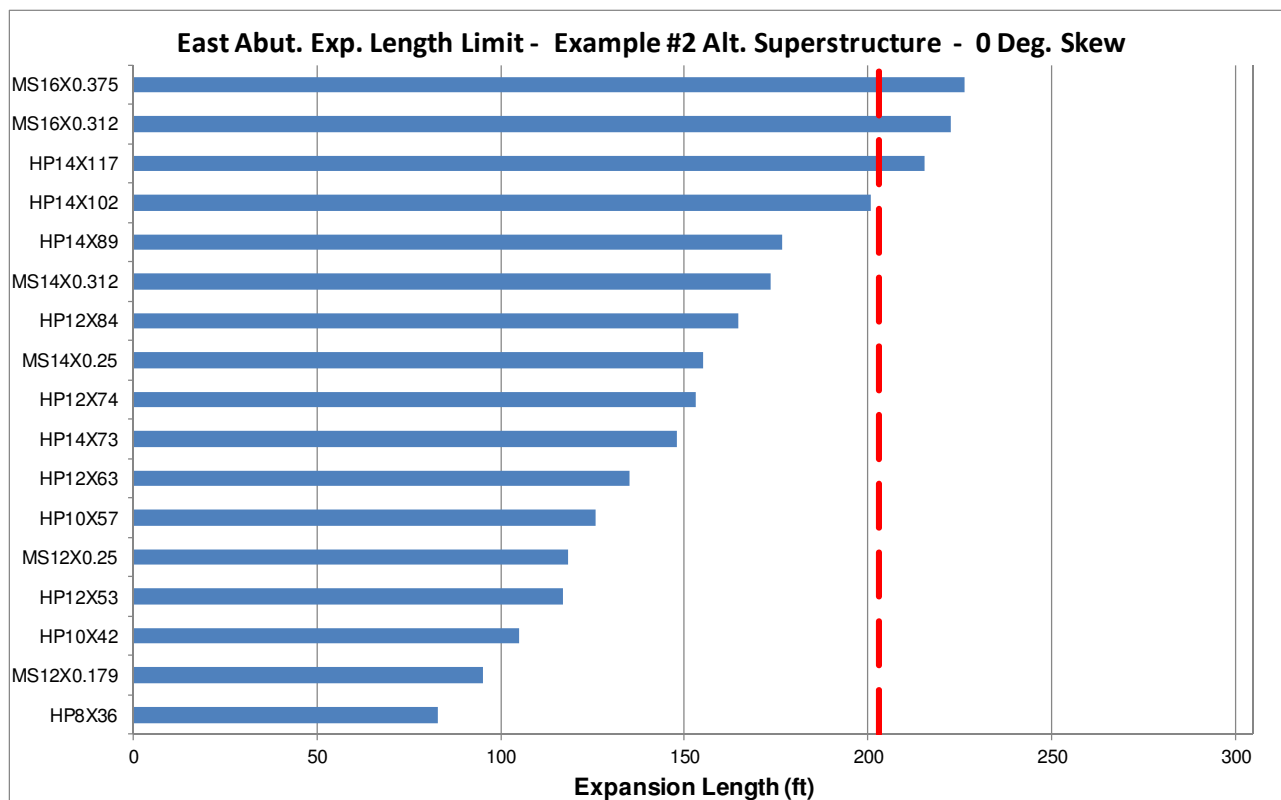
	BASE MODEL EXP. LENGTH (FT)	SUPERSTRUCTURE STIFFNESS CORRECTION FACTOR	EAST ABUTMENT		WEST ABUTMENT	
			SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT
HP14X117	305	1.18	0.6	0.71	1	1.18
HP14X102	288	1.16	0.6	0.70	1	1.16
HP14X89	256	1.15	0.6	0.69	1	1.15
HP14X73	217	1.14	0.6	0.68	1	1.14
HP12X84	244	1.12	0.6	0.67	1	1.12
HP12X74	229	1.12	0.6	0.67	1	1.12
HP12X63	204	1.11	0.6	0.66	1	1.11
HP12X53	177	1.10	0.6	0.66	1	1.10
HP10X57	193	1.09	0.6	0.65	1	1.09
HP10X42	162	1.08	0.6	0.65	1	1.08
HP8X36	129	1.07	0.6	0.64	1	1.07
MS12X0.25	176	1.12	0.6	0.67	1	1.12
MS14X0.25	224	1.16	0.6	0.69	1	1.16
MS14X0.312	247	1.17	0.6	0.70	1	1.17
MS16X0.312	305	1.22	0.6	0.73	1	1.22
MS16X0.375	305	1.24	0.6	0.74	1	1.24

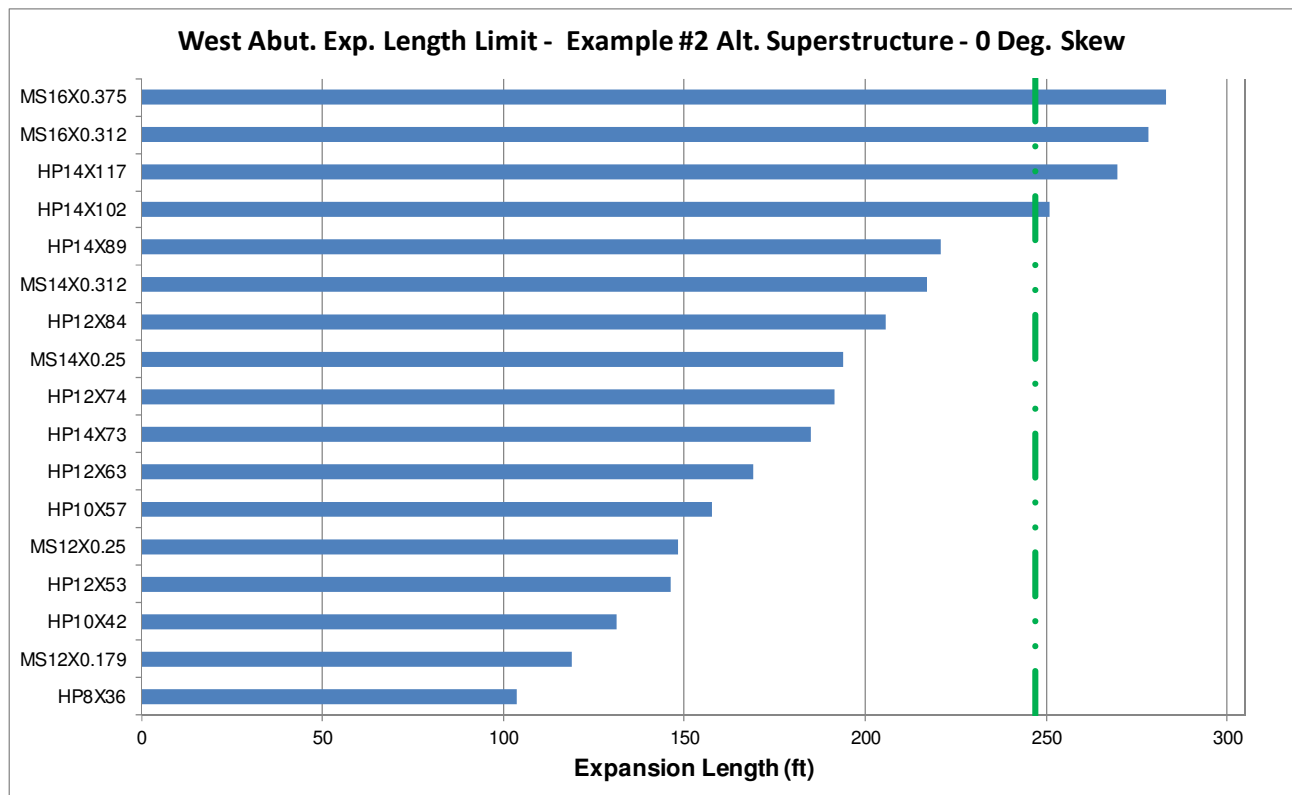




Note: If Example 2 had an average Q_u within the critical pile depth of 2.0 tsf at the west abutment and 2.5 tsf at the east abutment, the west abutment would also require a pile stiffness modifier which results in the distance from the centroid of stiffness to the west abutment and controlling expansion length increasing to approximately 247 ft. However, the permissible expansion length for the piles would also need to be adjusted for the Q_u correction factor at the west abutment by multiplying by the ratio of 1.5/2.0 (= 0.75). The following tables and graph show the various correction factors and corresponding expansion lengths for each pile. Conversely, if the controlling expansion length is divided by the Q_u correction factor for comparison with the pile limits for the base case and Q_u of 1.5 tsf (i.e., 247/0.75), this would result in an effective expansion length of approximately 329 ft which exceeds the maximum length of 305 ft and suggests the structure is unacceptable for integral abutments. However, by considering the benefit of the increased flexibility of the alternative superstructure for the subject example, the structure is able to utilize integral abutments.

	BASE MODEL EXP. LENGTH (FT)	SUPERSTRUCTURE STIFFNESS CORRECTION FACTOR	EAST ABUTMENT		WEST ABUTMENT	
			SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT
HP14X117	305	1.18	0.6	0.71	0.75	0.88
HP14X102	288	1.16	0.6	0.70	0.75	0.87
HP14X89	256	1.15	0.6	0.69	0.75	0.86
HP14X73	217	1.14	0.6	0.68	0.75	0.85
HP12X84	244	1.12	0.6	0.67	0.75	0.84
HP12X74	229	1.12	0.6	0.67	0.75	0.84
HP12X63	204	1.11	0.6	0.66	0.75	0.83
HP12X53	177	1.10	0.6	0.66	0.75	0.82
HP10X57	193	1.09	0.6	0.65	0.75	0.82
HP10X42	162	1.08	0.6	0.65	0.75	0.81
HP8X36	129	1.07	0.6	0.64	0.75	0.80
MS12X0.25	176	1.12	0.6	0.67	0.75	0.84
MS14X0.25	224	1.16	0.6	0.69	0.75	0.87
MS14X0.312	247	1.17	0.6	0.70	0.75	0.88
MS16X0.312	305	1.22	0.6	0.73	0.75	0.91
MS16X0.375	305	1.24	0.6	0.74	0.75	0.93





Example 3

This example is similar to Example 2 (a continuous 450 ft. long structure consisting of 3 – 150 ft. spans; except the average Q_u at west abutment = 1.5 tsf, the average Q_u at east abutment = 2.0 tsf), and the structure is flared. The west abutment is wider than the east abutment and has 10 piles compared to 6 piles at the east abutment.

Determine the centroid of stiffness from the west abutment.

$$\sum_{\text{Stiff. W. Abut.}} = \frac{(10 \text{ piles})(0 \text{ ft.}) + (6 \text{ piles})(1.18)(450 \text{ ft.})}{(10 \text{ piles}) + (6 \text{ piles})(1.18)} = 186.5 \text{ ft.}$$

The distance from the centroid of stiffness to the centerline of the east abutment is 263.5 ft. and is the controlling expansion length. However, because the Q_u at the east abutment is 2.0 tsf, the Q_u correction factor would cause the EEL to be:

$$(263.5 \text{ ft.}) \frac{(2.0 \text{ tsf.})}{(1.5)} = 351.3 \text{ ft.}$$

The Integral Abutment Pile Selection Chart indicates that this structure cannot be integral.