Article 5 Detail Design of Bridge Abutment in AASHTO-LRFD

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Detail Design of Bridge Abutment in AASHTO-LRFD

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Abstract: *This chapter describes the step wise design procedure for Abutment-Abutment cap-Pile Cap-Piles with structural reinforcement details by describing about the general conditions and common practices, design criteria, bridge length limits, soil conditions, skew angle, Alignment and geometry, grade, girder details, arrangement of piles under pile cap, dynamic loads allowance in pile design, construction sequence, negative moment connection Between the Integral abutment and the superstructure, wing walls, approach Slab, expansion joints and bearing pads.*

5.0   General

General considerations and common practices

Integral abutments are used to eliminate expansion joints at the end of a bridge. A joint-less bridge concept is defined as any design procedure that attempts to achieve certain goals by eliminating as many expansion joints as possible. The ideal joint-less bridge, for example, contains no expansion joints in the superstructure, substructure or deck.

Integral abutments are generally founded on one row of piles made of steel or concrete. The use of one row of piles reduces the stiffness of the abutment and allows the abutment to translate parallel to the longitudinal axis of the bridge. This permits the elimination of expansion joints and movable bearings. Because the earth pressure on the two end abutments is resisted by compression in the superstructure, the piles supporting the integral abutments, unlike the piles supporting conventional abutments, do not need to be designed to resist the earth loads on the abutments.

Note: The various formulas used in the design may be seen from the design Excel Worksheet of software ASTRA Pro by downloading from web site www.techsoftglobal.com under 'downloads'. The values in 'Red and Blue Color' in Excel Worksheet, are Design Input Data which may be changed by the User, but in Tab Pages the colored values are taken from other pages and should not be changed by the user. For any query write to techsoftinfra@gmail.com .

5.1 Abutment Design in AASHTO-LRFD

5.1.1 Integral Abutment AASHTO-LRFD Design Step 7.1

5.1.1.1 General Considerations and Common Practices

Integral abutments are used to eliminate expansion joints at the end of a bridge. They often result in "Joint less Bridges" and serve to accomplish the following desirable objectives:

· Long-term serviceability of the structure
· Minimal maintenance requirements
· Economical construction
· Improved aesthetics and safety considerations

A joint less bridge concept is defined as any design procedure that attempts to achieve the goals listed above by eliminating as many expansion joints as possible. The ideal Joint less bridge, for example, contains no expansion joints in the superstructure, substructure or deck.

Integral abutments are generally founded on one row of piles made of steel or concrete. The use of one row of piles reduces the stiffness of the abutment and allows the abutment

to translate parallel to the longitudinal axis of the bridge. This permits the elimination of expansion joints and movable bearings. Because the earth pressure on the two end abutments is resisted by compression in the superstructure, the piles supporting the integral abutments, unlike the piles supporting conventional abutments, do not need to be designed to resist the earth loads on the abutments.

When expansion joints are completely eliminated from a bridge, thermal stresses must be relieved or accounted for in some manner. The integral abutment bridge concept is based on the assumption that due to the flexibility of the piles, thermal stresses are transferred to the substructure by way of a rigid connection, i.e. the uniform temperature change causes the abutment to translate without rotation. The concrete abutment contains sufficient bulk to be considered as a rigid mass. A positive connection to the girders is generally provided by encasing girder ends in the reinforced concrete   Backwall. This provides for full transfer of forces due to thermal movements and live load rotational displacement experienced by the abutment piles.

5.1.1.2 Design Criteria

Neither the AASHTO-LRFD Specifications nor the AASHTO-Standard Specifications contain detailed design criteria for integral abutments. In the absence of universally accepted design criteria, many states have developed their own design guidelines. These guidelines have evolved over time and rely heavily on past experience with integral abutments at a specific area. There are currently two distinctive approaches used to design integral abutments:

·   One group of states design the piles of an integral abutment to resist only gravity loads applied to the abutment. No consideration is given to the effect of the horizontal displacement of the abutment on the pile loads and/or pile resistance. This approach is simple and has been used successfully. When the bridge is outside a certain range set by the state, e.g. long bridges, other considerations are taken into account in the design.

·   The second approach accounts for effects of different loads, in additional to gravity loads, when calculating pile loads. It also takes into account the effect of the horizontal movements on the pile load resistance. One state that has detailed design procedures following this approach is Pennsylvania.

The following discussion does not follow the practices of a specific state; it provides a general overview of the current state-of-practice.

5.1.1.3 Bridge Length Limits

Most states set a limit on the bridge length of joint less bridges beyond which the bridge is not considered a "typical bridge" and more detailed analysis is taken into account. Typically, the bridge length is based on assuming that the total increase of the bridge length under uniform temperature change from the extreme low to the extreme high temperature is 4 inches. This means that the movement at the top of the pile at each end is 2 inches or, when the bridge is constructed at the median temperature, a 1 inch displacement in either direction. This results in a maximum bridge length of 600 ft for concrete bridges and 400 ft. for steel bridges at locations where the climate is defined as "Moderate" in accordance to S3.12.2.1. The maximum length is shorter for regions defined as having a "cold" climate.

5.1.1.4 Soil Conditions

The above length limits assume that the soil conditions at the bridge location and behind the abutment are such that the abutment may translate with relatively low soil resistance. Therefore, most jurisdictions specify select granular fill for use behind integral abutments. In addition, the fill within a few feet behind the integral abutment is typically lightly compacted using a vibratory plate compactor (jumping jack). When bedrock, stiff soil and/or boulders exist in the top layer of the soil (approximately the top 12 to 15 ft.), it is typically required that oversized holes be drilled to a depth of approximately 15 ft.; the piles are then installed in the oversized holes. Subsequently, the holes are filled with sand. This procedure is intended to allow the piles to translate with minimal resistance.

5.1.1.5 Skew Angle

Earth pressure acts in a direction perpendicular to the abutments. For skewed bridges, the earth pressure forces on the two abutments produce a torque that causes the bridge to twist in plan. Limiting the skew angle reduces this effect. For skewed, continuous bridges, the twisting torque also results in additional forces acting on intermediate bents. In addition, sharp skews are suspected to have caused cracking in some abutment Back-walls due to rotation and thermal movements. This cracking may be reduced or eliminated by limiting the skew. Limiting the skew will also reduce or eliminate design uncertainties, backfill compaction difficulty and the additional design and details that would need to be worked out for the abutment U-Wing walls and approach slab.

Currently, there are no universally accepted limits on the degree of skew for integral abutment bridges.

5.1.1.6 Horizontal Alignment and Bridge Plan Geometry

With relatively few exceptions, integral abutments are typically used for straight bridges. For curved superstructures, the effect of the compression force resulting from the earth pressure on the abutment is a cause for concern. For bridges with variable width, the difference in the length of the abutments results in unbalanced earth pressure forces if the two abutments are to move the same distance. To maintain force equilibrium, it is expected that the shorter abutment will deflect more than the longer one. This difference should be considered when determining the actual expected movement of the two abutments as well as in the design of the piles and the expansion joints at the end of the approach slabs (if used).

5.1.1.7 Vertical Grade

Some jurisdictions impose a limit on the maximum vertical grade between abutments. These limits are intended to reduce the effect of the abutment earth pressure forces on the abutment vertical reactions.

5.1.1.8 Girder Types, Maximum Depth and Placement

Integral abutments have been used for bridges with steel I-beams, concrete I-beams, concrete bulb tees and concrete spread box beams.

Deeper abutments are subjected to larger earth pressure forces and, therefore, less flexible. Girder depth limits have been imposed by some jurisdictions based on past successful practices and are meant to ensure a reasonable level of abutment flexibility. Soil conditions and the length of the bridge should be considered when determining maximum depth limits. A maximum girder depth of 6 ft. has been used in the past. Deeper girders may be allowed when the soil conditions are favorable and the total length of the bridge is relatively short.

5.1.1.9 Type and Orientation of Piles

Integral abutments have been constructed using steel H-piles, concrete-filled steel pipe piles and reinforced and pre-stressed concrete piles. For H-piles, there is no commonly used orientation of the piles. In the past, H-piles have been placed both with their strong axis parallel to the girder's longitudinal axis and in the perpendicular direction. Both orientations provide satisfactory results.

5.1.1.10   Consideration of dynamic load allowance in pile design

Traditionally, dynamic load allowance is not considered in foundation design. However, for integral abutment piles, it may be argued that the dynamic load allowance should be considered in the design of the top portion of the pile. The rationale for this requirement is that the piles are almost attached to the superstructure, therefore, the top portions of the piles do no benefit from the damping effect of the soil.

5.1.1.11   Construction sequence

Typically, the connection between the girders and the integral abutment is made after the deck is poured. The end portion of the deck and the Backwall of the abutment are usually poured at the same time. This sequence is intended to allow the dead load rotation of the girder ends to take place without transferring these rotations to the piles.

Two integral abutment construction sequences have been used in the past:

·   One-stage construction:

In this construction sequence, two piles are placed adjacent to each girder, one pile on each side of the girder. A steel angle is connected to the two piles and the girder is seated on the steel angle. The abutment pier cap (the portion below the bottom of the beam) and the end diaphragm or Backwall (the portion encasing the ends of the beams) are poured at the same time. The abutment is typically poured at the time the deck in the end span is poured.

·   Two-stage construction:

Stage 1:   A pile cap supported on one row of vertical piles is constructed. The piles do not have to line up with the girders. The top of the pile cap reaches the bottom of the bearing pads under the girders. The top of the pile cap is required to be smooth in the area directly under the girders and a strip approximately 4 in. wide around this area. Other areas are typically roughened (i.e. rake finished).

Stage 2:   After pouring the entire deck slab, except for the portions of the deck immediately adjacent to the integral abutment (approximately the end 4 ft. of the deck from the front face of the abutment) the end diaphragm (Backwall) encasing the ends of the bridge girders is poured. The end portion of the deck is poured simultaneously with the end diaphragm.

5.1.1.12 Negative Moment Connection between the Integral Abutment and the Superstructure

The rigid connection between the superstructure and the integral abutment results in the development of negative moments at this location. Some early integral abutments showed signs of deck cracking parallel to the integral abutments in the end section of the deck due to the lack of proper reinforcement to resist this moment. This cracking was prevented by specifying additional reinforcement connecting the deck to the back (fill) face of the abutment. This reinforcement may be designed to resist the maximum moment that may be transferred from the integral abutment to the superstructure. This moment is taken equal to the sum of the plastic moments of the integral abutment piles. The section depth used to design these bars may be taken equal to the girder depth plus the deck thickness. The length of the bars extending into the deck is typically specified by the bridge owner. This length is based on the length required for the superstructure dead load positive moment to overcome the connection negative moment.

5.1.1.13 Wing walls

Typically, U-wing walls (wing walls parallel to the longitudinal axis of the bridge) are used in conjunction with integral abutments. A chamfer (typically 1 ft.) is used between the abutment and the wing walls to minimize concrete shrinkage cracking caused by the abrupt change in thickness at the connection.

5.1.1.14 Approach slab

Bridges with integral abutments were constructed in the past with and without approach slabs. Typically, bridges without approach slabs are located on secondary roads that have asphalt pavements. Traffic and seasonal movements of the integral abutments cause the fill behind the abutment to shift and to self compact. This often caused settlement of the pavement directly adjacent to the abutment.

Providing a reinforced concrete approach slab tied to the bridge deck moves the expansion joint away from the end of the bridge. In addition, the approach slab bridges cover the area where the fill behind the abutment settles due to traffic compaction and movements of the abutment. It also prevents undermining of the abutments due to drainage at the bridge ends. Typically, approach slabs are cast on polyethylene sheets to minimize the friction under the approach slab when the abutment moves.

The approach slab typically rests on the abutment at one end and on a sleeper slab at the other. The approach slab differs from typical roadway pavement since the soil under the approach slab is more likely to settle unevenly resulting in the approach slab bridging a longer length than expected for roadway pavement. Typically, the soil support under the approach slab is ignored in the design and the approach slab is designed as a one-way slab bridging the length between the integral abutment and the sleeper slab. Therequired length of the approach slab depends on the total depth of the integral abutment.

The sleeper slab should be placed outside the area where the soil is expected to be affected by the movement of the integral abutment. This distance is a function of the type of fill and the degree of compaction.

Due to the difference in stiffness between the superstructure and the approach slab, the interface between the integral abutment and the approach slab should preferably allow the approach slab to rotate freely at the end connected to the abutment. The reinforcement bars connecting the abutment to the approach slab should be placed such that the rotational restraint provided by these bars is minimized.

A contraction joint is placed at the interface between the approach slab and the integral abutment. The contraction joint at this location provides a controlled crack location rather than allowing a random crack pattern to develop.

5.1.1.15   Expansion joints

Typically, no expansion joints are provided at the interface between the approach slab and the roadway pavement when the bridge total length is relatively small and the roadway uses flexible pavement. For other cases, an expansion joint is typically used.

5.1.1.16   Bearing pads

Plain elastomeric bearing pads are placed under all girders when the integral abutment is constructed using the two-stage sequence described above. The bearing pads are intended to act as leveling pads and typically vary from ½ to ¾ in. thick. The pad length parallel to the girder's longitudinal axis varies depending on the bridge owner's specifications and the pad length in the perpendicular direction varies depending on the width of the girder bottom flange and the owner's specifications. It is recommended to block the area under the girders that is not in contact with the bearing pads using backer rods. Blocking this area is intended to prevent honeycombing of the surrounding concrete. Honeycombing will take place when the cement paste enters the gap between the bottom of girder and the top of the pile cap in the area under the girders not in contact with the bearing pads.

5.1.1.17 AASHTO-LRFD Design Step 7.1.1 - Gravity loads

Interior girder: un-factored loads
(See Table 5.3-3 for girder end shears)

Non-composite:

Girder	=	61.6	k
Slab and haunch	=	62.2	k
Exterior diaphragm	=	2.5	k
Total NC	=	126.3	k

Composite:

Parapets	=	8.9	k
Future wearing surface	=	12	k

Live load:
Maximum truck per lane (without impact or distribution factors) = 64.42 k
Minimum truck per lane (without impact or distribution factors) = -6.68   k
Maximum lane per lane = 30.81 k
Minimum lane per lane = -4.39   k

Exterior girder: un-factored loads
(See Table 5.3-7 for girder end shears)

Non-composite:

Girder	=	61.6	k
Slab and haunch	=	55.1	k
Exterior diaphragm	=	1.3	k
Total NC	=	118	k

Composite:

Parapets	=	8.9 k
Future wearing surface	=	8.1 k

Live load:
Maximum truck per lane (without impact or distribution factors) = 64.42 k
Minimum truck per lane (without impact or distribution factors) = -6.68 k
Maximum lane per lane = 30.81 k
Minimum lane per lane = -4.39 k
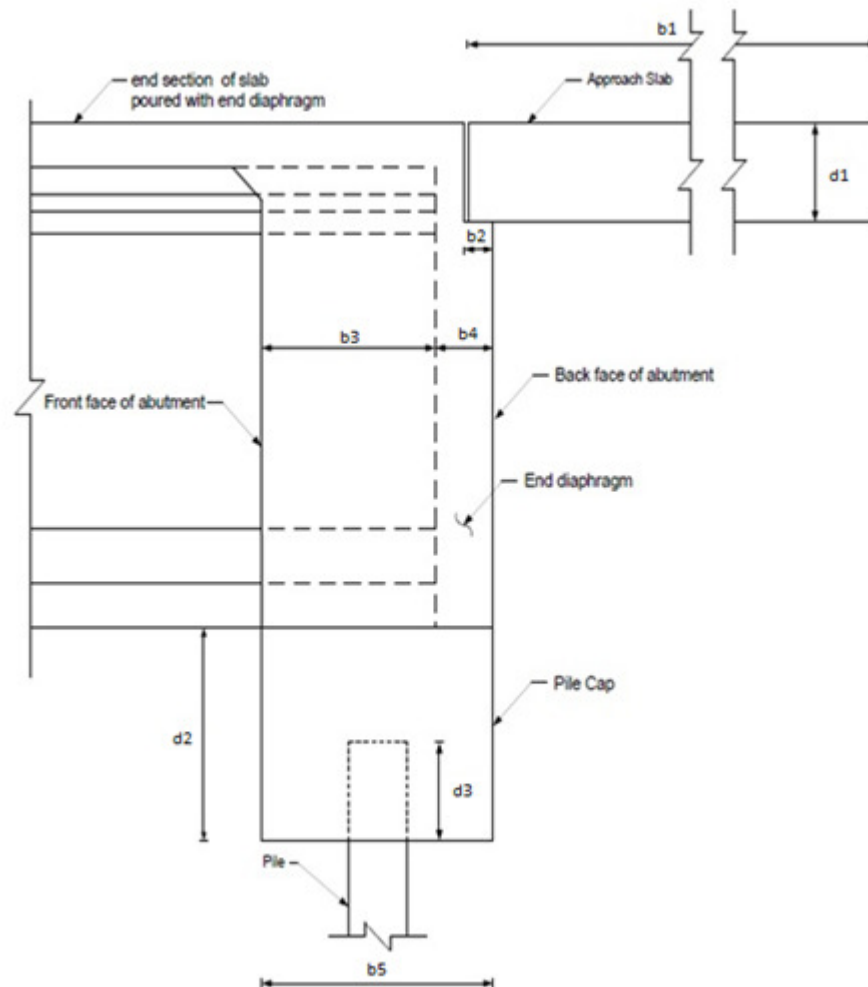
Figure 5.1   (AASHTO-LRFD 7.1-1) – General View of an Integral Abutment Showing Dimensions Used for the Example

b1 =   25   ft 300 in
b2 =   0.5 ft 6   in
b3 =   2   ft 24   in
b4 =   1   ft 12   in
b5 =   3   ft 36   in
d1 =   1.5   ft 18   in
d2 =   3.25 ft 39   in
d3 =   1.5   ft 18   in
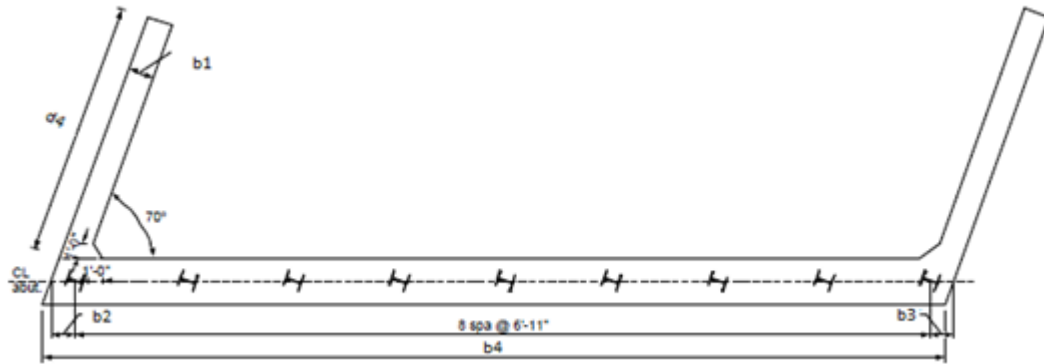
Figure 5.2   (AASHTO-LRFD Figure 7.1-2) – Plan View of the Integral Abutment

b1 =	1.6875	ft	20.25 in
b2 =	1.792	ft	21.504 in
b3 =	1.792	ft	21.504 in
b4 =	58.93	ft	707.16 in
d1 =	15	ft	180 in
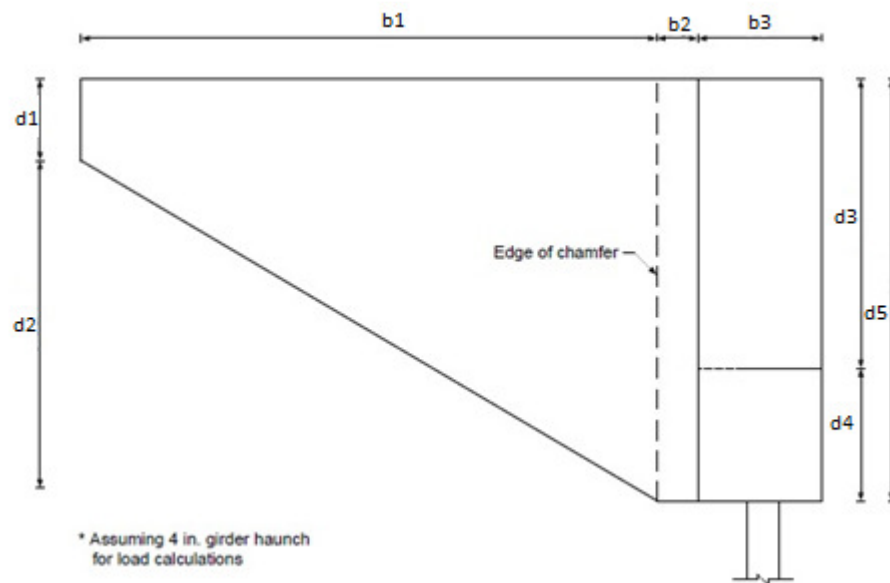
Figure 5.3   (AASHTO-LRFD Figure 7.1-3) – Elevation View of Integral Abutment and Tapered Wingwall

Detail Design of Bridge Abutment in AASHTO-LRFD

b1 =	14	ft	168	in
b2 =	1	ft	12	in
b3 =	3	ft	36	in
d1 =	2	ft	24	in
d2 =	8.3125	ft	99.75	in
d3 =	7.0625	ft	84.75	in
d4 =	3.25	ft	39	in
d5 =	10.3125	ft	123.75	in

In the next section, "w" and "P" denote the load per unit length and the total load, respectively. The subscripts denote the substructure component. Dimensions for each component are given in Figures 7.1-1 through 7.1-3.

Pile cap: un-factored loading

| Pile cap length along the skew | = | 55.354/cos 20 |
| | = | 58.91 |

| W_{cap} | = | 3.25X(3)X(0.15) |
| | = | 1.46 k/ft |

OR'
| P_{cap} | = | 1.46X(58.91) |
| | = | 86.01 k |

Concrete weight from the end diaphragm (approximate, girder volume not removed):

unfactored loading

Assuming bearing pad thickness of ¾ in., girder height of 72 in., haunch thickness of 4 in., and deck thickness of 8 in.:

pad thickness	=	0.75	in
girder height	=	72	in
haunch thickness	=	4	in
deck thickness	=	8	in

$W_{end\ dia}$ =   3((0.75 +72 +4+8)/12)(0.15)

         =   3.18   k/ft

OR'

$P_{end\ dia}$ =   3.18X(58.91)

         =   187.33 k

Wing wall: un-factored load

A_{wing} = (123.75/12)X(15) -½ (14)X(99.75/12)

         = 96.5 ft2

Wing wall thickness   = parapet thickness at the base

         = 20.25   in.   (given in Section 4)

Wing wall weight   = 96.5X(20.25/12)X(0.15)

         = 24.43 k

Chamfer weight   = (123.75/12)X(1)X(1)X(0.15)/2

         = 0.77 k

Notice that the chamfer weight is insignificant and is not equal for the two sides of the bridge due to the skew. For simplicity, it was calculated based on a right angle triangle and the same weight is used for both sides.

Weight of two wing walls plus chamfer   = 2X(24.43 + 0.77)

         = 50.4 k

Parapet weight   = 0.65 k/ft   (given in Section 5.2)

Parapet length on wing wall and abutment = 15 + 3/ sin 70

         = 8.19 ft.

$P_{parapet}$ =   2X(0.65)X(18.19)

         =   23.65 k   total weight

Approach slab load acting on the integral abutment: un-factored loading

Approach slab length   = 25 ft

Approach slab width between parapets   = 58.91 – 2((20.25/12)/sin 70)

         = 55.32 ft

Detail Design of Bridge Abutment in AASHTO-LRFD

Self weight of the approach slab:

$W_{approach\ slab}$	=	½ X(25)X(1.5)X(0.15)
	=	2.81 k/ft
OR'		
$P_{approach\ slab}$	=	2.81X(55.32)
	=	155.4 k

Future wearing surface acting on the approach slab (assuming 25 psf):

W_{FWS}	=	½ X(0.025)X(25)
	=	0.31 k/ft
OR'		
P_{FWS}	=	0.31X55.32
	=	17.15 k

Live load on the approach slab, reaction on integral abutment:

$P_{lane\ load}$	=	½ X(0.64)X(25)		S3.6124
	=	8	k (one lane)	

Notice that one truck is allowed in each traffic lane and that the truck load is included in the girder reactions. Therefore, no trucks were assumed to exist on the approach slab and only the uniform load was considered.

5.1.1.18 AASHTO-LRFD Design Step 7.1.2 - Pile Cap Design

The girder reactions, interior and exterior, are required for the design of the abutment pile cap. Notice that neither the piles nor the abutment beam are infinitely rigid. Therefore, loads on the piles due to live loads are affected by the location of the live load across the width of the integral abutment. Moving the live load reaction across the integral abutment and trying to maximize the load on a specific pile by changing the number of loaded traffic lanes is not typically done when designing integral abutments. As a simplification, the live load is assumed to exist on all traffic lanes and is distributed equally to all girders in the bridge cross section. The sum of all dead and live loads on the abutment is then distributed equally to all piles supporting the abutment.

The maximum number of traffic lanes allowed on the bridge based on the available width (52 ft. between gutter lines) is:

N_{lanes}	=	52 ft./12 ft.			
per lane	=	4.33	say	4	lanes
Number of Girders	=	6			

Factored dead load plus live load reactions for one interior girder, Strength I limit state controls (assume the abutment is poured in two stages as discussed earlier):

Maximum reaction Stage I:

$P_{SI(I)}$	=	1.25 X(girder + slab + haunch)
	=	1.25 x 126.3
	=	158 k

Notice that construction loads should be added to the above reaction if construction equipment is allowed on the bridge before pouring the Backwall (Stage II).

Maximum reaction for Final Stage:
Including the dynamic load allowance (for design of the pile cap top portion of the piles):

$P_{FNL(I)}$	=	1.25 x (DC) + 1.50 x (DW) + 1.75 x (LL + IM) x (Nlanes)/Ngirders
	=	1.25X(126.3 + 8.9) +1.5X(12) +1.75X(1.33X(64.42) +30.81)X4/6
	=	323 k

Without the dynamic load allowance (for design of the lower portion of the piles):

$P_{FNL(I)}$ = 298.3 k

Factored dead load plus live load reactions for one exterior girder, Strength I limit state controls:

Maximum reaction Stage I:

$P_{SI(E)}$ = 1.25X118

= 147.5 k

Notice that construction loads should be added to the above reaction.

Maximum reaction for Final Stage:

Including the dynamic load allowance:

$P_{FNL(E)}$ = 1.25 x (DC) + 1.50 x (DW) + 1.75 x (LL + IM) x (Nlanes)/Ngirders

= 1.25 x (118 +8.9) + 1.5 x (8.1) +1.75 x (1.33 x (64.42) +30.81) x4/6

= 306.7 k

Without the dynamic load allowance:

$P_{FNL(E)}$ = 281.8 k

5.1.1.19 AASHTO-LRFD Design Step 7.1.3   Design of Piles

Typically, integral abutments may be supported on end bearing piles or friction piles. Reinforced and pre-stressed concrete piles, concrete-filled steel pipe piles or steel H-piles may be used. Steel H-piles will be used in this example.

Typically, the minimum distance between the piles and the end of the abutment, measured along the skew, is taken as 1'-6" and the maximum distance is usually 2'-6". These distances may vary from one jurisdiction to another. The piles are assumed to be embedded 1'-6" into the abutment. Maximum pile spacing is assumed to be 10 ft. The minimum pile spacing requirements of S10.7.1.5 shall apply.

• From S10.7.1.5, the center-to-center pile spacing shall not be less than the greater of 30.0 in. or 2.5 pile diameters (or widths). The edge distance from the side of any pile to the nearest edge of the footing shall be greater than 9.0 in.

• According to S10.7.1.5, where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piles, the concrete cover at the sides of the piles shall be greater than 6.0 in., plus an allowance for permissible pile misalignment, and the piles shall project at least 6.0 in. into the cap. This provision is specifically for bent caps, therefore, keep 1'-6" pile projection for integral abutment to allow the development of moments in the piles due to movements of the abutment without distressing the surrounding concrete.

From Figure 7.1-2, steel H-piles are shown to be driven with their weak axis perpendicular to the centerline of the beams. As discussed earlier, piles were also successfully driven with their strong axis perpendicular to the centerline of the beams in the past.

According to S10.7.4.1, the structural design of driven concrete, steel, and timber piles must be in accordance with the provisions of Sections S5, S6, and S8 respectively. Articles S5.7.4, S5.13.4, S6.15, S8.4.13, and S8.5.2.2 contain specific provisions for concrete, steel, and wood piles. Design of piles supporting axial load only requires an allowance for unintended eccentricity. For the steel H-piles used in this example, this has been accounted for by the resistance factors in S6.5.4.2 for steel piles.

General pile design

As indicated earlier, piles in this example are designed for gravity loads only.

Generally, the design of the piles is controlled by the minimum capacity as determined for the following cases:

•   Case A - Capacity of the pile as a structural member according to the procedures outlined in S6.15. The design for combined moment and axial force will be based on an analysis that takes the effect of the soil into account.

•   Case B - Capacity of the pile to transfer load to the ground.

•   Case C - Capacity of the ground to support the load.

For piles on competent rock, only Case A needs to be investigated.

5.1.1.20 AASHTO-LRFD Design Step 7.1.3.1 Pile compressive resistance

Pile compressive resistance (S6.15 and S6.9.2)
The factored resistance of components in compression, Pr, is taken as:

Pr = φPn   (S6.9.2.1-1)

where:

      Pn = nominal compressive resistance specified in S6.9.4 and S6.9.5 (kip)
      φc = resistance factor for axial compression, steel only as specified in S6.5.4.2
      φc =   0.5   for H-piles assuming severe driving conditions

Check the width/thickness requirements per S6.9.4.2. Assume HP12x53 piles.

Slenderness of plates must satisfy:

$$\frac{b}{t} \leq k\sqrt{\frac{E}{F_y}}$$

                  (S6.9.4.2-1)

      where:

      k =   plate buckling coefficient as specified in Table S6.9.4.2-1
      k=   0.56   for flanges and projecting legs or plates
      b =   width of plate equals one-half of flange width as specified in Table S6.9.4.2-1
              (in.)

flange width = 12.045 in
b   =   12.045/2 =   6.02   in.
t   =   flange thickness (in.)
t   =   0.435 in.
b/t   =   6.02/0.435   =   13.8
E   =   29000
fy   =   36

$$k\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29,000}{36}}$$

$$k\sqrt{\frac{E}{F_y}} = 15.9$$

15.9   >   13.8   OK

Therefore, use S6.9.4.1 to calculate the compressive resistance.

(Notice that the b/t ratio for the webs of HP sections is always within the limits of Table S6.9.4.2-1 for webs and, therefore, need not be checked.)

For piles fully embedded in soil, the section is considered continuously braced and Eq. S6.9.4.1-1
is reduced to Pn = FyAs.

As =   15.5   in2

Pn =   36X15.5
=   558   k

Therefore, the factored resistance of components in compression, Pr, is taken as:

$P_r = \varphi P_n$ =   0.5x558   =   279   k

The above capacity applies to the pile at its lower end where damage from driving may have taken place. At the top of the pile, higher resistance factors that do not account for damage may be used. For piles designed for gravity loads only, as in this example, the resistance at the lower end will always control due to the lower resistance factor regardless if the dynamic load allowance is considered in determining the load at the top of the pile or not (notice that the dynamic load allowance is not considered in determining the load at the bottom of the pile).

5.1.1.21 AASHTO-LRFD Design Step 7.1.3.2 Determine of number of piles

Determine the number of piles required

Maximum total girder reactions for Stage I (detailed calculations of girder reactions shown earlier):

$P_{SI\ (Total)}$ = (2X147.5) + (4X158)

= 927   k

Maximum total girder reaction for final stage not including the dynamic load allowance (detailed calculations of girder reactions shown earlier):

$P_{FNL(Total)}$ = (2X281.8) + (4X298.3)

= 1756.8 k

Maximum factored DL + LL on the abutment, Strength I limit state controls:

PStr. I = PFNL(Total) + 1.25(DC) + 1.50(DW) + 1.75(LLmax)(Nlanes)

= 1756.8 +1.25X(86.01+187.33+50.4+23.65+155.4)

+1.5X(17.15)+1.75X(8)X(4)

= 2467   k

where:
"PFNL(Total)" is the total factored DL + LL reaction of the bridge girders on the abutment.

"DC" includes the weight of the pile cap, diaphragm, wingwalls, approach slab and parapet on the wingwalls.

"DW" includes the weight of the future wearing surface on the approach slab.

"LLmax" is the live load reaction from the approach slab transferred to the abutment
 (per lane)

"Nlanes" is the maximum number of traffic lanes that fit on the approach slab, 4 lanes.

Therefore, the number of piles required to resist the applied dead and live loads is:

Npiles = PStr. I/Pr

= 2467/279

= 8.84   piles, say   9   piles

5.1.1.22 AASHTO-LRFD Design Step 7.1.3.3   Pile spacing

<u>Pile spacing</u>

Total length of the pile cap   =   58.91   ft.
Assume pile spacing is 6'-11" =   6.917   ft.
which provides more than the recommended edge distance of 1'-6" for the piles.

Pile end distance   =   (58.91 – (8X6.917))/2
                         =   1.8   ft   (1'-9 ½ ")

5.1.1.23 AASHTO-LRFD Design Step 7.1.4 Back-wall design

The thickness of the abutment Back-wall is taken to be 3 ft.

Design of the pier cap for gravity loads

For an integral abutment constructed in two stages, the abutment is designed to resist gravity loads as follows:

•   Case A - The first stage of the abutment, i.e., the part of the abutment below the bearing pads, is designed to resist the self weight of the abutment, including the diaphragm, plus the reaction of the girders due to the self weight of the girder plus the deck slab and haunch.

•   Case B - The entire abutment beam, including the diaphragm, is designed under the effect of the full loads on the abutment.

Instead of analyzing the abutment beam as a continuous beam supported on rigid supports at pile locations, the following simplification is common in conducting these calculations and is used in this example:

•   Calculate moments assuming the abutment beam acting as a simple span between piles and then taking 80% of the simple span moment to account for the continuity. The location of the girder reaction is often assumed at the mid-span for moment calculations and near the end for shear calculations. This assumed position of the girders is meant to produce maximum possible load effects. Due to the relatively large dimensions of the pile cap, the required reinforcement is typically light even with this conservative simplification.

Required information:

Concrete compressive strength, f'c　 = 3　　ksi
Reinforcing steel yield strength, Fy　 = 60　　ksi
Pile spacing　　　　　　　　　　　 = 6.917　　　　ft

CASE A

The maximum factored load due to the girders and slab (from the interior girder):

Pu　 =　　　 1.5 x126.3
　　　 =　　　 189.45 k

Factored load due to the self weight of the pile cap and diaphragm:

wu　 =　　　 1.5X(1.46 + 3.18)
　　　 =　　　 6.96　　k/ft

Notice that only dead loads exist at this stage. The 1.5 load factor in the above equations is for Strength III limit state, which does not include live loads.

Flexural design for Case A

The maximum positive moment, Mu, assuming a simple span girder, is at midspan between piles. The simple span moments are reduced by 20% to account for continuity:

Mu　 =　　　　 Pul/4 + wul2/8
　　　 =　　　 0.8 x (189.45 x 6.917/4 +(6.96 x6.917)^2/8)
　　　 =　　　 295.4　　k-ft

Determine the required reinforcing at the bottom of the pile cap.

Mr = φMn　　　　　　　　　　 (S5.7.3.2.1-1)

The nominal flexural resistance, Mn, is calculated using Eq. (S5.7.3.2.2-1).

Mn = Asfy(ds – a/2)　　　　　　　 (S5.7.3.2.2-1)

where:

    As   =   area of non-pre-stressed tension reinforcement (in2), notice that available room will only allow four bars, two on either side of the piles. Use 4 #8 bars.

            =   4 X 0.79

            =   3.16   in2

fy =   specified yield strength of reinforcing bars (ksi)

fy =   60   ksi

ds =   distance from the extreme compression fiber to the centroid of the non-pre-stressed tensile reinforcement (in.)

Depth of pile cap   =   3.25   ft

Bottom cover   =   3   in

Bar diameter   =   1   in

            =   depth of pile cap – bottom cover – ½ diameter bar

ds   =   (3.25X12)-3 - ½X1

     =   35.5   in.

A   =   cβ1, depth of the equivalent stress block (in.)

    =   Asfy/0.85f'cb   (S5.7.3.1.1-4)

    =   (3.16 x60)/(0.85 x3 x3 x12)

    =   2.07   in.

Mn   =   (3.16X60)X(35.5 – 2.07/2)/12

     =   544.5 k-ft

Therefore,

Mr   =   0.9 x 544.5

    =   490   k-ft   >   295.4   OK

Negative moment over the piles is taken equal to the positive moment. Use the same reinforcement at the top of the pile cap as determined for the bottom (4 #8 bars).

By inspection:

• Mr > 4/3 (Mu). This means the minimum reinforcement requirements of S5.7.3.3.2 are satisfied.

• The depth of the compression block is small relative to the section effective depth. This means that the maximum reinforcement requirements of S5.7.3.3.1 are satisfied.

Shear design for CASE A

The maximum factored shear due to the construction loads assuming the simple span condition and girder reaction at the end of the span:

$$Vu \quad = \quad Pu + wul/2$$
$$= \quad 189.45 + (6.96 \times 6.917)/2$$
$$= \quad 213.5 \quad k$$

The factored shear resistance, Vr, is calculated as:

$$Vr = \quad \varphi Vn \qquad\qquad (S5.8.2.1\text{-}2)$$

The nominal shear resistance, Vn, is calculated according to S5.8.3.3 and is the lesser of:

$$Vn = \quad Vc + Vs \qquad\qquad (S5.8.3.3\text{-}1)$$
Or,
$$Vn = \quad 0.25f'cbvdv \qquad\qquad (S5.8.3.3\text{-}2)$$

where:
$Vc =$ $0.0316\beta\sqrt{f'c}bvdv$ (S5.8.3.3-3)
β = factor indicating ability of diagonally cracked concrete to transmit tension as specified in S5.8.3.4
β = 2
$f'c$ = specified compressive strength of the concrete (ksi)
$f'c$ = 3.0   ksi
bv = effective shear width taken as the minimum web width within the depth dv as determined in S5.8.2.9 (in.)
bv = 36   in.
dv = effective shear depth as determined in S5.8.2.9 (in.)

S5.8.2.9 states that dv is not to be taken less than the greater of 0.9de or 0.72h
$dv =$ $ds - a/2$
  = 35.5- (2.07/2)
  = 34.47 in.

$0.9ds$ =   0.9X35.5
  =   31.95 in

$0.72h$ =   0.72X(3.25X12)
  =   28.08 in

Therefore, dv should be taken as 34.47   in

Detail Design of Bridge Abutment in AASHTO-LRFD

$$Vc = (0.0316X2)X\sqrt{3}X(36X34.47)$$
$$= 135.8 \ k$$

Assuming shear reinforcement is #5 @ 10 in. spacing perpendicular to the pier cap longitudinal axis.

$$Vs = Avfydv/s \hspace{2cm} (S5.8.3.3-4)$$

where:

bar area	=	0.31 in2
Av	=	area of shear reinforcement within a distance "s" (in2)
	=	2 legs (0.31)
	=	0.62 in2
s	=	spacing of stirrups (in.)
	=	10 in
Vs	=	(0.62X60X34.47)/10
	=	128.2 k

The nominal shear resistance, Vn, is taken as the smaller of:

Vn	=	135.8 +128.2
	=	264 k
Or,		
Vn	=	0.25X3X36X34.47
	=	930.7 k

Therefore, use the shear resistance due to the concrete and transverse steel reinforcement.

Vr	=	φVn			
	=	0.9 x264			
	=	237.6 k	>	213.5	OK

CASE B

The maximum factored load due to all applied dead and live loads which include the approach slab, live load on approach slab, etc. The load due to the wingwalls is not included since its load minimally affects the responses at the locations where girder reactions are applied.

Point load:

P_{Str-I} = maximum factored girder reaction calculated earlier

P_{Str-I} = 323   K

Notice that the 323 k assumes that the live load is distributed equally to all girders. This approximation is acceptable since this load is assumed to be applied at the critical location for moment and shear. Alternately, the maximum reaction from the tables in Section 5.3 may be used.

Distributed load:

W_{Str}-I =   1.25 x (cap self wt. + end diaph. + approach slab) + 1.5 x (approach FWS) + 1.75 x (approach slab lane load) x (Nlanes)/Labutment

=   1.25x(1.46+ 3.18 + 2.81) +(1.5x0.31) + (1.75x8x4)/58.93
=   10.73   k/ft

Flexural design for CASE B

The maximum positive moment is calculated assuming the girder reaction is applied at the

midspan between piles and taking 80% of the simple span moment.

Mu   =   0.8x[((323x6.917)/4) +((10.73x(6.917)^2)/8)]
=   498.2   k-ft

Determine the required reinforcing at the bottom of the pile cap.

Mr   = φMn   (S5.7.3.2.1-1)
and
Mn   = As x fy x (ds – a/2)   (S5.7.3.2.2-1)

where:

      As   =   use 4 #8 bars

             =   4x0.79

             =   3.16   in2

      fy   =   60   ksi

total depth of int. abut. (no haunch) = 119.75   in

      ds   =   total depth of int. abut. (no haunch) − bottom cover − ½ bar diameter

             =   119.75 −3− ½X1

             =   116.25 in

      a   =   Asfy/0.85f'cb   (S5.7.3.1.1-4)

             =   (3.16X60)/(0.85X3X(3X12))

             =   2.07   in.

      Mn   =   (3.16X60)X(116.25 − 2.07/2)/12

             =   1820   k-ft

Therefore,

      Mr   =   0.9X1820

             =   1638   k-ft   >   498.2   OK

Negative moment over the piles is taken equal to the positive moment. Use the same reinforcement at the top of the abutment beam as determined for the bottom (4 #8 bars).

By inspection:

- Mr > 4/3(Mu).

- The depth of the compression block is small relative to the section effective depth.

Shear design for CASE B

Assume the girder reaction is adjacent to the pile.

The maximum factored shear due to all applied loading:

$$V_u \quad = \quad P_u + w_u l/2$$
$$= \quad 323 + 10.73 \times 6.917/2$$
$$= \quad 360.1 \text{ K}$$

The factored shear resistance, Vr, is calculated as:

$$V_r \quad = \quad \varphi V_n \qquad\qquad\qquad (S5.8.2.1-2)$$

The nominal shear resistance, Vn, is calculated according to S5.8.3.3 and is the lesser of:

$$V_n \quad = \quad V_c + V_s \qquad\qquad\qquad (S5.8.3.3-1)$$
Or,
$$V_n = \quad 0.25 \times f'c \times b_v \times d_v \qquad\qquad (S5.8.3.3-2)$$

where:

$V_c \quad = 0.0316 \qquad \beta \times \sqrt{f'c} \times b_v \times d_v \qquad (S5.8.3.3-3)$
$\beta = \quad 2$
$f'c = \quad 3 \qquad$ ksi
$b_v = \quad 36 \qquad$ in.
$d_e = \quad 116.25$ in. (calculated earlier)
$d_v = \quad d_e - a/2$
$d_v = \quad 116.25 - (2.07/2) \qquad = \qquad 115.2 \quad$ in.

$0.9 \times d_e = \quad 0.9 \times 116.25 \qquad = \qquad 104.6 \quad$ in.

$0.72 \times h = \quad 0.72 \times 119.75 \qquad = \qquad 86.22 \quad$ in.

Therefore, dv should be taken as 115.2   in.

The nominal shear resistance, Vn, is taken as the lesser of:

$V_c \quad = \quad 0.0316 \times 2 \times (\sqrt{3}) \times 36 \times 115.2$
$\qquad = \qquad 454 \qquad$ K

Notice that Vc is large enough, relative to the applied load, that the contribution of the transverse shear reinforcement, Vs, is not needed.

Or,

Vn	=	0.25x3x36x115.2
	=	3110.4 K

Therefore, use the shear resistance due to the concrete, Vc

Vr	=	φVn			
	=	0.9x454			
	=	408.6 K	>	360.1	OK

Typical reinforcement details of the abutment beam are shown in Figures 7.1-4 through 7.1-7. Notice that bar shapes vary depending on the presence of girders and/or piles at the section.
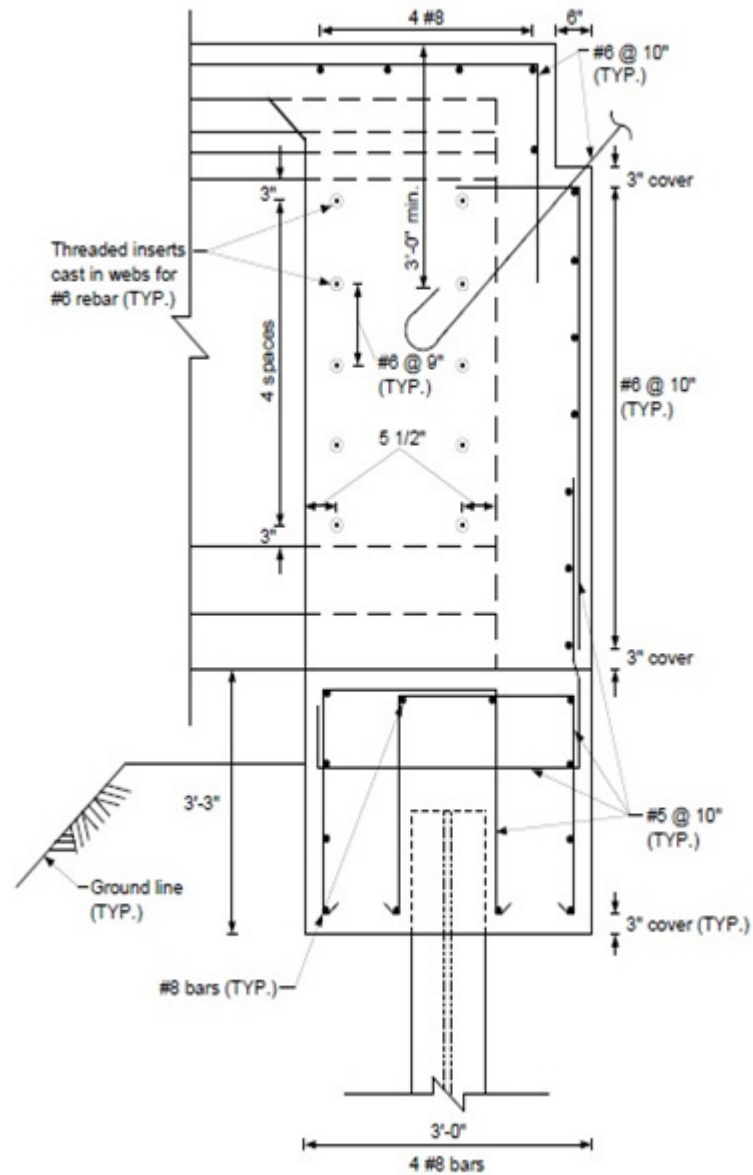
Figure 5.4 (AASHTO-LRFD Figure 7.1-4) – Integral Abutment Reinforcement, Girder and Pile exist at the Same Section
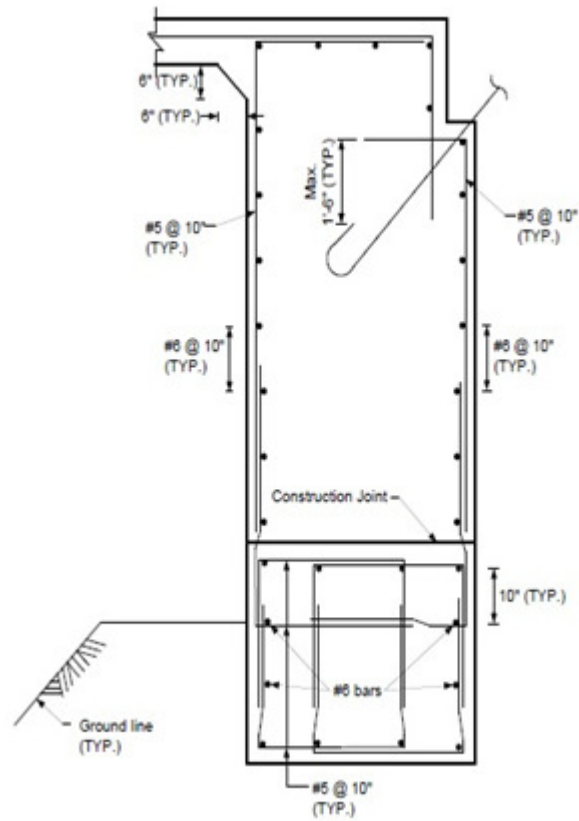
Figure 5.5 (AASHTO-LRFD Figure 7.1-5) – Integral Abutment Reinforcement, No Girder and No Pile at the Section
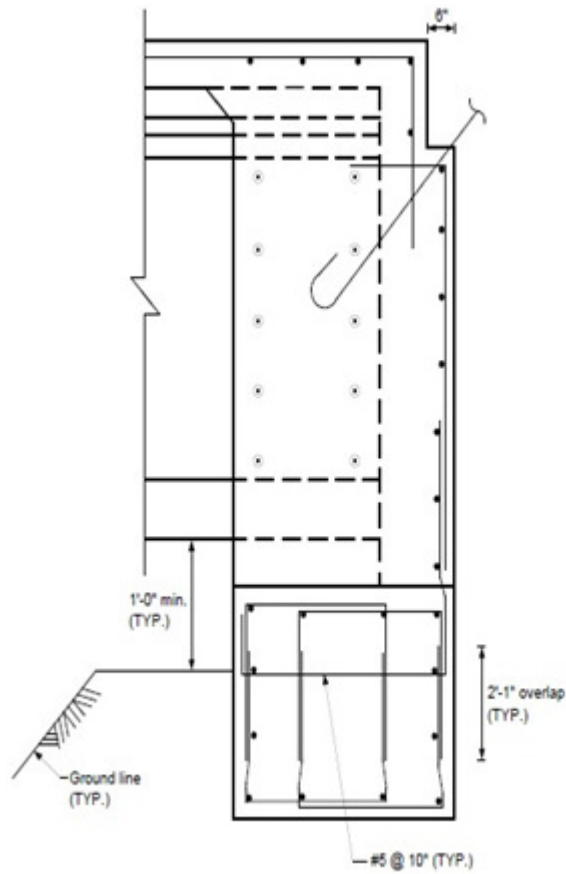
Figure 5.6 (AASHTO-LRFD Figure 7.1-6) – Integral Abutment Reinforcement, Girder, No Pile at the Section
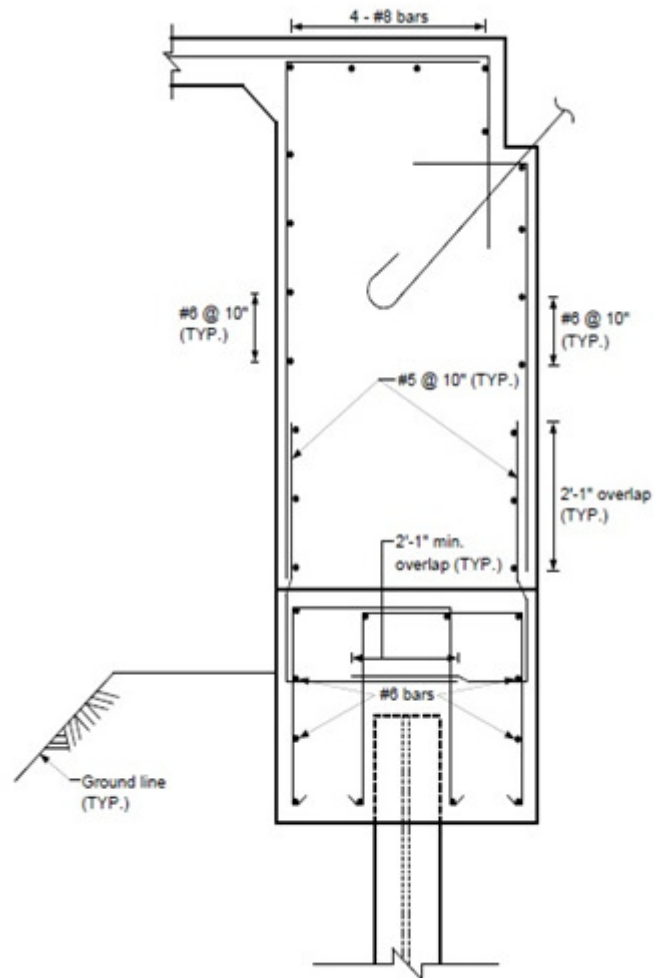
Figure 5.7 (AASHTO-LRFD Figure 7.1-7) – Integral Abutment Reinforcement, Pile without Girder
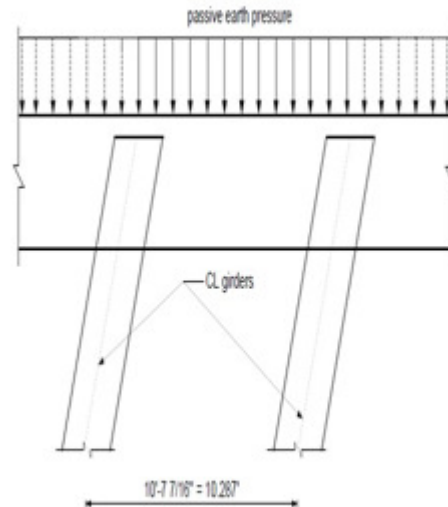
Figure 5.8 (AASHTO-LRFD Figure 7.1-8) – Passive Earth Pressure Applied to Backwall

Calculate the adequacy of the Backwall to resist passive pressure due to the abutment backfill material.

Passive earth pressure coefficient, kp = (1 + sin φ)/(1 − sin φ)
(Notice that kp may also be obtained from Figure S3.11.5.4-1)

wp = ½ γz2kp   (S3.11.5.1-1)

where:

wp	=	passive earth pressure per unit length of Backwall (k/ft)
γ	=	unit weight of soil bearing on the Backwall (kcf)
	=	0.13 kcf
Z	=	height of the Backwall from the bottom of the approach slab to the bottom of the pile cap (ft.)
	=	slab + haunch + girder depth + bearing pad thickness + pile cap depth − approach slab thickness
	=	(8/12) + (4/12) +6 + (0.75/12) +3.25 − 1.5
	=	8.81 ft.
φ	=	internal friction of backfill soil assumed to be 30°
wp	=	½ X(0.13)X(8.81)^2[(1 + sin 30)/(1 − sin 30)]
	=	15.1 k/ft of wall

Detail Design of Bridge Abutment in AASHTO-LRFD

Notice that developing full passive earth pressure requires relatively large displacement of the structure (0.01 to 0.04 of the height of the structure for cohesionless fill). The expected displacement of the abutment is typically less than that required to develop full passive pressure.

However, these calculations are typically not critical since using full passive pressure is not expected to place high demand on the structure or cause congestion of reinforcement.

No load factor for passive earth pressure is specified in the LRFD specifications. Assume the load factor is equal to that of the active earth pressure (φ = 1.5).

$$
\begin{aligned}
\varphi &= 1.5 \\
wu &= \varphi EHwp \\
&= 1.5 \times 15.1 \\
&= 22.65 \quad \text{k/ft of wall}
\end{aligned}
$$

The Backwall acts as a continuous horizontal beam supported on the girders, i.e., with spans equal to the girder spacing along the skew.

$$
\begin{aligned}
Mu &\cong wul2/8 \\
&= 22.65 \times (9.667/\cos 20) \times 2/8 \\
&= 300 \quad \text{k-ft/ft}
\end{aligned}
$$

Calculate the nominal flexural resistance, Mr, of the backwall.

$$Mr = \varphi Mn \qquad\qquad \text{(S5.7.3.2.1-1)}$$
and
$$Mn = Asfy(ds - a/2) \qquad\qquad \text{(S5.7.3.2.2-1)}$$

where:
As = area of the longitudinal reinforcement bars at front face (tension side) of the abutment (9 #6 bars)

   = 9X0.44   =   3.96   in2

Fy =   60   ksi

Width of Backwall	=	3	ft
Concrete Cover	=	3	ft
Vertical Bar Dia	=	0.625	ft
Bar Dia.	=	0.75	ft

ds   =   width of backwall – concrete cover – vertical bar dia. – ½ bar dia.
        =   (3X12) –3 – 0.625 – ½ (0.75)
        =   32   in

A   =   As x fy/0.85 x f'c x b   (S5.7.3.1.1-4)

where "b" is the height of the component

        =   (3.96X60)/(0.85X3X119.75)
        =   0.78   in

Mn   =   (3.96X60)X(32 – 0.78/2)/12
        =   626   k-ft/ft

Therefore, the factored flexural resistance, where φ = 0.9 for flexure (S5.5.4.2.1), is taken as:

Mr   =   0.9X626
        =   563   k-ft/ft >   300   k-ft/ft   OK

By inspection:
- Mr > 4/3(Mu).
- The depth of the compression block is small relative to the depth.

Check shear for the section of backwall between girders:

Vu   =   Pul/2   S = 9.667

        = 22.65x(9.667/sin 20)/2
        = 320.0945241

        116.5   k/ft

The factored shear resistance, Vr, is calculated as:

Vr = φVn   (S5.8.2.1-2)

The nominal shear resistance, Vn, is calculated according to S5.8.3.3 and is the lesser of:

Vn = Vc + Vs   (S5.8.3.3-1)
Or,

Vn = 0.25 x f'c x bv x dv   (S5.8.3.3-2)

where:

Vc	=	$0.0316 \times \beta \times \sqrt{f'c} \times bv \times dv$	(S5.8.3.3-3)
β	=	2	
f'c	=	3 ksi	
bv	=	effective horizontal beam width taken as the abutment depth (in.)	
	=	119.75 in	
dv	=	de – a/2	
	=	32 – (0.78/2)	
	=	31.61 in	

0.9 x de =	0.9X32	
=	28.8	in

0.72h =	0.72X36	
=	25.92	in

Therefore, dv should be taken as 31.61 in.

Ignore the contribution of the transverse reinforcement to the shear resistance (i.e., Vs = 0), Vn is taken as the smaller of:

Vc	=	0.0316X2X(√3)X119.75X31.61
	=	414.4 k/ft
Or,		
Vn	=	0.25 x3 x119.75 x 31.61
	=	2839 k/ft

Therefore, use the shear resistance due to the concrete, Vc

Vr =	φVn				
=	0.9 x414.4				
=	373	k/ft	>	116.5 k/ft	OK

5.1.1.24 AASHTO-LRFD Design Step 7.1.5 Wing Wall Design

There is no widely accepted method of determining design loads for the Wingwalls of integral abutments. The following design procedure will result in a conservative design as it takes into account maximum possible loads.

Two load cases are considered:

Load Case 1:
The Wingwall is subjected to passive earth pressure. This case accounts for the possibility of the bridge moving laterally and pushing the Wingwall against the fill. It is not likely thatthe displacement will be sufficient to develop full passive pressure. However, there is no available method to determine the expected pressure with certainty. This load case isconsidered under strength limit state.

Load Case 2:
The wingwall is subjected to active pressure and collision load on the parapet. Active pressure was considered instead of passive to account for the low probability that a collision load and passive pressure will exist simultaneously. This load case is considered at the extreme event limit state, i.e. φ = 1.0 (Table S3.4.1-1)

Required information:

Angle of internal friction of fill,   φ = 30 degrees

Coefficient of active earth pressure, ka = (1 – sin φ)/(1 + sin φ) = 0.33

Coefficient of passive earth pressure, kp = [(1 + sin φ)/(1 – sin φ)] / 3
$$ka/kp = 0.33/3 = 0.11$$

Load Case 1

From Figure 7.1-9 and utilizing properties of a right angle pyramid
[volume =   (1/3) x (base area)(height) and the center of gravity (applied at a distance measured from the vertical leg of the right angle pyramid)
        = ¼ base length].

Detail Design of Bridge Abutment in AASHTO-LRFD

Moment at the critical section for moment under passive pressure:
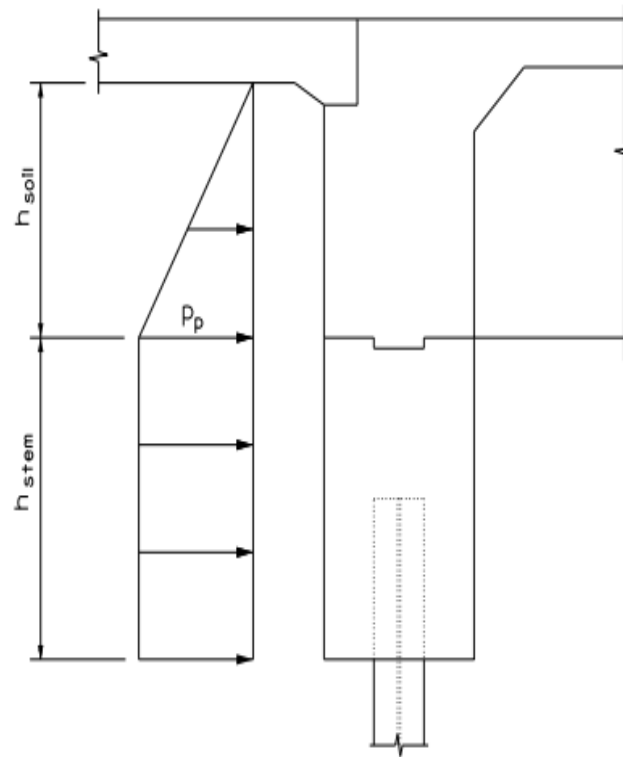
Figure 5.9 (AASHTO-LRFD Figure 7.1-9) – Critical Section for Moment under Passive Pressure

b1 =   14   ft
b2 =   1.6875 ft
b3 =   2   k/ft2
b4 =   3.24   ft
b5 =   3.44   k/ft2
b6 =   0.2   ft
d1 =   1.5   ft
d2 =   0.5   ft
d3 =   8.31   ft

Mp
=   [0.2X14X0.5X(14/2)] +[0.2X14X(8.31/2)X(14/3)] +[(1/3)X3.24X8.31X(14/2)X(14/4)]
=   284   k-ft

Minimum required factored flexural resistance, Mr = 284 k-ft.

Mr   =   φMn   (S5.7.3.2.1-1)

        where:
Mn   =   nominal resistance (k-ft)
          =   Mp
Φ   =   0.9   for flexure at the strength limit state   (S5.5.4.2)

Min. required Mn =   284/0.9
          =   316   k-ft

<u>Load Case 2</u>

Moment on the critical section for moment under active pressure:

Ma   =   0.11X284
          =   31.2   k-ft

Moment from collision load on the parapet:

From SA13.2 for Test Level 5, the crash load on the parapet is equal to 124 kips and is applied over a length of 8 ft.

Load on the parapet   =   124   kips
Applied length   =   8   ft

Maximum collision moment on the critical section:

M   =   124X(14 – 8/2)
          =   1240   k-ft

Total moment for Load Case 2,
M_{total} =   1240   +   31.2   =   1271.2 k-ft

The minimum required factored flexural resistance,
Mr   =   1271.2 k-ft
Mr   =   φMn   (S5.7.3.2.1-1)
where: φ   =   1   for flexure at the extreme event limit state

Detail Design of Bridge Abutment in AASHTO-LRFD

Min. required

> Mn   =   1271.2/1
>   =   1271.2 k-ft

From the two cases of loading:

Mn required =   1,271.20   k-ft

<u>Develop a section that provides the minimum nominal flexural resistance</u>

<u>Required information:</u>

Assuming reinforcement of #8 @ 6 in.
Number of bars within the 10.3125 ft. height of the wing wall =   22   bars
Parapet thickness at base   = 20.25   in
Section thickness   = Parapet thickness at base = 20.25 in.
Concrete cover   = 3   in.

The nominal flexural resistance, Mn, is taken as:
Mn   = As x fy x (ds – a/2)   (S5.7.3.2.2-1)

where:
ds   =   section thickness – cover – ½ bar diameter
       =   20.25 – 3 – ½ x1   =   16.75   in.

bar dia. =   1   in
bar area =   0.79   in2

As   =   22x0.79   =   17.38 in2

fy =   60
f'c =   3

a =   Asfy/0.85f'cb   (S5.7.3.1.1-4)
  =   17.38 x60/(0.85 x3 x123.75)
  =   3.3   in.

Mn =   Asfy(ds – a/2)
   =   17.38 x 60 x(16.75 – 3.3/2)/12
   =   1312   k-ft   >   1,271.20   OK

Secondary reinforcement of the wingwall is not by design, it is only meant for shrinkage. Use #6 @ 12 in. spacing as shown in Figure 7.1-10.
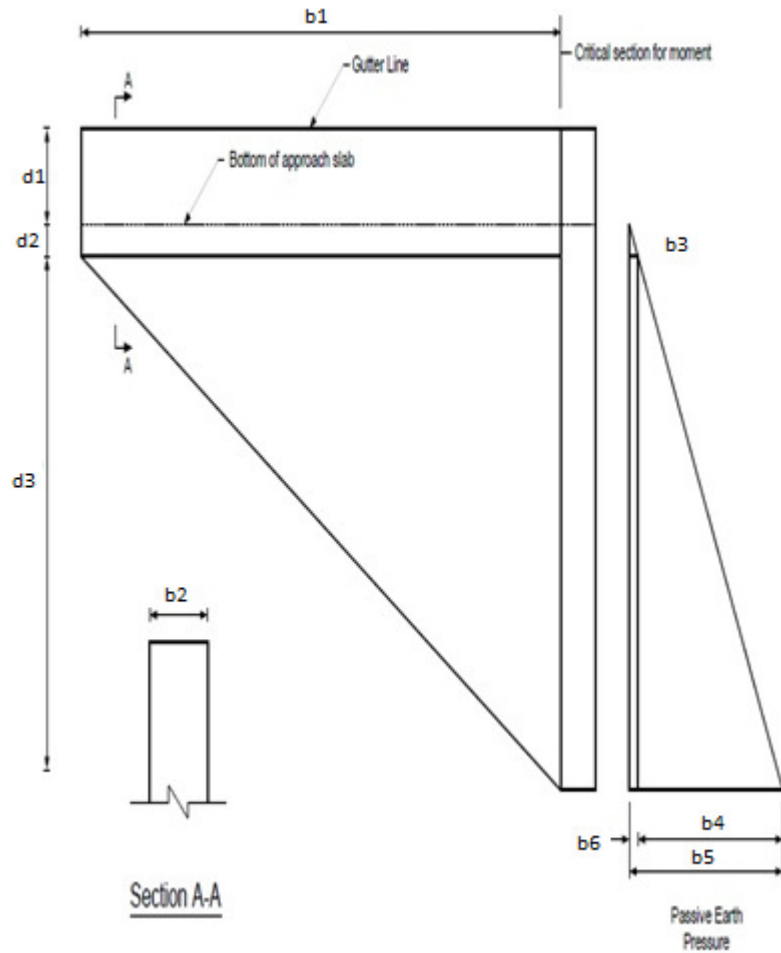
Figure 5.10 (AASHTO-LRFD Figure 7.1-10) – Wingwall Dimensions

b1 =   14   ft
b2 =   1.6875 ft
b3 =   2   k/ft2
b4 =   3.24   ft
b5 =   3.44   k/ft2
b6 =   0.2   ft
d1 =   1.5   ft
d2 =   0.5   ft
d3 =   8.3125 ft
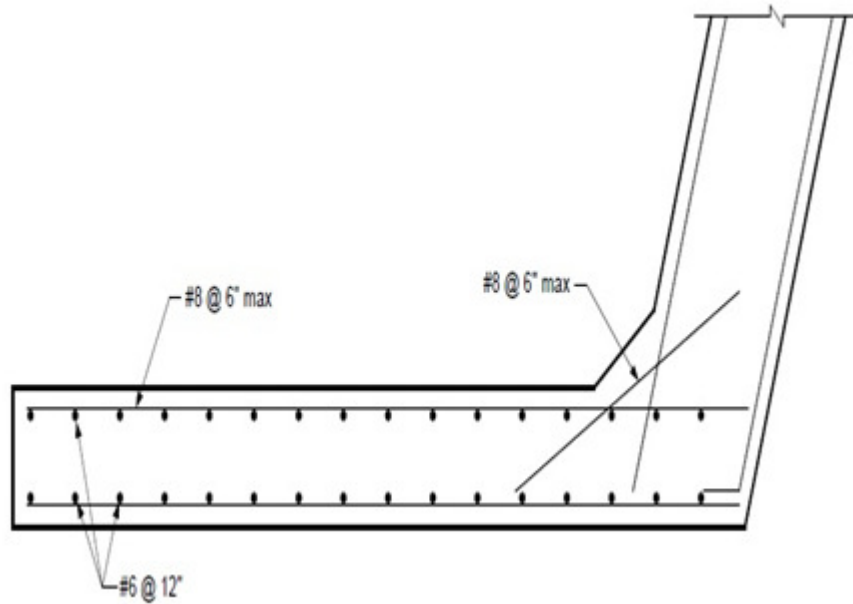
Figure 5.11 (AASHTO-LRFD Figure 7.1-11) – Wingwall Reinforcement

5.1.1.25 AASHTO-LRFD Design Step 7.1.6 Design of approach slab

Approach slab loading for a 1 ft. wide strip:

W_{self} =   0.15 x 1.5
        =   0.225   k/ft
W_{FWS} =   0.025   k/ft

Factored distributed dead loading:

$W_{Str\,I}$ =   (1.25x0.225) +(1.5x0.025)
      =   0.32   k/ft

<u>Live load distribution width (S4.6.2.3)</u>

The equivalent strip width of longitudinal strips per lane for both shear and moment is calculated according to the provisions of S4.6.2.3.

- For single lane loaded
  $E = 10 + 5\sqrt{L1W1}$ (S4.6.2.3-1)

- For multiple lanes loaded
  $E = 84.0 + 1.44\sqrt{L1W2} \leq 12.0W/NL$ (S4.6.2.3-2)

Where:
E = equivalent width (in.)
L1 = modified span length taken equal to the lesser of the actual span or 60.0 ft. (ft.)
W1 = modified edge-to-edge width of bridge taken to be equal to the lesser of the actual width

or   60.0 ft. for multilane loading,
or   30.0 ft. for single-lane loading
W = physical edge-to-edge width of bridge (ft.)

NL = number of design lanes as specified in S3.6.1.1.1

L1 =   25   ft
W1 =   30   ft
Esingle =   10+5X√(25X30)   =   146.9 in.

L1   =   25   ft
W2   =   55.34 ft
Emult. =   84 +1.44X√(25X55.34)=   137.6 in.

(12X55.34)/ 4   =   166.02 in

Therefore, the equivalent strip width is:

E =   137.6 in.

Live load maximum moment:

Lane load: max moment = 0.64x25^2/8 = 50 k-ft

Truck load: max moment = 207.4 k-ft (from live load analysis output for a 25 ft. simple span)

Total LL + IM   = 50+ 1.33 x207.4
                = 25.8   k-ft

Total LL + IM moment per unit width of slab
                = 325.8/(137.6/12)
                = 28.4 k-ft/ft

Maximum factored positive moment per unit width of slab due to dead load plus live load:

Mu   =   wl2/8 + 1.75(LL + IM moment)
        =   0.32 x (25^2)/8 + (1.75 x28.4)
        =   74.7   k-ft

The factored flexural resistance, Mr, is taken as:

Mr   = φMn   (S5.7.3.2.1-1)
and
Mn   = Asfy(d – a/2)   (S5.7.3.2.2-1)

where:

As   =   use #9 bars at   9   in. spacing

      =   1 x (12/9)

      =   1.33   in^2 per one foot of slab

fy   =   60   ksi

Slab depth   =   1.5   ft

Cover (cast against soil)   =   3   in

Bar diameter   =   1.128 in

d   =   slab depth – cover (cast against soil) – ½ bar diameter

     =   1.5 x 12 – 3 – ½ (1.128)

     =   14.4   in.

a   =   Asfy/0.85f'cb   (S5.7.3.1.1-4)

     =   (1.33 x 60)/[0.85X(3 x 12)]

     =   2.61   in.

Mn   =   1.33 x 60 x (14.4 – 2.61/2)/12

      =   87.1   k-ft

Therefore,

Mr   =   0.9X87.1

      =   78.4   k-ft   >   74.7   k-ft   OK

5.1.1.26 AASHTO-LRFD Design Step 7.1.6.1 - Bottom distribution reinforcement

Bottom distribution reinforcement (S9.7.3.2)

For main reinforcement parallel to traffic, the minimum distribution reinforcement is taken as a percentage of the main reinforcement:

$100/\sqrt{25} = 20\%$

Main reinforcement: #9 @ 9 in.   = 1x(12/9) = 1.33   in2/ft

Required distribution reinforcement = 0.2x1.33 = 0.27   in2/ft

Use #6 @ 12 in. = 0.44 in2/ft >   0.27   in2/ft   required reinforcement OK
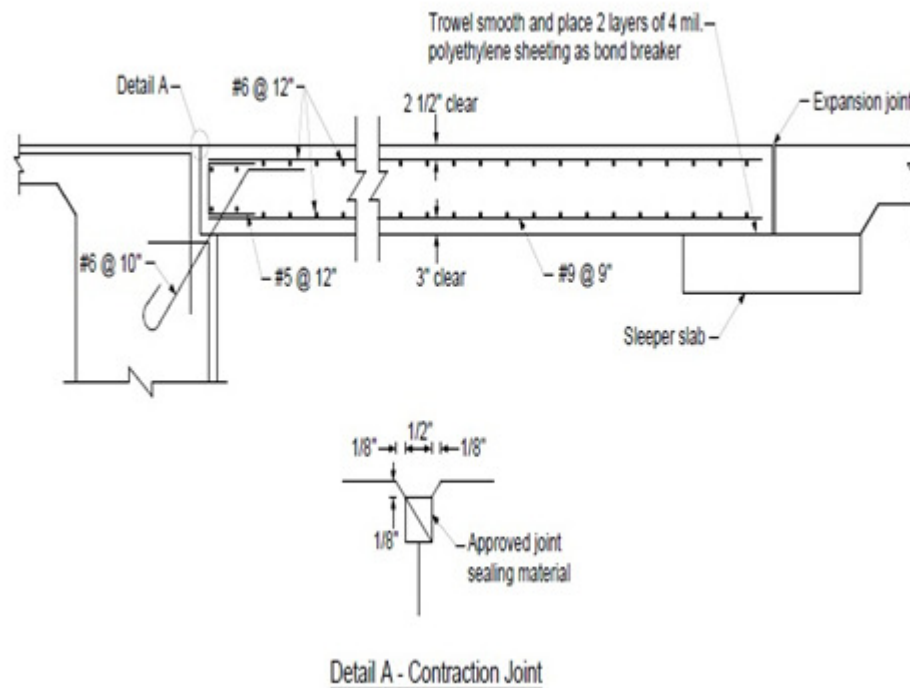
Figure 5.12 (AASHTO-LRFD Figure 7.1-12) – Typical Approach Slab Reinforcement Details

5.1.1.27 AASHTO-LRFD Design Step 7.1.7 Sleeper slab

No design provisions are available for sleeper slabs. The reinforcement is typically shown as a standard detail. If desired, moment in the sleeper slab may be determined assuming the wheel load is applied at the midpoint of a length assumed to bridge over settled fill, say a 5 ft. span length.
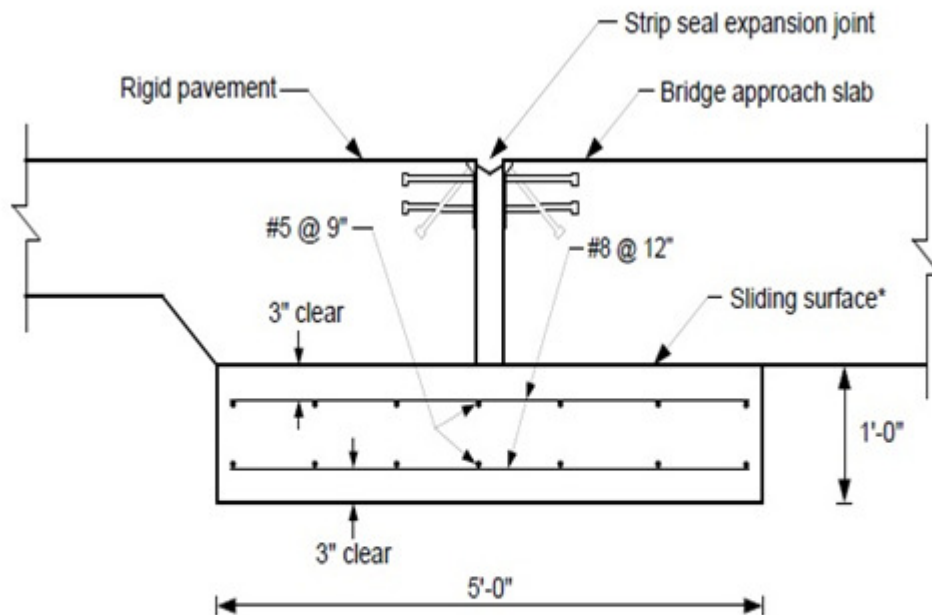
Figure 5.13 (AASHTO-LRFD Figure 7.1-13) – Sleeper Slab Details Used by the Pennsylvania Department of Transportation

5.2 Bearing Design

5.2.1 AASHTO-LRFD Design Step 6.1 - Obtain Design Criteria

For this bearing design example, an abutment bearing was chosen. It was decided that the abutment would have expansion bearings. Therefore, the bearing design will be for an expansion bearing.

Refer to Design Step 1 for introductory information about this design example. Additional information is presented about the design assumptions, methodology, and criteria for the entire bridge, including the bearing design.

The following units are defined for use in this design example:

$$K = 1000lb \qquad ksi = \frac{K}{in^2}$$

For bearing design, the required design criteria includes:

1. Longitudinal and transverse movement
2. Longitudinal, transverse, and vertical rotation
3. Longitudinal, transverse, and vertical loads

Most of the above information is typically obtained from the superstructure design software output, which is the case for this bearing design (first trial of girder design):

DLserv =	78.4	K	Service I limit state dead load
LLserv =	110.4	K	Service I limit state live load (including dynamic load allowance)
θsx =	0.0121	rad	Service I limit state total rotation about the transverse axis (see Figure 6-1)
Psd =	67.8	K	Strength limit state minimum vertical force due to permanent loads (used in Design Step 6.12)

5.2.2 AASHTO-LRFD Design Step 6.2 - Select Optimum Bearing Type

Selecting the optimum bearing type depends on the load, movement capabilities, and economics. Refer to STable 14.6.2-1 and SFigure 14.6.2-1 for guidance on selecting the most practical bearing type. For the abutment bearing design, a steel-reinforced elastomeric bearing was selected. If the loads were considerably larger, pot bearings, which are more expensive than elastomeric bearings, would be an option.

5.2.3 AASHTO-LRFD Design Step 6.3 - Select Preliminary Bearing Properties

Once the most practical bearing type has been selected, the preliminary bearing properties must be defined. The bearing properties are obtained from the Specifications, as well as from past experience. The following preliminary bearing properties were selected:

Bearing Pad Configuration

Pad length (bridge longitudinal direction):	Lpad = 14	in	
Pad width (bridge transverse direction):	Wpad =	15	in
Elastomer cover thickness:	hrcover =	0.25	in
Elastomer internal layer thickness:	hrinternal =	0.375	in
Number of steel reinforcement layers:	Nstlayers =	9	
Steel reinforcement thickness:	hreinf =	0.1196	in

Material Properties   S14.7.6.2 & S14.7.5.2

Elastomer hardness:	HshoreA =	50	
Elastomer shear modulus:	G =	0.095 ksi	STable 14.7.5.2-1

Elastomer creep deflection at 25 years
divided by the instantaneous deflection: Cd =   0.25   STable 14.7.5.2-1
Steel reinforcement yield strength: Fy =   50   ksi

5.2.4 AASHTO-LRFD Design Step 6.4 - Select Design Method (A or B) C14.7.5.1

For this design example, Method A will be used. Method A usually results in a bearing with a lower capacity than a bearing designed with Method B. However, Method B requires additional testing and quality control. Method A is described in S14.7.6, while Method B is described in S14.7.5.

5.2.5 AASHTO-LRFD Design Step 6.5 - Compute Shape Factor S14.7.6.1 & S14.7.5.1

The shape factor for individual elastomer layers is the plan area divided by the area of perimeter free to bulge.

For steel-reinforced elastomeric bearings, the following requirements must be met prior to calculating the shape factor:

1. All internal layers of elastomer must be the same thickness.

2. The thickness of the cover layers cannot exceed 70 percent of the thickness of the internal layers.

From Design Step 6.3, all internal elastomer layers are the same thickness, which satisfies Requirement 1. The following calculation verifies that Requirement 2 is satisfied:

$0.70 \cdot hrinternal$ =   0.26   in
hrcover   =   0.25   in   OK

For rectangular bearings without holes, the shape factor for the ith layer is: S14.7.5.1

$$S_i = \frac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$$

The shape factor for the cover layers is then:

$$S_{cov} = \frac{L_{pad} \cdot W_{pad}}{2 \cdot h_{rcover} \cdot (L_{pad} + W_{pad})}$$

Scov   =   14.48

The shape factor for the internal layers is then:

$$S_{int} = \frac{L_{pad} \cdot W_{pad}}{2 \cdot h_{rinternal} \cdot (L_{pad} + W_{pad})}$$

Sint   =   9.66

5.2.6 AASHTO-LRFD Design Step 6.6 - Check Compressive Stress S14.7.6.3.2

The compressive stress check limits the compressive stress in the elastomer at the service limit state as follows:

$$\sigma_s \leq 1.0 \text{ksi} \quad \text{and} \quad \sigma_s \leq 1.0 \cdot G \cdot S$$

The compressive stress is taken as the total reaction at one of the abutment bearings for the service limit state divided by the elastomeric pad plan area. The service limit state dead and live load reactions are obtained from the Opis superstructure output. The shape factor used in the above equation should be for the thickest elastomer layer.

Service I limit state dead load:

DLserv = 78.4 K

Service I limit state live load
(including dynamic load allowance):

LLserv = 110.4 K

$$\sigma_s = \frac{DL_{serv} + LL_{serv}}{\left(L_{pad} \cdot W_{pad} \right)}$$

σs   =   0.899 ksi

1.0·G·Sint   =   0.918 ksi   OK

The service average compressive stress due to live load only will also be computed at this time. It will be needed in Design Step 6.11. Again, the service limit state live load value was obtained from Opis superstructure output.

$$\sigma_L = \frac{LL_{serv}}{\left(L_{pad} \cdot W_{pad} \right)}$$

σL =   0.526 ksi

5.2.7 AASHTO-LRFD Design Step 6.7 - Check Compressive Deflection

The compressive deflection due to the total load at the service limit state is obtained from the following equation:

$$\delta = \Sigma \varepsilon_i \cdot h_{ri}$$

S14.7.5.3.3

For this design example, the instantaneous compressive strain was approximated from CTable 14.7.5.3.3-1 for 50 durometer reinforced bearings using a compressive stress of 0.899 ksi and a shape factor of 9.66

$$\varepsilon_{int} \quad = \quad 0.04 \qquad \text{CTable} \qquad 14.7.5.3.3\text{-}1$$

The instantaneous deflection is then:

$$\delta_{inst} = 2 \cdot \varepsilon_{int} \cdot h_{rcover} + 8 \cdot \varepsilon_{int} \cdot h_{rinternal}$$
$$\delta inst = 0.14 \qquad in$$

The effects of creep should also be considered. For this design example, material-specific data is not available. Therefore, calculate the creep deflection value as follows:

$$\delta_{creep} = C_d \cdot \delta_{inst}$$

STable 14.7.5.2-1

$$\delta creep = 0.035 \qquad in$$

The total deflection is then:

$$\delta_{total} = \delta_{inst} + \delta_{creep}$$
$$\delta total = 0.175$$

The initial compressive deflection in any layer of a steel-reinforced elastomeric bearing at the service limit state without dynamic load allowance shall not exceed $0.07 h_{ri}$.

S14.7.6.3.3

In order to reduce design steps, the above requirement will be checked using the deflection calculated for the service limit state including dynamic load allowance. If the compressive deflection is greater than $0.07 h_{ri}$, then the deflection without dynamic loadallowance would need to be calculated.

$$\delta int1layer \qquad = \quad 0.015 \quad in$$
$$0.07hrinternal \qquad = \quad 0.026 \qquad \qquad OK$$

5.2.8 AASHTO-LRFD Design Step 6.8 - Check Shear Deformation

The shear deformation is checked to ensure that the bearing is capable of allowing the anticipated horizontal bridge movement. Also, the shear deformation is limited in order to avoid rollover at the edges and de-lamination due to fatigue caused by cyclic expansion and contraction deformations. The horizontal movement for this bridge design example is based on thermal effects only. The thermal movement is taken from Design Step 7.6 for the controlling movement, which is contraction. Other criteria that could add to the shear deformation include construction tolerances, braking force, and longitudinal wind if applicable. One factor that can reduce the amount of shear deformation is the substructure deflection. Since the abutment height is relatively short and the shear deformation is relatively small, the abutment deflection will not be taken into account (S14.7.6.3.4 and C14.7.5.3.4).

The bearing must satisfy:

$$h_{rt} \geq 2 \cdot \Delta_s$$

$$h_{rt} = 2 \cdot h_{rcover} + 8 \cdot h_{rinternal}$$

$$h_{rt} = \quad 3.50 \quad in$$

$\Delta contr \qquad = \qquad 0.636 \quad in \qquad$ from Design Step 7.6 for thermal contraction

$\gamma_{TU} \qquad = \qquad 1.2 \qquad$ for the service limit state

$$\Delta_s = \gamma_{TU} \cdot \Delta_{contr}$$

$\Delta s \qquad = \qquad 0.76 \quad in$
$2 \times \Delta s \qquad = \qquad 1.52 \quad in$

$3.50 \quad in \qquad \geq \qquad 1.52 \quad in \qquad OK$

5.2.9 AASHTO-LRFD Design Step 6.9 - Check Rotation or Combined Compression and Rotation

Since Design Method A was chosen, combined compression and rotation does not need to be checked. The rotation check ensures that no point in the bearing undergoes net uplift and is as follows: S14.7.6.3.5

$$\sigma_s \geq 0.5 G \cdot S \cdot \left(\frac{L}{h_{ri}}\right)^2 \cdot \frac{\theta_{sx}}{n} \qquad \text{(associated with rotation about transverse axis)}$$

and

$$\sigma_s \geq 0.5 G \cdot S \cdot \left(\frac{W}{h_{ri}}\right)^2 \cdot \frac{\theta_{sz}}{n} \qquad \text{(associated with rotation about longitudinal axis)}$$

S14.7.6.3.5d

$$\sigma_s \quad = \quad 0.899 \ \text{ksi}$$

The service rotation due to the total load about the transverse axis was taken from Analysis:

$$\theta_{sx} \quad = \quad 0.0121 \ \text{rad}$$

Construction Tolerance

For spans over approximately 100 feet, it is good engineering practice to include an additional 0.005 radians of rotation about both pad axes to account for construction tolerances. The number of interior layers is:

$$n \quad = \quad 8 + 0.5 + 0.5$$
$$= \quad 9.00$$

$$0.5 \cdot G \cdot S_{int} \cdot \left(\frac{L_{pad}}{h_{rinternal}}\right)^2 \cdot$$
$$\frac{\theta_{sx}}{(8+1)}$$

$$= \quad 0.860 \ \text{ksi} \quad < \quad 0.899 \ \text{ksi} \quad \text{OK}$$

The service rotation due to the total load about the longitudinal axis is negligible compared to the service rotation about the transverse axis. Therefore, the check about the longitudinal axis will be assumed to be negligible and is not computed in this bearing design example.

5.2.10 AASHTO-LRFD Design Step 6.10 - Check Stability

The total thickness of the pad shall not exceed the least of L/3 or W/3.   S14.7.6.3.6

$L_{pad}/3$ = 4.67 in $W_{pad}/3$ = 5.00

The total thickness of the pad based on the preliminary dimensions is:

$$h_{total} = 2 \cdot h_{rcover} + 8 \cdot h_{rinternal} + N_{stlayers} \cdot h_{reinf}$$

htotal   =   4.5764 in   <   4.67   in   OK

              =   4.5764 in   <   5.00   in   OK

5.2.11 AASHTO-LRFD Design Step 6.11 - Check Reinforcement

The thickness of the steel reinforcement must be able to sustain the tensile stresses induced by compression in the bearing. The reinforcement thickness must also satisfy the requirements of the AASHTO LRFD Bridge Construction Specifications.   S14.7.6.3.7

For the service limit state:

$$h_s \geq \frac{3 h_{max} \cdot \sigma_s}{F_y}$$

$$h_{max} = h_{rinternal}$$

hmax = 0.375   in
σs =   0.899   ksi
Fy =   50   ksi

$$\frac{3 \cdot h_{max} \cdot \sigma_s}{F_y} = \quad 0.02023 \text{ in}$$

hreinf =   0.1196 in   OK

For the fatigue limit state:

$$h_s \geq \frac{2 h_{max} \cdot \sigma_L}{\Delta F_{TH}}$$

From Design Step 6.6, the service average compressive stress due to live load only is:

σL   =   0.526 ksi

ΔFTH   =   24   ksi

$$\frac{2 \cdot h_{max} \cdot \sigma_L}{\Delta F_{TH}} = \quad 0.0164 \text{ in}$$

0.0164 in   <   hreinf =   0.1196 in   OK

5.2.12 AASHTO-LRFD Design Step 6.12 - Design for Anchorage   175

The bearing pad must be secured against transverse horizontal movement if the factored shear force sustained by the deformed pad at the strength limit state exceeds one-fifth of the minimum vertical force due to permanent loads, Psd.   S14.7.6.4

Psd = 67.8   K

The maximum factored shear force sustained by the deformed pad at the strength limit state is obtained from Design Step 7.6, adding wind on superstructure and wind on live load. The maximum shear force will occur when wind is taken at 0 degrees.

The shear force due to wind on superstructure is taken from Table 7-1:

$$WS = 30.69 \ K$$

The shear force due to wind on live load is taken from Table 7-2:

$$WL = 6.00 \ K$$

The controlling shear force is either from Strength III or Strength V:

Factored shear force per bearing for Strength III:

$$\gamma_{WS} = 1.4$$
$$\gamma_{WL} = 0.00$$

$$V_{windstrIII} = \frac{\left(\gamma_{WS} \cdot WS + \gamma_{WL} \cdot WL\right)}{5}$$

$$V_{windstrIII} = 8.59 \ K$$

Factored shear force per bearing for Strength V:

$$\gamma_{WS} = 0.40$$
$$\gamma_{WL} = 1.00$$

$$V_{windstrV} = \frac{\left(\gamma_{WS} \cdot WS + \gamma_{WL} \cdot WL\right)}{5}$$

$$V_{windstrV} = 3.66 \ K$$

Use:

$$V_{max} = max\left(V_{windstrIII}, V_{windstrV}\right)$$

$$= \quad 8.59 \quad K$$

$$\frac{1}{5} \cdot P_{sd} = \quad 13.56 \text{ K}$$

Since the maximum shear force at the strength limit state does not exceed one-fifth of the minimum vertical force due to permanent dead loads, the pad does not need to be secured against horizontal movement.

5.2.13 AASHTO-LRFD Design Step 6.13 - Design Anchorage for Fixed Bearings

The abutment bearings are expansion in the longitudinal direction but fixed in the transverse direction. Therefore, the bearings must be restrained in the transverse direction. Based on Design Step 6.12, the expansion bearing pad does not need to be secured against horizontal movement.   S14.8.3.1

However, based on S3.10.9.2, the horizontal connection force in the restrained direction cannot be less than 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake. In addition, since all abutment bearings are restrained in the transverse direction, the tributary permanent load can be taken as the reaction at the bearing.   S3.10.9.2

Also, γ_{EQ} is assumed to be zero. Therefore, no tributary live loads will be considered. This transverse load will be used to design the bearing anchor bolts for this design example.
                      C3.4.1

For the controlling girder (interior):

$$DLserv = \quad 78.4 \quad K$$

The maximum transverse horizontal earthquake load per bearing is then:
                      S14.8.3.1 and S6.13.2.7

$$H_{EQ} = 0.1 \cdot DL_{serv}$$

$$HEQ = 7.84 \quad K$$

The factored shear resistance of the anchor bolts per bearing is then:

$$b_{dia} = \quad 0.625 \quad in$$

Assume two 0.625 inch diameter A 307 bolts with a minimum tensile S6.4.3 strength of 60 ksi: S6.13.2.7

for threads excluded from shear plane

$$R_n = 0.48 \cdot A_b \cdot F_{ub} \cdot N_s$$

$\phi s = \quad 0.65 \quad$ resistance factor for A 307 bolts in shear

$$Ab = \pi \cdot b_{dia}^2 / 4$$
$$= 0.31 \quad in^2$$

$$Fub = 60 \quad ksi$$

$$Ns = 2 \quad \text{(number of bolts)}$$

$$R_n = 0.48 \cdot A_b \cdot F_{ub} \cdot N_s$$

$$Rn = 17.86 \ K$$

$$Rr = \phi s \cdot Rn$$

$$11.61 \ K \quad \geq \quad HEQ = 7.84 \quad OK$$

Once the anchor bolt quantity and size are determined, the anchor bolt length must be computed. As an approximation, the bearing stress may be assumed to vary linearly from zero at the end of the embedded length to its maximum value at the top surface of the concrete. The bearing resistance of the concrete is based on S5.7.5.   S14.8.3.1

$$\phi_b \cdot P_n = \phi_b \cdot 0.85 \cdot f_c \cdot A_1 \cdot m \qquad\qquad \text{S5.7.5}$$

$$Stress_{brg} = \frac{\phi_b \cdot P_n}{A_1}$$

$$Stress_{brg} = \phi_b \cdot 0.85 \cdot f_c \cdot m$$

Assume: $m = 0.75$ (conservative assumption)

         $\phi b = 0.7$ for bearing on concrete   S5.5.4.2.1

$$Stress_{brg} = \phi b \cdot 0.85 \cdot (4ksi) \cdot m$$
$$1.785 \ ksi$$

The total transverse horizontal load is:

$$HEQ = 7.84 \quad K$$

The transverse load per anchor bolt is then:

$$P_{1bolt} = \frac{H_{EQ}}{2}$$

$$P_{1bolt} \quad = \quad 3.92 \quad K$$

Using the bearing stress approximation from above, the required anchor bolt area resisting the transverse horizontal load can be calculated.

$$A_1 = \frac{P_{1bolt}}{\left(\dfrac{Stress_{brg} + 0}{2}\right)}$$

$$A1 = \quad 4.39 \quad in2$$

A1 is the product of the anchor bolt diameter and the length the anchor bolt is embedded into the concrete pedestal/beam seat. Since we know the anchor bolt diameter, we can now solve for therequired embedment length.

$$L_{embed} \qquad = \qquad A1/bdia = \quad 7.02 \quad in$$

Individual states and agencies have their own minimum anchor bolt embedment lengths. For this design example, a minimum of 12 inches will be used.

$$\text{Use:} \quad L_{embed} = \quad \textcolor{red}{12} \quad in$$

5.2.14 AASHTO-LRFD Design Step 6.14 - Draw Schematic of Final Bearing Design
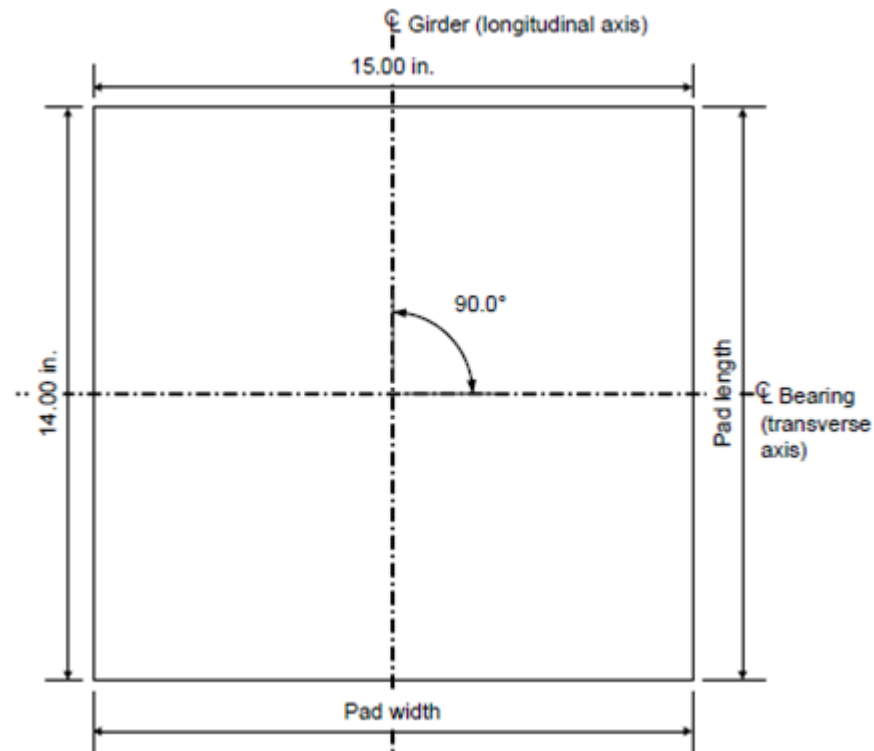
Figure 5.14 (AASHTO-LRFD Figure No. 6-1) - Bearing Pad Plan View
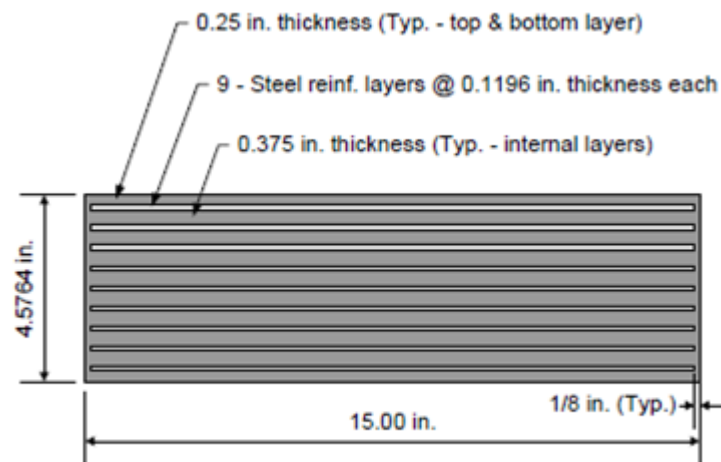
Figure 5.15 (AASHTO-LRFD Figure No. 6-2) - Bearing Pad Elevation View
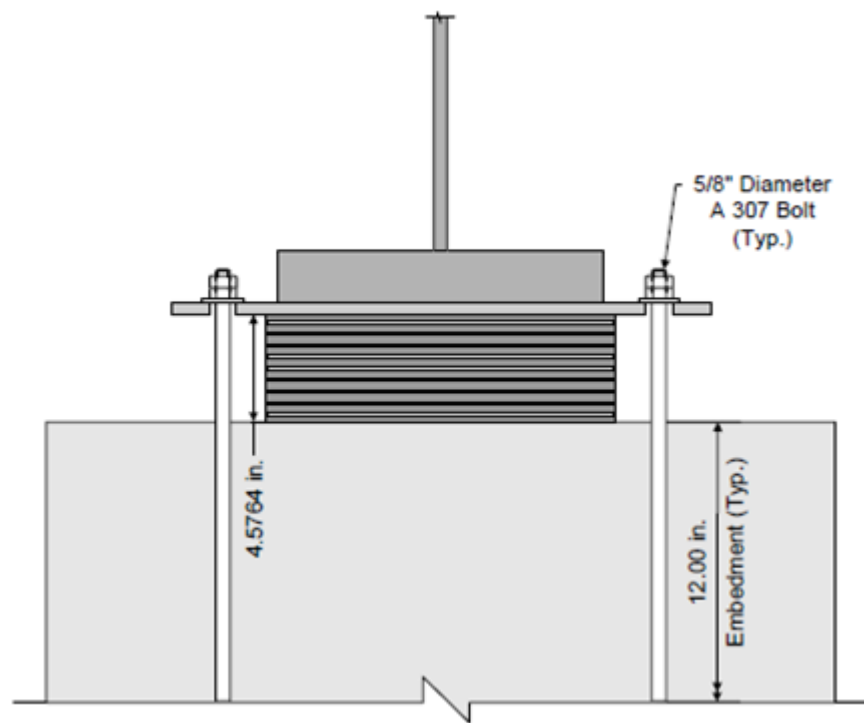
Figure 5.16 (AASHTO-LRFD Figure No. 6-3) – Anchor Bolt Embedment

5.3 Computer Applications for the Design of Bridge Abutment in AASHTO LRFD

This section is continuation of the Article '1 Grillage Analysis of Bridge Superstructure' previously submitted in ResearchGate.

We refer to the Final step (Step 34) of Section 1.7, which mentions the Support Reactions for the design of Abutments and Piers.

Here, we get the indicative Maximum Vertical Reaction and Bending Moments about X and Z axes. These forces may be taken for the design of 'Abutments' and 'Piers', but the designs will also calculate the forces in the design Excel Worksheet as per the actual loads and boundary conditions during the design process.

Step 1 Open the Main Screen of ASTRA Pro by double clicking on desktop icon,

Step 2 Select menu item File >> Select Working Folder,
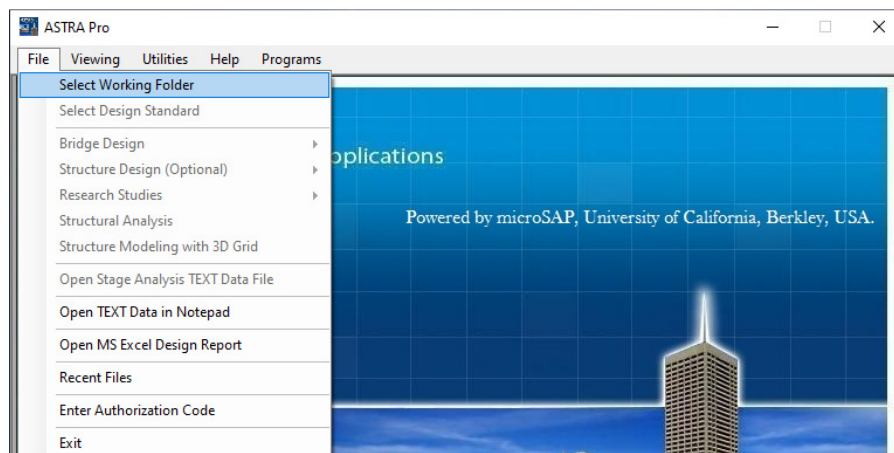
Step 3 The folder "Work" is selected (or may be created and selected) from desktop,
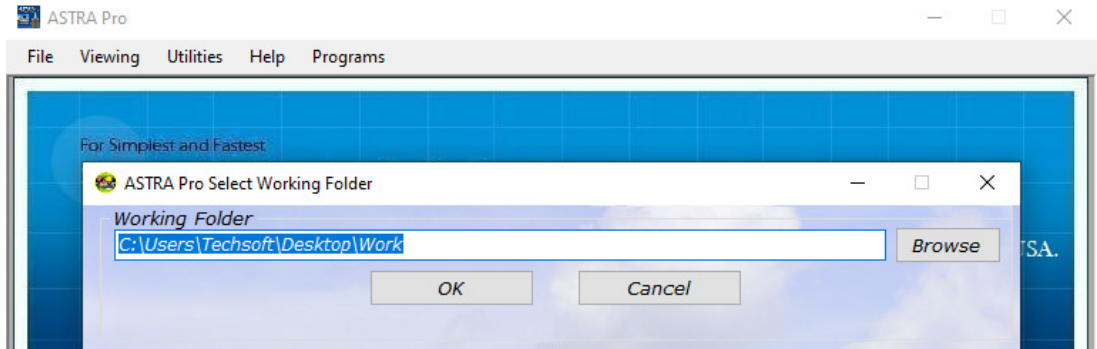
Step 4 Select menu item, File >> Bridge Design >> Pre Stressed Concrete (PSC) I-Girder Bridge >> Limit State Method
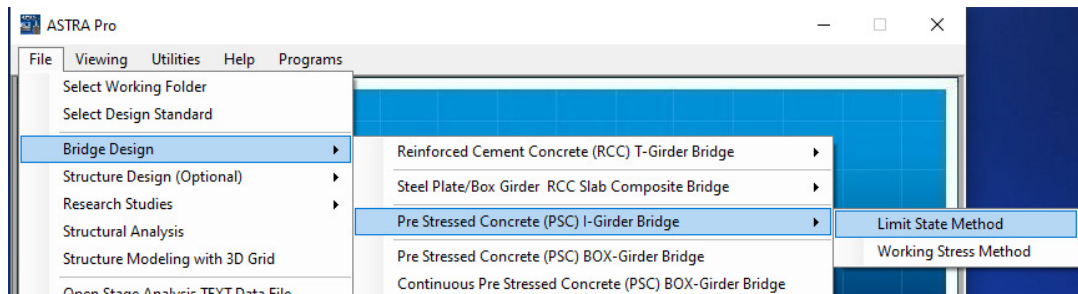
Step 5 Select Design Standard 'AASHTO – LRFD Standard',
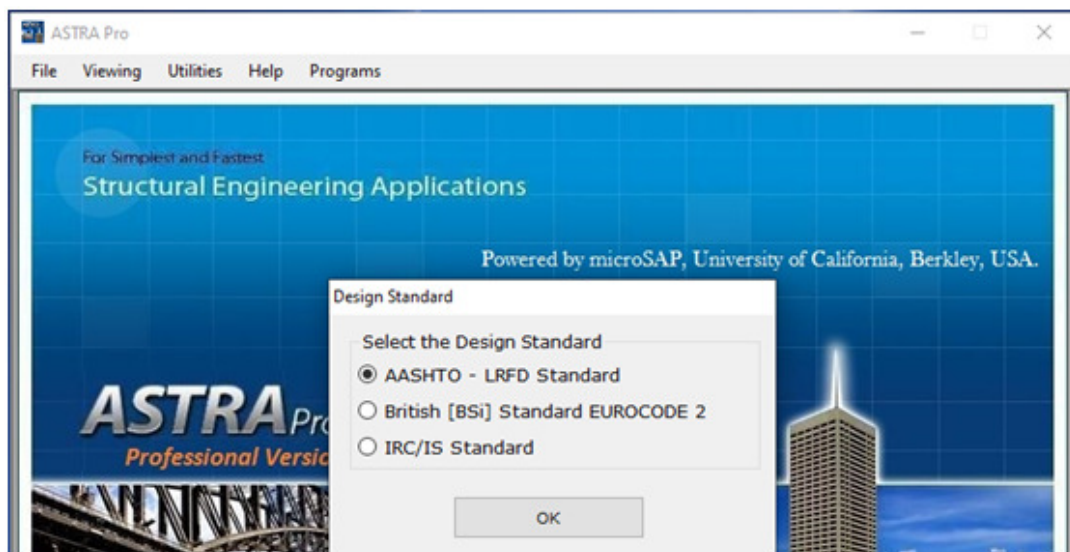
Step 6 **This session is continuation of last session described in section 1.3, of Chapter 1 for Grillage Analysis and was created as 'DESIGN JOB# 01', which is now opened here.**
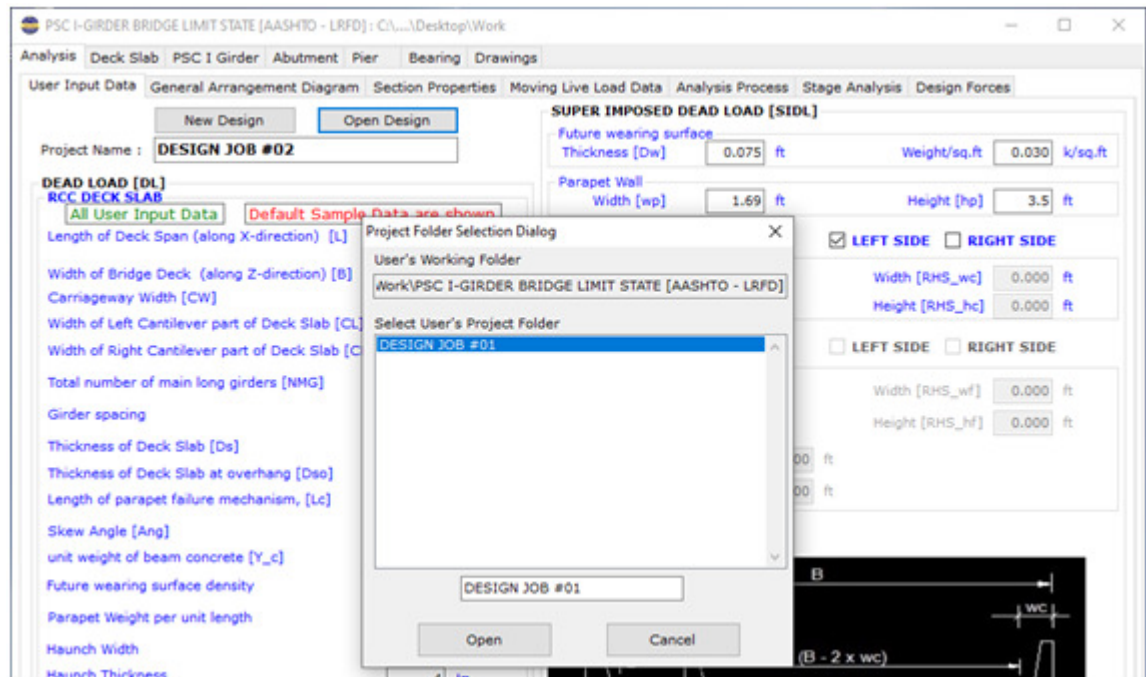
Step 7 **As the project 'DESIGN JOB #1' is opened, message comes, click on 'OK',**
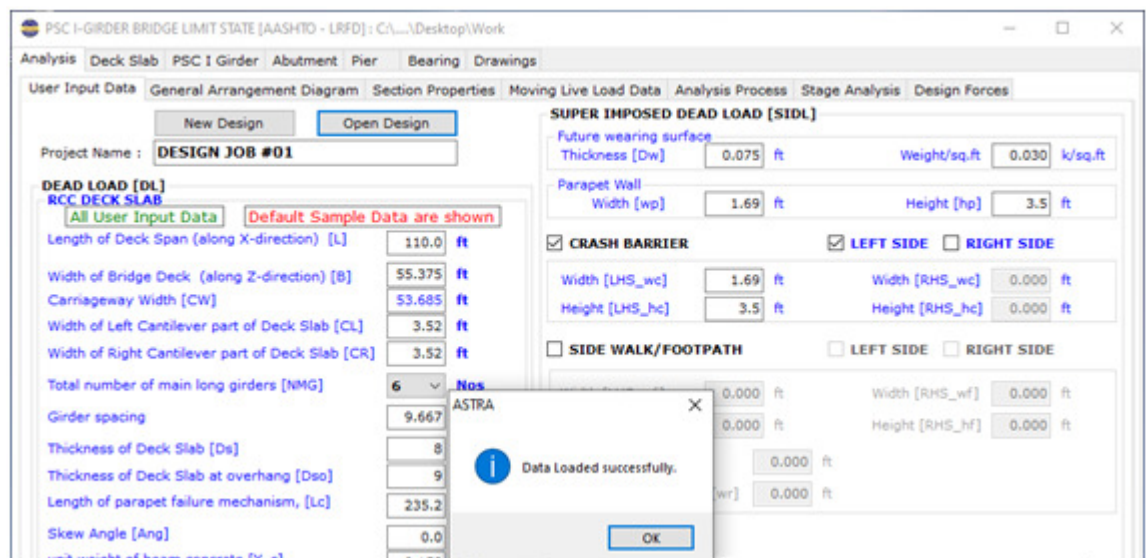
Detail Design of Bridge Abutment in AASHTO-LRFD

Step 8 Open tab page 'Design Forces', and select results of 'Normal analysis',
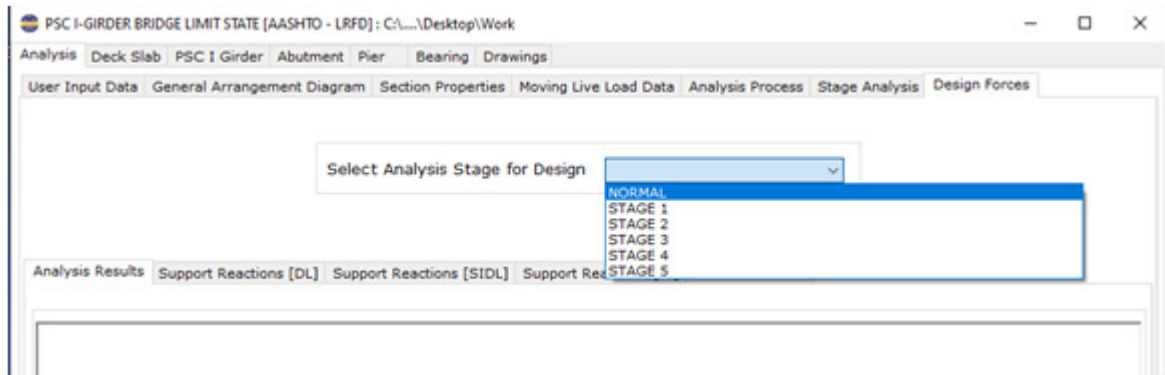
Step 9 The results of 'Normal Analysis' is selected and the tab page 'Design Forces' is selected, The Reaction Forces at supports are described as obtained from analysis.
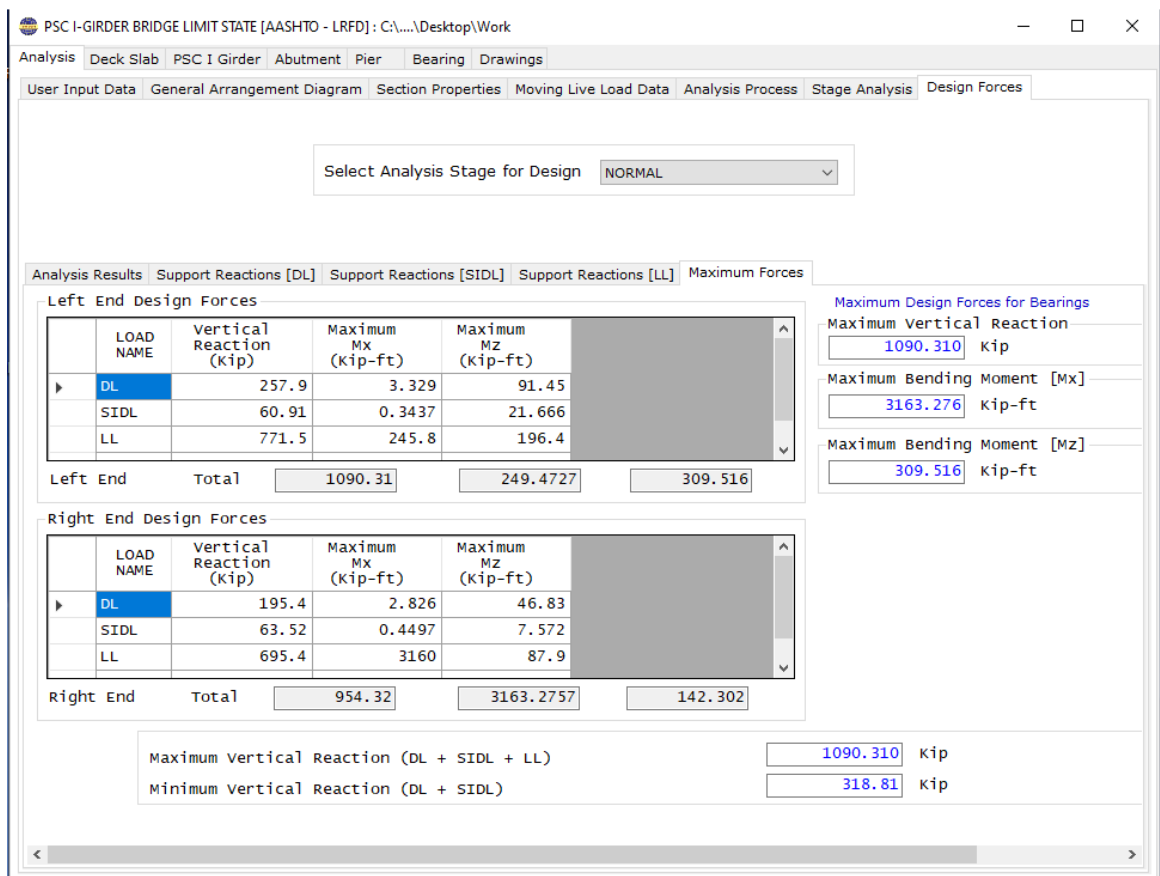
Step 10 Open tab page 'Abutment', change default data as required, click on button 'Process for New Design….', message comes, wait to get the 'Excel Design Worksheet',

Step 11 View the design details, various step wise pages are provided at the bottom. Some design data may be changed in the worksheet to modify the design as desired,

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Step 12 Save the Abutment design with a file name as 'Abutment Design with Pile Foundation in LSM (AASHTO-LRFD).xlsx' on the desktop and close the design worksheet,
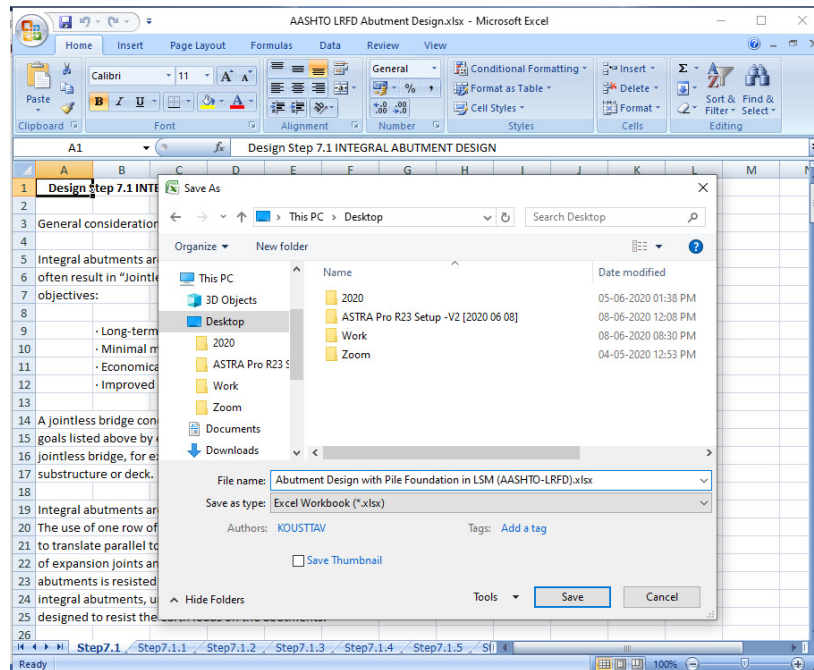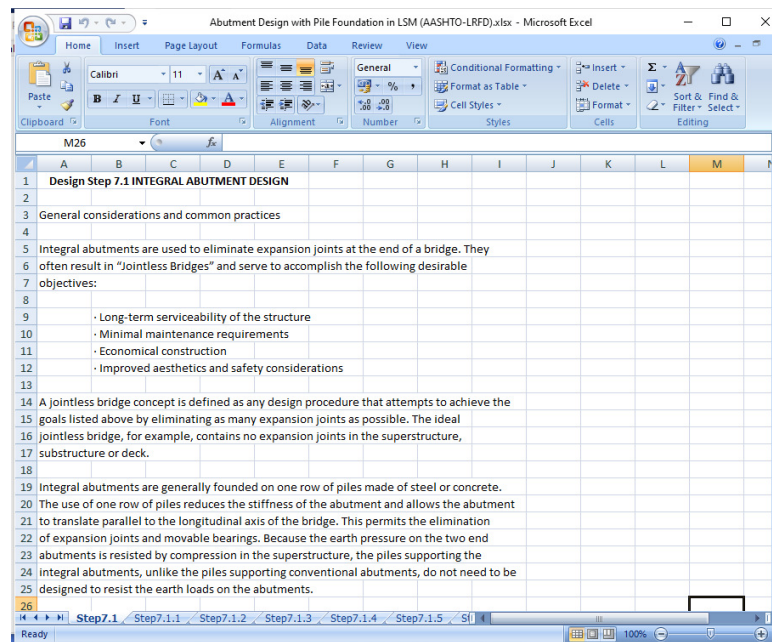

Step 13 The design worksheet file is password protected, to re-open the saved design worksheet, Click on button 'Open User's Design….' Select the saved file and open,
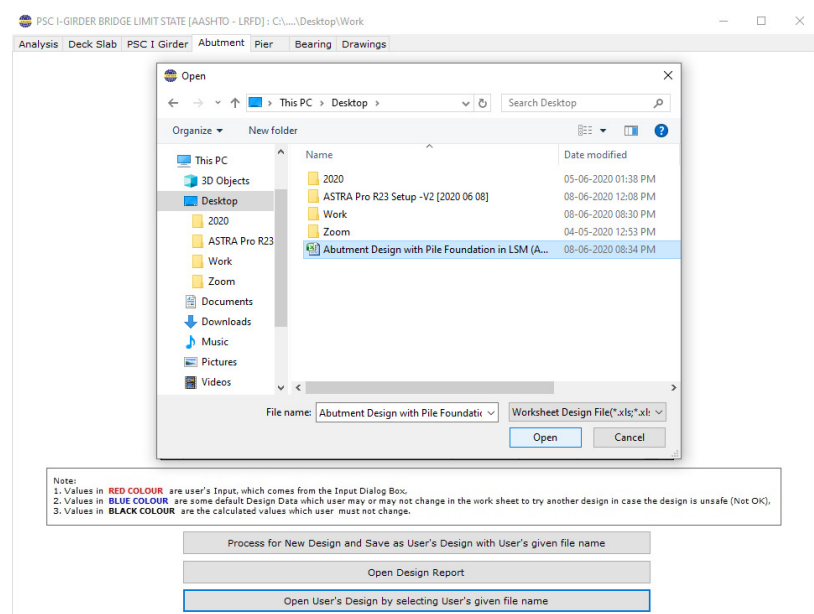
Step 14   The saved design worksheet is once again opened,

Step 15   The processes for the designs of Deck Slab and PSC I-Girder in AASHTO-LRFD are now over, and user is finally come back to the ASTRA Pro main screen.

This is the end of the current session for design of Bridge Abutment in AASHTO-LRFD.

References:

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2. Bridge Engineering Handbook, Second Edition, Superstructure Design, Edited by Wai Fah Chen and Lian Duan, Published by CRC Press, Taylor & Francis Group, 6000 Broken Sound Parkway, NW, Suite 300, Boca Raton, FL 33487, Web site: www.crcpress.com

3. British Standard, Eurocode 2: Design of concrete structures, BS EN 1992-1-1:2004, Published by: British Standards Institution.

4. British Standard, Design Manual for Roads And Bridges, Volume 1, Section 3, Part 14, BD 37/01, Loads For Highway Bridges, Published by: British Standards Institution.

5. British Standard, BS 5400-3 : 2000, Steel, concrete and composite bridges Đ, Part 3: Code of practice for design of steel bridges

6. British Standard, BS 5400-4 : 1990, Steel, concrete and composite bridges — Part 4: Code of practice for design of concrete bridges

7. ASTRA Pro User's Manual and Design Manual, Techsoft Engineering Services, Web site: www.techsoftglobal.com and www.roadbridgedesign.com