DESIGN OF ABUTMENT

General design Consideration

Abutment design loads usually include vertical and horizontal loads from the bridge superstructure, vertical and lateral soil pressures, abutment gravity load, and the live-load surcharge on the abutment backfill materials. An abutment should be designed so as to withstand damage from the earth pressure, the gravity loads of the bridge superstructure and abutment, live load on the superstructure or the approach fill, wind loads, and the transitional loads transferred through the connections between the superstructure and the abutment. Any possible combinations of those forces, which produce the most severe condition of loading, should be investigated in abutment design.

![Diagram of Design loads for abutment]

1. Dead load of superstructure
2. Dead load of wall and footing
3. Dead load of earth on heel side
4. Dead load of earth on toe side
5. Lateral earth pressure on the rear of the wall including surcharge
6. Live load on superstructure
7. Temp. and shrinkage

Fig. 1 Design loads for abutment

The current AASHTO Bridge Design Specifications recommend that either the service load design or the load factor design method be used to perform an abutment design. However, due to the uncertainties in evaluating the soil response to static, cycling, dynamic, and seismic loading, the service load design method is usually used for abutment stability checks and the load factor method is used for the design of abutment components.
The load and load combinations as shown in Fig. 1 may cause abutment sliding, overturning, and bearing failures. Those stability characteristics of abutment must be checked to satisfy certain restrictions. For the abutment with spread footings under service load, the factor of safety to resist sliding should be greater than 1.5; the factor of safety to resist overturning should be greater than 2.0; the factor of safety against soil bearing failure should be greater than 3.0. For the abutment with pile support, the piles have to be designed to resist the forces that cause abutment sliding, overturning, and bearing failure. The pile design may utilize either the service load design method or the load factor design method.

**Miscellaneous Design Considerations**

**Abutment Wingwall**

Abutment wingwalls act as a retaining structure to prevent the abutment backfill soil and the roadway soil from sliding transversely. A wingwall design similar to the retaining wall design, however, live-load surcharge needs to be considered in wingwall design. Fig. 2 shows the design loads for a conventional cantilever wingwall. Bridge wingwalls may be designed to sustain some damage in a major earthquake, as long as bridge collapse is not predicted.

![Design of Abutment](image_url)

Fig. 2 design loading for cantilever wing wall.

\[
M_{AA} = \frac{WL^3}{24} \left[ 3h^2 + (H + 4S)(H + 2h) \right]
\]

\[
P = \frac{WL}{6} \left[ H^2 + (h + H)(h + 3S) \right]
\]

\[
\overline{X} = \frac{M_{AA}}{P}
\]
Abutment Drainage

A drainage system is usually provided for the abutment construction. The drainage system embedded in the abutment backfill soil is designed to reduce the possible buildup of hydrostatic pressure, to control erosion of the roadway embankment, and to reduce the possibility of soil liquefaction during an earthquake. For a concrete-paved abutment slope, a drainage system also needs to be provided under the pavement. The drainage system may include pervious materials, PVC pipes, weep holes, etc. Fig. 3 shows a typical drainage system for highway bridge construction.

![Fig. 3 Typical abutment drainage system](image)

**Design Example**

A 150 ft long 8-lane concrete deck girder bridge is proposed over a small canal as shown in the following figure. Based on geotechnical information and roadway requirements, a seat type abutment is selected. The abutment in transverse direction is 80 ft. From the analysis and calculation of the superstructure, the loads on abutment are listed below:

- Superstructure dead load = 1630 kips
- HS20 Live load = 410 kips
- 1.15 P-load + 1.0 HS load = 280 kips
- Longitudinal live load = 248 kips
- Longitudinal seismic load = 326 kips
- Transverse seismic load = 1241 kips
- Bridge temp. displacement = 2.0 in
- Bridge seismic displacement = 6.5 in
Geotechnical Information

- Live load surcharge: 2.0 ft
- Unit wt. of back fill: 120 pcf
- Allowable soil bearing pressure: 4.0 ksf (1.8 tsf)
- Soil lateral pressure coefficient ($K_a$): 0.3
- Friction coefficient ($f$): $\tan 30^\circ$
- Soil liquefaction potential: Very low
- Ground acceleration: 0.3g

Design Criteria

- Abutment design: USD/LRFD
- Stability analysis: WSD/ASD

Design Assumptions

1. Superstructure vertical load acting on the center line of the abutment footing
2. The soil passive pressure by the soil at the abutment toe is neglected
3. Steel $F_y = 60$ ksi (414 MPa)
4. Concrete $f'_c = 4350$ psi (30 MPa)
5. Unit wt. of concrete = 150 lb/ft$^3$

Solution

1. Design of abutment support width

Responding to seismic load, bridges usually accommodate a large displacement. To provide support at abutments for a bridge with large displacement, enough support width at the abutment must be designed. The minimum abutment support width, as shown in Fig. 4, may be...
equal to the bridge displacement resulting from a seismic elastic analysis or be calculated by the following equation, whichever is larger:

\[ N = (305 + 2.5L + 10H)(1 + 0.002S^2) \]

Where,

- \( N \) = Support width (mm)
- \( L \) = Length in m of the bridge deck to the adjacent expansion joint, or to the end of bridge deck; for single span bridge \( L \) equals the length of the bridge deck/girder
- \( S \) = angle of skew in degree
- \( H \) = Average ht. (m) of the piers supporting the bridge deck from the abutment to the adjacent expansion joint, or to the end of the bridge deck; for simple span bridge \( H = 0 \)

Here, \( L = 45 \) m

\[ N = 420 \text{ mm}. \text{ Add 75 mm for temp. movement. So total requirement for support width = 495 mm. Provide 500 mm = 20” (OK)} \]
2. Check for abutment stability

A preliminary abutment configuration is given in Fig. 5.

Fig. 5 Abutment preliminary configuration

**Vertical load on abutment**

Load per girder x Nos. of girder / Total width of the deck = 1630/80 = 20.4 kips/ft

**Sliding:**

Approximate vertical load = 20.4 + 0.12 \((2+4+14)\times8 + 0.15(15\times2+2.5\times14)\) = 20.6 + 19.2 + 9.75 = 49.0 kips/ft

Horizontal resistance, \(H_R = 49\times\tan(30) = 28.3\) k/"

Horizontal load, \(H = (248+326)/80 + 0.12K_s\times20\times20/2 + 0.12\times2\times20\timesK_a = 7.175 + 8 + 1.6 = 16.78\) k/"

F.S against sliding \(= \frac{H_R}{H} = \frac{28.3}{16.78} = 1.7 > 1.5\) (OK)
Overturning:

Approx. resisting moment, \( M_R = 20.4(4.5+0.83)+19.2(4+7)+4.5\times15/2+5.25\times5.75=385 \text{ k-ft/ft} \)

Overturning moment, \( M = 7.175\times16+8\times20/3+1.6\times20/2=184.2 \text{ k-ft/ft} \)

\[
F.S \text{ against overturning} = \frac{M_R}{M} = \frac{385}{184.2} = 2.1 > 1.5 \text{ (OK)}
\]

Soil bearing Stability:

Total vertical load = 49 k/

\[
\text{Soil bearing pressure} = \frac{\text{Toal vertical load}}{\text{Footing area}} = \frac{49 + 5.125}{15 \times 1} = 3.6 \text{ ksf} < 4.0 \text{ ksf} \text{ (OK)}
\]

3. Design of back-wall

Design wheel load (HS 20-44) = 16 k

Assumed loaded length = 4 ft

Load per unit width = 16/4 = 4 k/

\[
\text{Moment } M = \frac{4\times10}{12} + \frac{K_a\times0.12\times4\times2\times4}{2} + \frac{K_a\times0.12\times4\times4}{2\times3} + 7.175\times2 = 18.35 \text{ k-ft/ft}
\]

Assume \( a = 2'' \)
\[ A_i = \frac{1.5M}{\phi f_y (d - a/2)} = \frac{1.5 \times 18.35 \times 12}{0.9 \times 60 (8.5 - 1)} = 0.82 \text{ in}^2 / \text{ft} \]

\[ a = \frac{Asfy}{0.85 f_y b} = \frac{0.82 \times 60}{0.85 \times 4.35 \times 12} = 1.11'' \]

For this revised \( a \), will only refine the \( A_s = 0.76 \text{ in}^2 \) (OK)

This should be adjusted with the Stem reinforcement.

\[ V_c = 2 \sqrt{f_y b_u d} = 2\sqrt{4350 \times 12 \times 8.5} = 13.45 \text{ kips} \]

\[ V_u = \phi V_c = 0.85 \times 13.45 = 11.44 \text{ kips} > 7.535 \text{ kips} \text{ (OK)} \]

No shear reinforcement is needed.

4. Design of Abutment Stem

Lateral shear, \( V = 7.175 + 0.12 \times 0.3 \times (2 \times 18 + 18 / 2) = 8.795 \text{ kips} / \text{ft} \)
Lateral moment, \( M = 7.175 \times 16 + 0.12 \times 0.3(2 \times 18 \times 18 / 2 + 18 \times 18 / 6) = 128.4 \text{ k-ft/ft} \)

\( d = 30 - 2 = 28" \)

Assume \( a = 5" \)

\[
A_f = \frac{1.5M}{\phi f_y (d - a / 2)} = \frac{1.5 \times 128.4 \times 12}{0.9 \times 60 (28 - 2.5)} = 1.68 \text{ in}^2 / \text{ft}
\]

\[
a = \frac{A_{sfy}}{0.85 f_Y b} = \frac{1.68 \times 60}{0.85 \times 4.35 \times 12} = 2.27"
\]

For this revised \( a \), will only refine the \( A_s = 1.6 \text{ in}^2 \) (OK)

\[
V_c = 2 \sqrt{f_c b_a d} = 2 \sqrt{4350 \times 12 \times 28} = 44.3 \text{ kips}
\]

\[
V_u = \phi V_c = 0.85 \times 44.3 = 37.7 > 8.8 \text{ k} \text{ (OK)}
\]

No shear reinforcement is needed.

5. Design of Abutment Footing

Considering all load combinations, the soil bearing pressure diagram under the abutment footing are shown in Figure below:

Here the footing is eccentrically loaded and stem wall is not at the center of the footing. That is way a vertical load \( P \) and a bending moment \( M \) will be induced on the footing and the resulting eccentricity will be, \( e = M / P \). The resulting bearing pressure with vary linearly and is given by this equation;
If the eccentricity falls beyond the kern of the section, a tension will develop along the edge of the footing. Since, soil can’t transmit tension, the net pressure will increase as a result. For rectangular footing of size \(a \times b\), the maximum pressure can be found from,

\[
q_{\text{max}} = \frac{2P}{3bm} ; \quad q_{\text{max}} \leq q_a
\]

Total vertical force on the soil, \(P = 49 \text{ k/ft}\)

The resultant moment about the toe of the footing, \(M_T = M_R - M = 385.184 = 201 \text{ k-ft/ft}\)

Eccentricity, \(e = \frac{M_N}{P} = \frac{201}{49} = 4.1' \approx \frac{15}{6} = 2.5'\) which is larger than the Kern of the section. So, uplift will be developed.

The live load can be considered to calculate max. soil pressure.

So, total vertical load = Dead Load + live Load = 49+410/80 =54 k/ft

The maximum soil pressure, \(q_{\text{max}} = \frac{2P}{3bm} = \frac{2 \times 54}{3 \times 1 \times 3.4} = 10.5 > q_a \text{ (4ksf)}\)

Pile foundation is needed. Here the toe and heel should be designed as pile cap.

6. Design of Pile and Pile Cap

Total vertical load = 54x80 = 4320 kip

Assume length of Pile = 50 ft, Diameter of the pile is = 12 in

Capacity of a single pile of 12” Ø = 50 tons

Nos of pile reqd. = \(\frac{4320}{(2.2 \times 50)}\) =39.3 \approx 40

Arrangement of pile can be as follows;
Use 2% steel. \( A_s = \frac{\pi D^2}{4} \times 0.01 = 2.26 \text{ in}^2 \)

Capacity of a single pile,

\[
P = 0.85 \left( 0.25 f'_c A_s + A_s f_s \right) = 0.85 \left( 0.25 \times 4.350 \times 110.4 + 2.26 \times 0.4 \times 60 \right) = 148 \text{ k > 110k (OK)}
\]

Provide 8 nos 16mm Ø main bars with 8mm Ø spirals.

Design of Pile Cap

Check for punching and flexural shear.

Determine depth \( d \)

And provide reinforcement.
7. Design of Wingwall

The geometry of wingwall is (as shown in Fig. 2 and figure below),

\[ h = 3.0 \text{ ft}; \quad S = 2.0 \text{ ft}; \quad L = 18.25 \text{ ft} \quad \text{and} \quad H = 18 \text{ ft}, \quad \text{thickness of the wingwall} \quad t = 12'' \]

Lateral earth pressure (eq. fluid pressure) on the wingwall, \( w = K \gamma = 0.3 \times 120 = 36 \text{ lb} = 0.036 \text{ k} \)

\[
V_{A-A} = \frac{wL}{6} \left[ H^2 + (h + H)(h + 3S) \right] \\
= \frac{0.036 \times 18.25}{6} \left[ 18^2 + (3+18)(3+3 \times 2) \right] = 56 \text{ kips}
\]
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\[ M_{A-a} = \frac{wL^2}{24} \left[ 3h^2 + (H + 4S)(H + 2h) \right] \]
\[ = \frac{0.036 \times 18.25^2}{24} \left[ 3 \times 3^2 + (18 + 4 \times 2)(18 + 2 \times 3) \right] = 325 \text{ k-ft} \]

\( d = 12 - 2 = 10'' \)

Assume, \( a = 2'' \)

\[ A_s = \frac{1.5M}{\phi f_y (d - a/2)} = \frac{1.5 \times 325 \times 12}{0.9 \times 60 (10 - 1)} = 12 \text{ in}^2 \]
\[ a = \frac{A_{sfy}}{0.85f_yb} = \frac{1.68 \times 60}{0.85 \times 4.35 \times 18 \times 12} = 0.9'' \]

For this revised \( a \), will only refine the \( A_s = 11.4 \text{ in}^2 \) (OK)

Provide 16mm Ø bar @ 6” O.C

\[ V_c = 2\sqrt{f'_c b_v d} = 2\sqrt{4350 \times 18 \times 12 \times 10} = 285 \text{ kips} \]
\[ V_u = \phi V_c = 0.85 \times 44.3 = 242 \text{ k>56 k} \text{ (OK)} \]

No shear reinforcement is needed.

8. Reinforcement detailing

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