

Design principles of totally prefabricated counterfort retaining wall system compared with existing cast-in-place concrete structures

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- An alternative to cast-in-place concrete retaining walls and the subject of this article is a totally prefabricated concrete counterfort retaining wall, which is composed of a wall component (face panel and counterforts) and a base slab connected through headed anchors.
- The proposed prefabricated concrete counterfort retaining wall design was developed according to the 2012 *AASHTO LRFD Bridge Design Specifications* and compared with an existing cast-in-place counterfort system in Chicago, Ill., for both structural and economic performance.
- The prefabricated concrete counterfort retaining wall's design strength (moment and shear) surpassed that of the existing system with an overall reduction in concrete volume of 57%.

Substructure systems, specifically retaining walls and abutments, constitute a major facet of the bridge construction process. Currently, a majority of substructure construction work is conducted using cast-in-place concrete. However, cast-in-place concrete construction can be associated with several difficulties and drawbacks, such as prolonged site preparation procedures, mitigated work zone safety due to exposure of workers to active traffic, traffic congestion, the requirement for skilled workers, and environmental costs.¹ As a result, the need for shorter construction periods is shifting interest toward accelerated bridge construction methods, such as incorporating precast concrete products in construction. The implementation of precast concrete products in construction provides several economic, safety, and environmental advantages.^{2,3} Precast concrete products are cast using high-performance concrete, for its high strength and durability, under high levels of quality control, which enhances the consistency and uniformity of the materials during mass production and therefore improves the durability of the final product.⁴

As a response to the reported accelerated bridge construction needs, a totally prefabricated concrete counterfort retaining wall system was proposed.^{5,6} The system was optimized and designed according to the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.⁷ It is composed of a face panel strengthened with three counterforts and a base slab. These two components are

connected on-site through headed anchors that extend from the counterforts and are grouted to the shear pockets in the base slab.

This study develops the design principles of the proposed system. It also presents a comparison between this system and an existing typical cast-in-place concrete counterfort retaining wall system. The study highlights the main details, parameters, and assumptions taken in both systems. The advantages of using the proposed prefabricated system and its suitability for widespread adoption in the specified site are examined from an economic point of view. Finally, a parametric study was performed to consider different design parameters, such as various wall heights H , counterfort extension-to-heel length ratio R_{ch} , and anchor bar size in order to facilitate the design process for precast concrete producers.

Background

There have been several attempts to study the applicability of precast concrete elements in bridge and highway construction at the superstructure and substructure levels. These attempts were mostly focused on superstructures, such as bridge decks⁸⁻¹⁰ and support systems such as piers, columns, bent caps, and footings.¹¹⁻¹³ However, few studies have covered fully precast concrete retaining walls or abutment systems.

One such study was conducted to implement the use of a totally precast concrete cantilever retaining wall system.¹⁴ The system consisted of two components: a precast concrete base slab and a precast concrete stem. The components were cast off-site and transported to the construction site, where they were assembled. The length of the segments was limited to 12 ft (3.7 m). The recorded wall height ranged from 4 to 26 ft (1.2 to 7.9 m) to facilitate shipping and handling. However, the main disadvantage of using the conventional cantilever retaining wall is that a relatively thick stem cross section might be required, depending on the wall height, to control cracking and deflections. This imposes difficulties related to shipping and handling because the weight of the component will increase with the increase in the thickness.

As a second example, a fully precast concrete bridge was constructed.¹⁵ The bridge is 115 ft (35.1 m) long and 3 ft (0.9 m) deep with a precast concrete box beam superstructure. A fully precast concrete abutment system was proposed in which a precast concrete abutment stem was connected to a precast concrete base slab. Steel reinforcing bars were extended from the base slab and embedded in the precast concrete stem through grouted sleeves to maintain full moment connection. The system is said to cut down the time required to construct a typical abutment from approximately one month for the cast-in-

place concrete version to only two days for the precast concrete version. To maintain full composite action between the stem and the base, many grouted sleeves may be required. This may impose time-consuming alignment difficulties and require specially trained workers.

Existing cast-in-place counterfort retaining wall

The existing structure chosen for this study is a cast-in-place concrete counterfort retaining wall in Chicago, Ill. The wall is 21 ft 6 in. (6.55 m) high measured from the bottom of the base to the top of the wall. The total width of the base slab is 16 ft (4.9 m). **Figure 1** shows the typical details for geometry and reinforcement. The existing counterforts are 18 in. (460 mm) thick and spaced every 11 ft (3.4 m).

- The counterfort spacing-to-base length ratio can be calculated by dividing the spacing between the counterforts by the total length of the base slab. The typical counterfort spacing-to-base length ratio for the existing structure is 0.84. For a typical base length of 13 ft (4.0 m), the counterfort spacing-to-base length ratio will increase if the spacing between the counterforts increases and vice versa. The ratio can be optimized to yield a more efficient design, as will be shown in later sections. A high counterfort spacing-to-base length ratio indicates that each counterfort is designed to resist a significant amount of load from soil pressure and surcharge loads distributed over 11 ft (3.4 m) tributary area per counterfort. This affects the existing typical design in three major aspects:
- The counterforts require a large cross section and additional steel reinforcement. The counterforts, which act as T beams, will also be extended to a longer distance to increase web depth and, therefore, increase the moment arm to resist applied load. In most cases, the counterfort will be extended to the end of the base slab (Fig. 1).
- The face panel is designed as a one-way slab spanning between the counterforts, which act as supports. The increased spacing between the counterforts requires additional thickness and steel reinforcement to resist the applied positive and negative moments at midspan between the counterforts and over the counterforts, respectively. Furthermore, some additional thickness may be required to control the shear demand in the section at the supports (counterforts).
- The base slab requires similar attention to that of the face panel. The base slab is assumed to act as a one-way slab spanning between the counterforts, which act as supports.

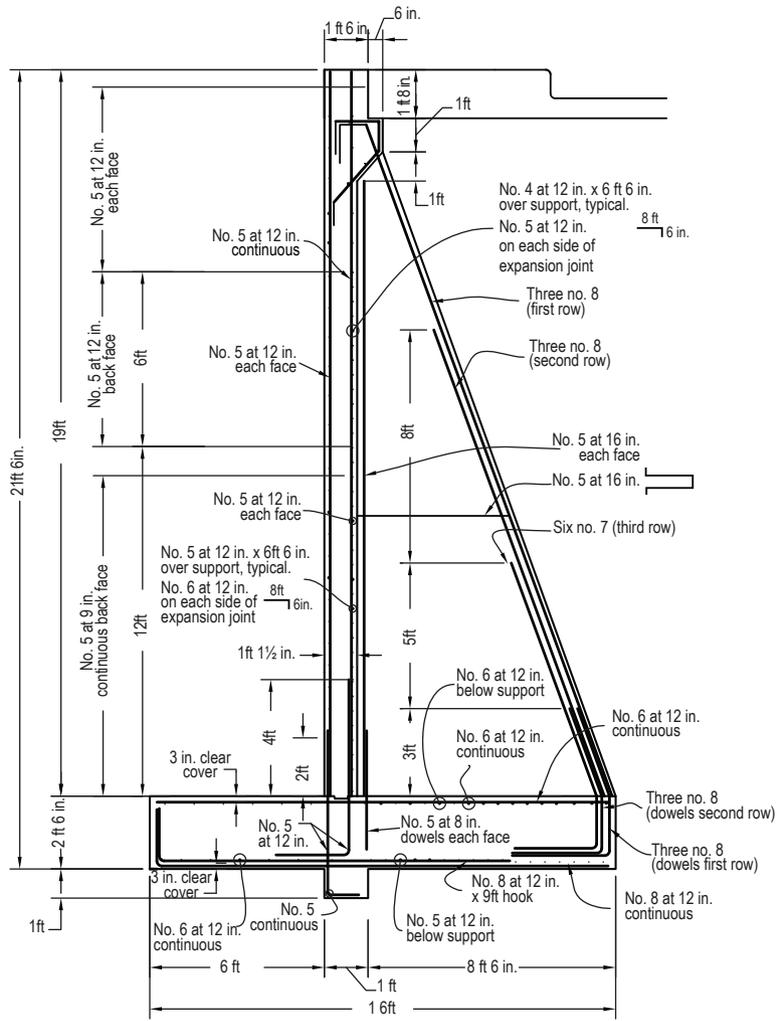


Figure 1. The typical details for geometry and reinforcement. Note: no. 4 = 13M; no. 5 = 16M; no. 6 = 19M; no. 7 = 22M; no. 8 = 25M; 1 in. = 25.4 mm; 1 ft = 0.305 m.



Figure 2. Structural component of the proposed system.

Proposed precast concrete counterfort retaining wall system

The proposed substructure system is composed of two precast concrete structural components: the wall component, which encompasses the face panel and the counterforts, and the base slab (Fig. 2). The system is cast off-site, transported to the construction site, and erected in the least possible amount of time.

The wall component is connected to the base slab using headed anchors. The headed anchors play the most important role in maintaining full composite action between the structural components. Moreover, the anchors are designed to resist the overturning moments and shear forces applied on the system. The counterforts are connected to the face panel through extended L-shaped bars, which enforces the full composite action between them. As a result, counterforts were designed and analyzed as T beams with the face panel as flange and the counterfort as web. Figure 3 represents typical details for the new features introduced in the precast concrete counterfort retaining wall that distinguish it from the cast-in-place concrete counterfort retaining wall.

The tapered concrete cylinder (Fig. 3) is used to create the truncated shear pockets in the base slab in which the extended headed anchors are embedded. The cylinders are

wrapped with a layer of grease to facilitate debonding and are placed in the corresponding location before concrete is placed. They are then removed after the concrete sets to create the truncated shear pockets. Figure 4 shows the final assembly of the proposed system.

Main concepts used for designing the prefabricated concrete counterfort retaining wall

Strengthening a retaining wall with counterforts changes the structural behavior of the retaining wall. In conventional cantilever retaining wall systems, the face panel is the load-resisting component. However, when counterforts are added to the cantilevered wall, the counterforts become the main load-resisting component, with the face panel simply acting as a continuous one-way slab spanning over the counterforts. This allows the cross section of the wall to be reduced significantly while satisfying the strength and serviceability requirements of the AASHTO LRFD specifications. The critical locations in the counterfort retaining wall system to which special attention must be given are as follows:

- counterfort and anchors: bottom section of the counterfort where the bending moment and shear forces are maximum for cantilever-type retaining walls

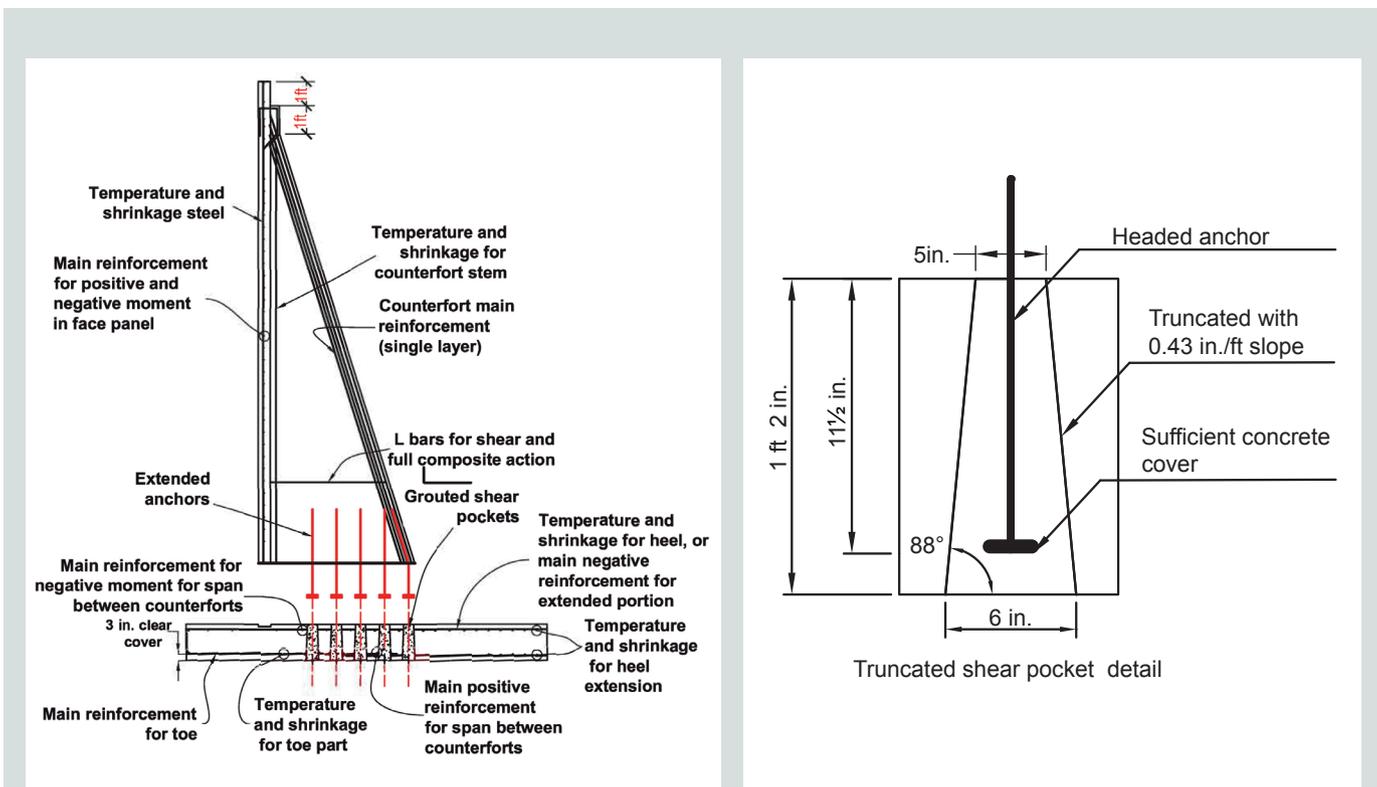


Figure 3. Anchor details for typical construction of the proposed wall. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

- face panel design: for a transverse strip taken at the bottom of the face panel, the midspan between the counterforts for positive moment, and over the counterforts for negative moment
- base slab design: for a transverse strip taken at the base slab, the midspan between the counterforts for positive moment, and over the counterforts for negative moment

The load calculations are divided into vertical and lateral loads applied on the retaining wall as per AASHTO LRFD specifications section 3.3.2.

DC = self-weight of each component

EV = vertical earth pressure

EH = horizontal earth pressure

LS = horizontal and vertical surcharge load

Per section 3.8.8 of the Illinois Department of Transportation's *Bridge Manual*,¹⁶ a live load surcharge LS of 2 ft (0.6 m) of soil should be added to the earth pressure to account for live load. Two cases are considered for placing live load surcharge as per the AASHTO LRFD specifications' Fig. C11.5.6.3. In case 1, live load surcharge is

placed behind the heel of the base slab. This configuration helps maximize the lateral overturning forces applied on the retaining wall without simultaneously increasing the stabilizing vertical forces. It is used for checking the stability of the system against overturning and sliding. In case 2, the live load surcharge is extended over the heel of the base slab. The configuration in case 2 maximizes both lateral and vertical forces. It is used to study the bearing capacity and eccentricity limits of the system.

Design assumptions to be considered for the prefabricated concrete counterfort retaining wall

Geometry

Two main geometric parameters highly contribute to the structural behavior of a prefabricated concrete counterfort retaining wall: the counterfort spacing-to-base length ratio and the length of the counterfort extension along the heel. Counterfort spacing-to-base length ratio controls the tributary load area assigned to each counterfort. When the ratio is reduced, the load applied to each counterfort is reduced and therefore the required thickness of the counterforts is reduced.

Moreover, when the counterfort spacing-to-base length ratio is reduced, the tributary load area applied to the

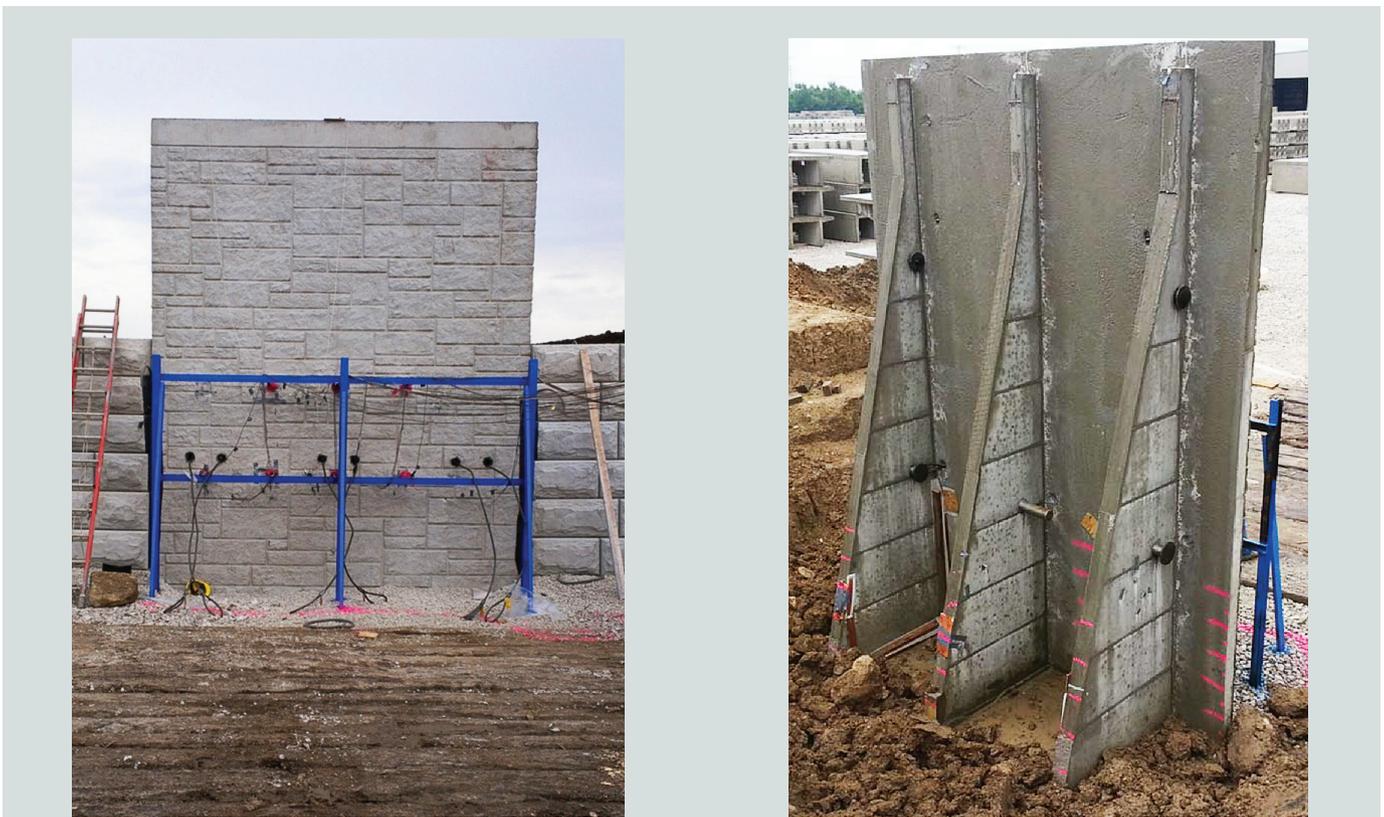


Figure 4. Final assembly of the proposed system.

continuous span's face panel and base slab is reduced. Therefore, this ratio has a major influence on the structural design of the face panel and base slab in the longitudinal direction.

In addition, the bottom depth of the counterfort measured along the interface with the base slab is an important factor that controls the design of the counterfort. The increase in the counterfort–base slab interface distance (counterfort extension) enhances the flexural moment capacity of the counterfort by enlarging the effective depth of the cross section. In the present study, a spacing–to–base length ratio of 0.35 and a counterfort extension–to–heel length ratio of 0.6 were considered, where the slab heel is the distance from the end of the counterfort to the end of the slab and the counterfort extension is the distance from the back face of the face panel to the end of the counterfort along the length of the base slab.

Headed anchors and main steel reinforcement in the counterforts

The design of headed anchors and counterfort main reinforcement is based on two main assumptions:

- The anchors maintain full composite action between the counterforts and the base slab. As a result, the main steel is designed to resist the entire flexural load applied on the counterfort.
- The headed anchors connecting the counterforts to the base slab are designed to fully resist the bending moments and shear forces at the bottoms of the counterforts.

Face panel

The face panel is assumed to act as a continuous slab spanning over the counterforts, which act as support to the face panel. The optimized geometry of the face panel allows the positive and negative bending moments within the face panel to be equalized and significantly reduced. Therefore, the thickness of the face panel is reduced to 152 mm (6 in.) and one layer of steel is provided in the middle of the cross section that can resist both the negative and positive moments. The optimization of the cross section of the face panel is described in the following sections.

L bars connecting the counterforts to the face panel

L bars are used to maintain composite action between the face panel and the counterforts. They are designed to have sufficient development length inside each counterfort and the face panel.

Base slab (heel and toe)

The design of the heel in the base slab is divided into two parts: the cantilever portion extending to the back of the counterforts and the continuous slab portion spanning between the counterforts. The heel is subjected to the soil pressure acting below the footing slab and the vertical weight of the soils and surcharge acting above the footing slab. The toe part is treated as a cantilever beam subjected to upward soil pressure. The soil above the toe was conservatively ignored.

Design procedure

The design procedure of the proposed system is similar to that of a cast-in-place concrete counterfort retaining wall for the typical components. However, it is different for the components where the headed anchors are introduced. It is reasonable to highlight the main aspects of the design procedure for a prefabricated concrete counterfort retaining wall. The design procedure is as follows:

1. Calculate all of the applicable loads in compliance with AASHTO LRFD specifications.
2. Determine the loads acting on each counterfort.
3. Perform the necessary stability checks to ensure that the system meets all of the required safety factors for stability. The system is checked against overturning, sliding, failure due to loss of contact (eccentricity), and bearing pressure.
4. Assume the counterforts are acting as T sections with the face panel as the flange and the counterfort as the web. In this case, the counterforts are assumed to be in full composite action with the base slab. Design for the required moment capacity and provide steel reinforcement that meets the minimum reinforcement requirements. Check for crack control requirements and provide temperature and shrinkage steel.
5. For the same loads taken at the bottom of the counterfort, the headed anchors are designed to resist all of the applied flexural and shear loads. The design of the anchors should also meet the specifications for minimum reinforcement. Moreover, the resistance of the shear pockets against pull-out failure should be examined per the requirements of ACI 318-11¹⁷ to prevent premature failure in the shear pockets before yielding of the anchors. Finally, the development length of the headed anchor should be studied and provided based on the selected bar size.
6. Consider a 1 ft (0.3 m) strip for the face panel assuming that the face panel acts as a one-way slab spanning over the counterforts, which act as a support. Design

the necessary reinforcement for the positive moment at midspan and similarly for the negative moment over the counterforts. The main reinforcement is the longitudinal reinforcement. Provide temperature and shrinkage reinforcement as vertical steel bars.

7. For the base slab, consider a 1 ft (0.3 m) strip between the counterforts, assuming the slab to act as a one-way slab between the counterforts. The bearing pressure should be calculated along with the moment due to the vertical loads (vertical soil pressure and vertical surcharge load) acting behind the face panel. The strip is designed for the negative moment at the counterforts and the positive moment between counterforts. Longitudinal top and bottom steel bars are the main reinforcement for negative and positive moments, respectively. For base slabs with extended heels (that is, the counterforts do not reach the end of the base slab), the extended portion should be treated as cantilever and provided with main reinforcement as transverse top bars. The toe is designed as a cantilever with trapezoidal bearing pressure acting below. The toe is provided with main steel as transverse bottom reinforcement.
8. Check for shear capacity at all of the locations designed in steps 3 through 7.
9. Check for development length, pull-out load, and bearing load for the headed anchors at the level of the base slab to ensure that it meets the code requirements for pull-out resistance and development length.

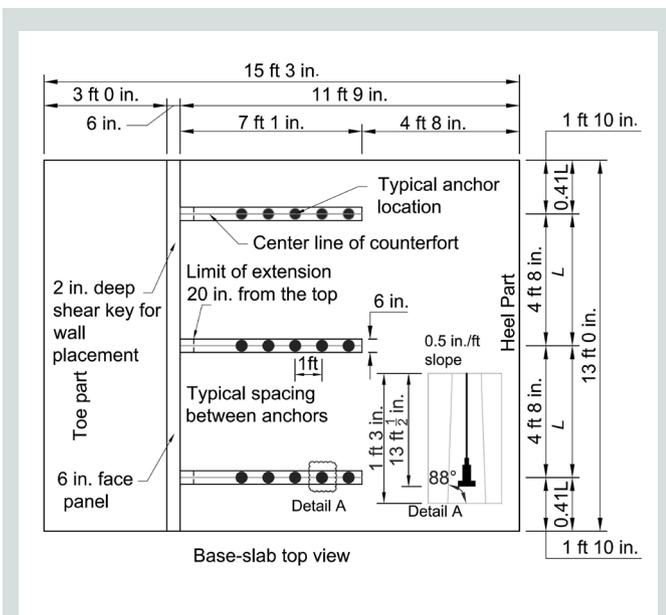


Figure 5. Details of the base-slab geometry. Note: L = spacing center to center between two adjacent counterforts. 1 in. = 25.4 mm; 1 ft = 0.305 m.

Optimization

The number and spacing of counterforts greatly influences the structural design of the face panel and the base slab. When the counterfort spacing-to-base length ratio is reduced, the bending moments in the face panel are minimized and a relatively thinner concrete face panel may be used. The choice of the counterfort spacing-to-base length ratio is simply based on conventional beam theory. Using conventional beam theory, the bending moment in the face panel at midspan between counterforts is equivalent to the negative moment over each counterfort if the length of overhang is $0.41L$, where L is the spacing center to center between two adjacent counterforts. The resulting distribution of bending and shear stresses allows reducing the face-panel thickness and using only one layer of steel reinforcement. One layer of steel can resist equivalent positive and negative moments simultaneously.⁵ **Figure 5** clarifies the spacing of the counterforts resulting from the optimization process.

Material properties

The material properties used in the design and analysis of the proposed system are as follows:

- clear cover for precast concrete components $c_f = 1.50$ in. (38.1 mm)

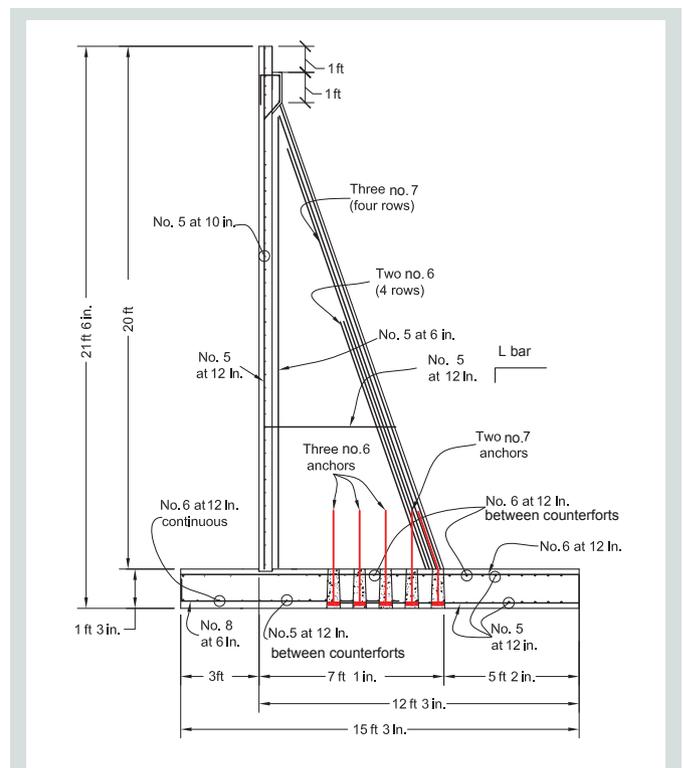


Figure 6. Details of the reinforcement in the proposed system. Note: no. 5 = 16M; no. 6 = 19M; no. 7 = 22M; no. 8 = 25M. 1 in. = 25.4 mm; 1 ft = 0.305 m.

- steel reinforcement yield strength $f_y = 60$ ksi (414 MPa)
- steel modulus of elasticity $E_s = 29,000$ ksi (200 GPa)
- concrete compressive strength $f'_c = 7.5$ ksi (52 MPa)
- density of concrete $\gamma_c = 150$ lb/ft³ (24 kN/m³)
- modulus of elasticity of concrete $E_c = 4888$ ksi (33.70 GPa)
- modular ratio $n = E_s/E_c$ (per AASHTO LRFD specifications section 5.7.1) = 6
- dry earth density $\gamma_s = 125$ lb/ft³ (19.6 kN/m³)
- angle of internal friction $\phi_s = 30$ degrees
- coefficient of active earth pressure (AASHTO LRFD specifications section 3.11.5.7.1) $k_a = 0.51$
- allowable soil bearing resistance provided by geotechnical report $q_{all_prov*} = 10$ kip/ft² (480 kPa)
- factored soil bearing resistance provided by geotechnical report $q_{u_prov*} = 15$ kip/ft² (720 kPa)

The soil properties were obtained from the geotechnical report. Concrete compressive strength is determined from sample cylinders obtained from the precast concrete manufacturer.

Table 1. Load notations and load factors

| Notation | | Load description | Load factors | | |
|----------|-----------------|------------------------------------------|--------------|------------|---------|
| | | | Service I | Strength I | |
| | | | | Minimum | Maximum |
| Vertical | DC1 | Self-weight of face panel | 1 | 0.9 | 1.25 |
| | DC2 | Self-weight of base | 1 | 0.9 | 1.25 |
| | DC3 | Self-weight of counterfort stem | 1 | 0.9 | 1.25 |
| | EV4 | Vertical earth pressure on the base heel | 1 | 1 | 1.35 |
| | EV5 | Vertical earth pressure on the base toe | 1 | 1 | 1.35 |
| | LS _v | Vertical surcharge load | 1 | 0 | 1.75 |
| Lateral | P _{EH} | Horizontal earth pressure | 1 | 0.9 | 1.5 |
| | LS _h | Horizontal surcharge load | 1 | 0 | 1.75 |

Table 2. Stability checks based on AASHTO LRFD specifications section 11.6.3

| Limit state | Stability check | Factor of safety limit | Calculated factor of safety | Check |
|-------------|--------------------------------------------------|------------------------|-----------------------------|-------|
| Service I | Failure due to overturning | 2 | 2.31 | OK |
| | Failure due to sliding | 1.5 | 5.58 | OK |
| | Eccentricity limits (middle 2/3 of footing), in. | 1/3 base = 60 | 32.5 | OK |
| | Bearing capacity failure, kip/ft ² | 15 | 10 | OK |
| Strength I | Failure due to sliding | 1.5 | 3.63 | OK |
| | Eccentricity limits (middle 2/3 of footing), in. | 1/3 base = 60 | 58.1 | OK |
| | Bearing capacity failure, kip/ft ² | 15 | 6.78 | OK |

Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4448 kN.

Design limit states and stability requirements

Service I and Strength I design limit states are used for load calculations per the AASHTO LRFD specifications Table 3.4.1-1. **Table 1** shows the load notations and load factors.

Stability requirements are checked at the service limit state for overturning, bearing resistance, eccentricity, and sliding. At the strength limit state, stability is checked for bearing resistance, eccentricity, and sliding, taking into account the minimum and maximum load combinations in accordance with AASHTO LRFD specifications sections 11.6.3.2, 11.6.3.3, and 11.6.3.6, respectively.

The proposed system was chosen to have a height equivalent to the existing types of retaining wall. The typical width of the retaining wall is limited to 13 ft (4.0 m). This dimension is generally limited by transportation restrictions. For a total wall height of 21 ft 6 in. (6.55 m), a 15.25 ft (4.648 m) long base slab is chosen to sufficiently satisfy the stability requirements of the AASHTO LRFD specifications at service and strength limit states. **Table 2** summarizes the results for the stability checks.

Final design for flexure

Counterfort reinforcement The main reinforcement in the counterforts is designed to resist the entire applied lateral load on the system. Three rows of no. 7 (22M) bars and two rows of no. 6 (19M) bars were provided in the form of one layer as per the AASHTO LRFD specifications sections 8.16.1.2 and 5.7.3.3.2 for flexural design and minimum required reinforcement, respectively. In addition, no. 5 (16M) bars at 6 in. (150 mm) spacing were provided as vertical steel reinforcement in the web for temperature and shrinkage. Moreover, no. 4 (13M) L-shaped bars spaced at 9 in. (230 mm) were provided in the horizontal direction for two purposes: shear resistance in the counterfort web and maintaining full composite action between the counterfort and the face panel.

Headed anchors The anchors constitute the most important component because they provide the connection between the counterforts and the face panel. Similar to the design of the counterforts, the anchors are designed to fully resist the total applied lateral load. The applied lateral loads are divided into flexural moment and shears. The provided anchors were three no. 6 (19M) and two no. 7 (22M) headed anchors starting from the end of the counterfort extension.

Base slab The design of the base slab is divided into three sections: design of the toe, the heel between the counterforts, and the extended part of the heel. The toe is subjected

to soil pressure generated below the base slab. The provided main reinforcement in the toe was no. 8 (25M) bars at 6 in. (150 mm) spacing. The section of the heel between the counterforts is treated as a continuous slab spanning between the counterforts, which act as the supports. It is subjected to the applied vertical load of the soil. In case of a rigid pavement, the vertical component of the live load surcharge can be neglected. The main reinforcement of the heel between the counterforts was no. 6 (19M) bars at 12 in. (300 mm) spacing for positive moment (top) and no. 5 (16M) bars at 12 in. (300 mm) spacing for negative moment (bottom) provided in the transverse direction. The cantilever part of the heel is assumed fixed at the end of the counterfort extension. The provided top steel reinforcement was no. 6 (19M) bars at 12 in. (300 mm) spacing, which replaced the top reinforcement for temperature and shrinkage.

Face panel The face panel is designed as a continuous one-way slab spanning between the counterforts, which act as supports. The optimization process using the beam theory led to equivalent positive bending moments at the midspan between the counterforts and negative bending moment over the counterforts. As a result, the thickness of the face panel can be reduced to 6 in. (150 mm) and one layer of steel (no. 5 [16M]) at 10 in. (250 mm) can be used to resist both equivalent positive and negative bending moments. The vertical reinforcement in the face panel was provided as no. 5 (16M) at 12 in. (300 mm) for temperature and shrinkage reinforcement.

Figures 5 and 6 present the layout of the base slab and the reinforcement details of the proposed wall, respectively.

Check for shear resistance

The location of the shear critical section d_v is calculated according to AASHTO LRFD specifications section 5.8.2.9. The shear resistance of concrete is checked at six critical locations depending on the loading application for each component:

- the critical section for shear at the face panel at distance d_v from the counterfort
- at the level of concrete at the bottom of the counterfort
- at the level of anchors between the counterfort and the base slab
- at the critical section for shear in the toe part of the base slab
- at the assumed fixity point of the cantilever section in the base heel
- at the critical section for shear in the base slab

Anchors are distributed along the interface distance between the counterfort and the heel. When the loads are applied, the anchors will be subjected to tension. Therefore, cracks will generate in the concrete around the anchors and propagate toward the inside of the counterfort web causing shear failure. To prevent this situation, the spacing between the temperature and shrinkage reinforcing bars (vertical steel) was reduced from 16 to 6 in. (410 to 150 mm) to create an arrest mechanism to the crack propagation. This behavior is discussed in detail in Farhat et al.¹⁸

Development length and pull-out resistance of anchors

Development length The headed anchors are responsible for maintaining full composite action between the counterforts and the base slab. The anchors are subject to failure either by yielding of steel or by pullout in the shear pocket. Therefore, it is important to check whether sufficient resistance to anchor pullout is provided along with sufficient development length to prevent failure in the shear pocket. The development length requirements are developed in ACI 318-11. According to ACI 318-11 Eq. (12.6.1) for the development length of headed anchors, the net bearing area of the head (that is, the area of the head minus the area of the bar) is required to be greater than four times the area of the bar. In addition, the spacing of the anchors is required to be greater than four times the bar diameter.

$$l_{dt} = \left(\frac{0.016\psi_e f_y}{\sqrt{f'_{cdt}}} \right) d_b \quad (\text{ACI 318-11 Eq. [12.6.2]})$$

where

l_{dt} = development length for headed anchors inside the base slab (ACI 318-11 section 12.6.2)

ψ_e = modification factor for epoxy-coated bars (ACI 318-11 section 12.6.2)

f_y = steel reinforcement yield strength = 60,000 psi (414 MPa)

f'_{cdt} = concrete compressive strength not exceeding 6000 psi (41 MPa)

d_b = bar diameter

The following properties are needed for development length and pull-out resistance calculations:

- bar diameter (no. 7 [22M]) $d_b = 0.875$ in. (22 mm)
- bar area $A_b = 0.6$ in.² (400 mm²)

- diameter of anchor head $d_{ha} = 2.8$ in. (71 mm)
- thickness of anchor head $t_{ha} = 0.625$ in. (15.9 mm)
- number of threads per inch $n_{ta} = 8$
- area of anchor head $A_{ha} = 6.16$ in.² (397 mm²)
- net bearing area $A_{net\ bearing} = A_{ha} - A_b = 5.56$ in.² (359 mm²)
- spacing between anchors $S_a = 12$ in. (300 mm)
- check to see whether the net bearing area is greater than four times area of the bar (ACI 318-11 section 12.6.1)
 $A_{net\ bearing} - 4A_b = 3.16$ in.² (204 mm²) > 0 → OK
- check to see whether anchor spacing is greater than four times the bar diameter (ACI 318-11 section 12.6.1)
 $S_a - 4d_b = 8.5$ in. (220 mm) > 0 → OK

The required development length for the headed anchor calculated by ACI 318-11 Eq. (12.6.2) was found to be 13 in. (330 mm). The minimum required base slab thickness to ensure full development of the anchors is calculated as $l_{dt} + c_l + t_{ha}$, which is equal to 15 in. (380 mm), whereas the base slab thickness used was 16 in. (410 mm).

Pull-out resistance The resistance to pullout is divided into two parts: resistance against shear failure and resistance against bearing pressure. The following properties are used to calculate the shear and bearing resistance of the grouted pocket.

- top diameter of shear pocket $D_{pocket_top} = 5$ in. (130 mm)
- bottom diameter of shear pocket $D_{pocket_bot} = 6$ in. (150 mm)
- loaded area of contact between concrete and grout $A_{surface} = 463.31$ in.² (29,891 mm²)
- reduction factor for shear (ACI 318-11 section 9.3.2.4) $\phi_{vn} = 0.75$
- reduction factor for bearing (ACI 318-11 section 9.3.2.4) $\phi_{bearing} = 0.65$
- angle of the truncated shear pocket sides $Slope_{angle} = 88$ degrees
- ultimate design tensile axial load in the anchor $T_u = 36$ kip (160 kN)

The value for the nominal shear strength V_n of the grouted shear pocket is calculated using ACI 318-11 Eq. (11-3).

$$V_n = 2\lambda\sqrt{f'_c}A_{surface}\sin\theta \quad (\text{ACI 318-11 [Eq. 11-3]})$$

where

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete (1.0 for normalweight concrete)

θ = angle of inclination of shear pocket

The calculated value of nominal shear strength V_n is 80.25 kip (356.9 kN) using the data listed previously. The nominal shear resistance V_n is 60.2 kip (277 kN). The ultimate design tensile axial in the anchor T_u equals 36 kip (160 kN). This indicates that the shear strength of the concrete interface with the grout can resist the shear component of the applied pullout load.

In a similar manner, the bearing strength of the grouted shear pocket can be calculated using ACI 318-11 Eq. (10.14.1). The surface area of the conical frustum was calculated from the top to the level of the provided development length of the headed anchor.

$$R_u = 0.85\phi_{bearing}f'_cA_{surface}\cos\theta \quad (\text{ACI 318-11 Eq. [10.14.1]})$$

The value of the nominal bearing load was 67 kip (300 kN) using ACI 318-11 Eq. (10.14.1), which exceeded the ultimate pull-out load necessary to cause yielding in the anchor. This indicates that the bearing strength of the concrete interface with the grout is capable of resisting the bearing component of the applied pull-out load. The value

of the bearing strength ensures yielding in the steel anchor before crushing in the concrete inside the shear pocket.

Comparison between the proposed precast concrete counterfort retaining wall system and cast-in-place concrete system

The proposed system was optimized to have geometric efficiency that can be reflected in the form of a reduction in the weight, sizes, and concrete volume of all of the wall components compared with the existing cast-in-place wall. **Table 3** presents a comparison of the general properties of the existing wall and the proposed wall.

Analysis of Table 3 shows a significant reduction in the concrete volume in the proposed system of 57% compared with the volume of concrete in the existing system. Table 3 also shows a significant reduction (53%) in the total weight of the structure in the proposed system. The large weight reduction provides an important advantage for transportation and handling purposes.

The existing retaining wall was designed in 1968 using AASHTO standard specifications. To provide a reasonable comparison between the existing structure and the proposed system, the design of the existing was reevaluated using the AASHTO LRFD specifications. **Table 4** shows a comparison between the existing retaining wall using the AASHTO standard specifications and the AASHTO LRFD specifications and the proposed precast concrete wall using the AASHTO LRFD specifications for a typical base width of 13 ft (4.0 m). The comparison focuses on bending moment and shear forces at the critical location in every wall component. The ratio of

Table 3. Comparison of general properties of existing cast-in-place concrete wall and proposed wall

| | Properties | Existing structure | Proposed wall | Reduction in proposed wall, % |
|--------------------------------|---------------------------------------------|--------------------|---------------|-------------------------------|
| Weight of component | Weight of base slab, kip | 78 | 37.2 | 52 |
| | Weight of wall component, kip | 77.5 | 35.46 | 54 |
| | Total weight, kip | 155.5 | 72.63 | 53 |
| Geometry | Thickness of base slab, in. | 30 | 15 | 50 |
| | Thickness of face panel, in. | 13.5 | 6 | 56 |
| | Thickness of counterforts, in. | 18 | 6 | 67 |
| | Number of counterforts | 2 | 3 | -50 |
| Concrete volume for components | Volume of face panel, ft ³ | 277.9 | 131.6 | 53 |
| | Volume of base slab, ft ³ | 520 | 247.8 | 52 |
| | Volume of all counterforts, ft ³ | 245.9 | 104.8 | 57 |

Note: 1 in. = 25.4 mm; 1 ft³ = 0.028 m³; 1 kip = 4.448 kN.

Table 4. Comparison of existing retaining wall using AASHTO standard and LRFD specifications and proposed precast concrete wall using AASHTO LRFD specifications

| Component | Property | Location | Existing structure | | Proposed wall |
|-------------------------|-----------|--------------------------|--------------------------------|----------------------------|----------------------------|
| | | | AASHTO standard specifications | AASHTO LRFD specifications | AASHTO LRFD specifications |
| Face panel | M_r/M_u | Midspan | 1.42 | 1.57 | 7.01 |
| | M_r/M_u | Counterfort | 1.67 | 1.75 | 7.01 |
| | V_r/V_u | Distance d_v from face | 1.53 | 1.22 | 2.2 |
| Counterfort | M_r/M_u | Bottom of counterfort | 2.13 | 1.9 | 1.42 |
| | V_r/V_u | Bottom of counterfort | 2.23 | 1.81 | 1.7 |
| Toe | M_r/M_u | Face of stem | 1.7 | 1.11 | 1.25 |
| | V_r/V_u | Distance d_v from face | 1.71 | 1.26 | 1.47 |
| Heel (continuous strip) | M_r/M_u | Midspan | 1.97 | 1.9 | 2.19 |
| | M_r/M_u | Counterfort | 2.77 | 2.3 | 3.07 |
| | V_r/V_u | Distance d_v from face | 2.17 | 1.84 | 3.21 |
| Heel (cantilever strip) | M_r/M_u | End of counterfort | n/a | n/a | 3.07 |
| | V_r/V_u | End of counterfort | n/a | n/a | 3.84 |
| Anchors | M_r/M_u | Top face of base slab | n/a | n/a | 1.43 |
| | V_r/V_u | Top face of base slab | n/a | n/a | 1.053 |

Note: d_v = location of shear critical section; M_r = factored moment capacity; M_u = applied factored moment; n/a = not applicable; V_r = factored shear capacity; V_u = applied factored shear.

Table 5. Required minimum base thickness for increasing anchor bar diameter

| Anchor bar size | Anchor diameter, in. | Head thickness, in. | Development length, in. | Minimum base thickness, in. | Weight of slab per longitudinal length, kip/ft |
|-----------------|----------------------|---------------------|-------------------------|-----------------------------|------------------------------------------------|
| No. 5 | 0.625 | 0.5 | 9.3 | 12 | 2 |
| No. 6 | 0.75 | 0.56 | 11 | 13 | 2.1 |
| No. 7 | 0.875 | 0.625 | 12.9 | 15 | 2.4 |
| No. 8 | 1 | 0.625 | 14.9 | 17 | 2.8 |
| No. 9 | 1.128 | 0.68 | 16.8 | 19 | 3.1 |

Note: no. 5 = 16M; no. 6 = 19M; no. 7 = 22M; no. 8 = 25M; no. 9 = 29M; 1 in. = 25.4 mm; 1 kip/ft = 1.356 kN/m.

the design moment to the applied factored moment M_r/M_u is used. This ratio provides an indication of the safety factor present in the section and, therefore, the effectiveness of the section. Similarly, the ratio of the design shear capacity to the applied factored shear V_r/V_u is considered at the critical location for shear. For locations where the loading is on the same face undergoing tension, the critical location for shear is assumed to be at the face of the section. For locations where the loading is on the same face undergoing compression, the critical section for shear (location d_v) is calculated per the AASHTO LRFD specifications section 5.8.2.9.

Analysis of Table 4 shows that the AASHTO LRFD specifications design of the existing retaining wall exhibits lower values for the moment and shear ratios compared with the standard specifications design. However, the comparison between the proposed wall and the existing wall using the AASHTO LRFD specifications shows that the proposed wall generally exhibits higher moment and shear ratios. This reflects the efficiency in the design of the proposed system. The moment and shear ratios at the bottom of the counterfort and the anchors show a lower value compared with the same location in the existing wall. This is because the design was based on choosing the optimum

Table 6. Results of the parametric study showing all the included variables

| H , ft | L_b , ft | R_{ch} | ρ_c , % | Anchors | ρ_a , % | t_b , in. | w_w , kip | w_b , kip | Anchors M_r/M_u | Counterfort M_r/M_u | Counterfort V_r/V_u |
|----------|------------|----------|--------------|-------------------------|--------------|-------------|-------------|-------------|-------------------|-----------------------|-----------------------|
| 16 | 12 | 0.5 | 0.4 | Four no. 6 | 0.82 | 13 | 21.5 | 25.4 | 1.3 | 1 | 1.35 |
| | | 0.6 | 0.34 | Two no. 6 + two no. 5 | 0.52 | 13 | 22.8 | 25.4 | 1.5 | 1.2 | 1.15 |
| | | 0.7 | 0.37 | One no. 6 + three no. 5 | 0.4 | 13 | 24.2 | 23.4 | 1.7 | 1.7 | 1.05 |
| | | 0.8 | 0.42 | One no. 6 + three no. 5 | 0.34 | 13 | 25.4 | 23.4 | 2 | 2 | 1.05 |
| 17 | 13 | 0.5 | 0.46 | Five no. 6 | 1.03 | 13 | 23.9 | 27.5 | 1.4 | 1.2 | 1.48 |
| | | 0.6 | 0.39 | One no. 6 + four no. 5 | 0.57 | 13 | 25.4 | 27.5 | 1.5 | 1.4 | 1.13 |
| | | 0.7 | 0.4 | Five no. 5 | 0.44 | 12 | 26.9 | 25.4 | 1.6 | 1.8 | 1.04 |
| | | 0.8 | 0.44 | Five no. 5 | 0.37 | 12 | 28.5 | 25.4 | 1.9 | 2.4 | 1.04 |
| 18 | 13.5 | 0.5 | 0.47 | One no. 7 + four no. 6 | 0.98 | 15 | 25.6 | 32.9 | 1.4 | 1.1 | 1.44 |
| | | 0.6 | 0.4 | Two no. 6 + three no. 5 | 0.57 | 13 | 27.5 | 28.5 | 1.4 | 1.3 | 1.08 |
| | | 0.7 | 0.44 | Two no. 6 + three no. 5 | 0.46 | 13 | 29.1 | 28.5 | 1.7 | 1.8 | 1.08 |
| | | 0.8 | 0.48 | One no. 6 + four no. 5 | 0.37 | 13 | 30.9 | 28.5 | 1.9 | 2.4 | 1.01 |
| 19 | 14 | 0.5 | 0.48 | Four no. 7 | 0.84 | 15 | 27.7 | 34.1 | 1.4 | 1.1 | 1.31 |
| | | 0.6 | 0.44 | Two no. 7 + two no. 6 | 0.56 | 15 | 29.5 | 34.1 | 1.6 | 1.4 | 1.13 |
| | | 0.7 | 0.48 | One no. 7 + three no. 6 | 0.43 | 15 | 31.3 | 34.1 | 1.8 | 1.9 | 1.04 |
| | | 0.8 | 0.52 | One no. 7 + three no. 6 | 0.37 | 15 | 33.2 | 34.1 | 2.1 | 2.5 | 1.04 |
| 20 | 14.5 | 0.5 | 0.5 | Five no. 7 | 1.12 | 15 | 29.7 | 35.3 | 1.4 | 1 | 1.46 |
| | | 0.6 | 0.45 | Five no. 6 | 0.63 | 13 | 32.1 | 30.6 | 1.3 | 1.3 | 1.05 |
| | | 0.7 | 0.52 | Five no. 6 | 0.52 | 13 | 34.1 | 30.6 | 1.6 | 1.8 | 1.05 |
| | | 0.8 | 0.55 | Five no. 6 | 0.44 | 13 | 36.3 | 30.6 | 2 | 2.4 | 1.05 |
| 21 | 15 | 0.5 | 0.58 | One no. 8 + four no. 7 | 1.09 | 17 | 31.6 | 41.4 | 1.5 | 1.2 | 1.42 |
| | | 0.6 | 0.53 | One no. 7 + four no. 6 | 0.62 | 15 | 34.1 | 36.6 | 1.4 | 1.4 | 1.03 |
| | | 0.7 | 0.55 | One no. 7 + four no. 6 | 0.51 | 15 | 36.4 | 36.6 | 1.7 | 1.9 | 1.03 |
| | | 0.8 | 0.59 | One no. 7 + four no. 6 | 0.43 | 15 | 38.7 | 36.6 | 2 | 2.5 | 1.03 |
| 22 | 15.5 | 0.5 | 0.59 | Two no. 8 + three no. 7 | 1.07 | 17 | 33.8 | 42.8 | 1.5 | 1.1 | 1.36 |
| | | 0.6 | 0.57 | Five no. 7 | 0.77 | 15 | 36.6 | 37.8 | 1.6 | 1.4 | 1.19 |
| | | 0.7 | 0.6 | Three no. 7 + two no. 6 | 0.55 | 15 | 39.1 | 37.8 | 1.7 | 1.9 | 1.06 |
| | | 0.8 | 0.66 | Three no. 7 + two no. 6 | 0.47 | 15 | 41.6 | 37.8 | 2.1 | 2.6 | 1.06 |
| 23 | 16 | 0.5 | 0.64 | Three no. 8 + two no. 7 | 1.07 | 17 | 36 | 44.2 | 1.4 | 1.1 | 1.31 |
| | | 0.6 | 0.61 | Five no. 7 | 0.73 | 15 | 39.1 | 39 | 1.4 | 1.4 | 1.08 |
| | | 0.7 | 0.64 | Four no. 7 + one no. 6 | 0.56 | 15 | 41.9 | 39 | 1.7 | 1.9 | 1.02 |
| | | 0.8 | 0.7 | Four no. 7 + one no. 6 | 0.47 | 15 | 44.6 | 39 | 2 | 2 | 1.02 |

Note: H = height of the wall measured from the bottom of the base slab to the top of the wall; L_b = total length of the base slab; M_r = design moment; M_u = applied factored moment; R_{ch} = counterfort extension-to-heel length ratio; t_b = total thickness of the base slab; V_r = design shear capacity; V_u = applied factored shear; w_b = weight of the base slab; w_w = weight of the wall component including the stem and the counterforts; ρ_a = reinforcement ratio of the anchors; ρ_c = reinforcement ratio of the counterforts. no. 5 = 16M; no. 6 = 19M; no. 7 = 22M; no. 8 = 25M; no. 9 = 29M; 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 ft = 0.305 m.

extension for the counterfort from the rear side of the face panel along the length of the heel. This was done to control and minimize the weight of the wall component. The values of the moments and shear ratios at the bottom of the counterfort can be simply raised by increasing the extension distance that increases the moment arm and therefore increases the moment and shear capacities.

Parametric study

A parametric study was conducted to evaluate the structural performance of the proposed system for a variety of configurations. It was used to provide a basis to compare the effect of increasing wall height on anchor selection, counterfort reinforcement, base slab thickness, and total weight of the wall components. When the wall height increases, the flexural moment and shear force increase. Therefore, additional strength was necessary to meet the code requirements. This can be attained by either increasing the length of the extension of the counterfort or by increasing the amount of steel reinforcement within the allowable strain limits. From the development length calculations presented previously, the minimum thickness of the base slab is controlled by the development length to be provided for the headed anchors. The minimum base thickness is calculated as the required development length that varies according to the anchor diameter plus the thickness of the head and the clear cover. **Table 5** presents the required minimum base thickness for increasing anchor bar diameter and the corresponding weight assuming a typical 13 ft (4.0 m) wide slab. The required development length increases with bar size, causing the thickness of the base slab to increase (Table 5). The increasing base thickness imposes difficulties and restrictions for transportation and handling. As a result, the parametric study was performed to optimize the design so that minimum base and wall weights were obtained.

Generally, the length of the base slab increases when the height of the wall increases in order to maintain the stability requirements assuming constant soil properties. To account for this change, the ratio of the counterfort

extension-to-heel length R_{ch} is introduced in the parametric study. The same soil conditions introduced in the design were used in the parametric study.

The data in **Table 6** are divided into three sections. The first section is the reinforcement ratio in the counterfort ρ_c , which increases with the increase of R_{ch} , except for the 0.6 ratio. For R_{ch} of 0.5, the steel ratio is controlled by the moment strength value. When R_{ch} is increased to 0.6, the ratio of steel drops for all of the studied cases. This is due to the increase in the moment arm of the counterfort, which increases the moment capacity, and this reduces the required area of steel. However, when R_{ch} is raised to 0.7 and 0.8, ρ_c shows an increasing trend. At this level, the moment arm is enormously increased and the steel ratio is controlled by the control of cracking requirement.

The second section is the reinforcement ratio of the anchors ρ_a . The reinforcement ratio of the anchors is highest with extension to a heel length ratio of 0.5. It decreases with the increase of R_{ch} . However, for R_{ch} values of 0.7 and 0.8, the design is controlled by shear forces at the interface between the counterfort and the base slab. This can be deduced from the shear capacity to ultimate shear force values V_u/V_a in Table 6. As a result, the area of anchors provided cannot be further reduced. Additional analysis of Table 6 shows that an R_{ch} value of 0.6 allows reduction of the bar size for wall heights above 20 ft (6 m). This reduction is pronounced in **Fig. 7**, which represents the variation of the ratios of steel in the counterfort and anchors versus the counterfort extension-to-heel length ratio. For example, a 20 ft high wall requires five no. 7 (22M) anchor bars for extension to heel length ratio of 0.5 and five no. 6 (19M) anchor bars for a ratio of 0.6. This reduction permits a reduction of the minimum required thickness of the base slab, resulting in significant weight reduction (Table 5). The lowest steel ratio required in the counterforts at all heights is when R_{ch} is 0.6. In addition, the value of ρ_a exhibits a sharp drop when the value of R_{ch} increases from 0.5 to 0.6. When the ratio is increased to 0.7 and 0.8, the value of ρ_a drops at a shallower slope. The ratio of steel in the anchors and counterforts can be interpolated for counter-

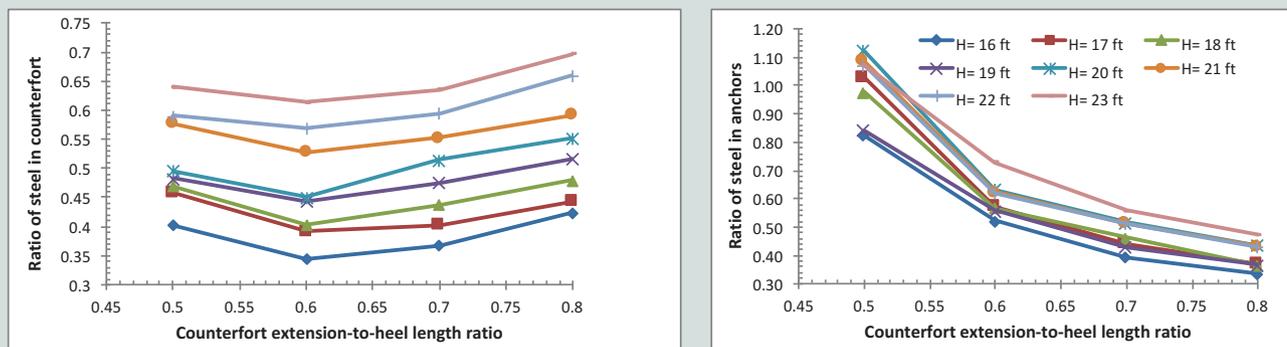


Figure 7. Variation of steel ratios in the counterfort and anchors versus the extension-to-heel ratio. Note: 1 ft = 0.305 m.

fort extension to heel ratios other than the specified. This relation can be useful in determining the optimum geometry and steel reinforcement to obtain the lightest possible structure while satisfying all code requirements.

The third section is the weight of the components. The 0.5 ratio has the lowest wall weight w_w and the highest base slab weight w_b . The low extension-to-heel ratio helps reduce the weight of the wall component; however, it requires the use of larger anchor bars, which causes the minimum base thickness to increase. With the increase in the extension-to-heel length ratio, the wall weight increases and the base slab weight decreases.

Conclusion

This study develops the design principles for totally prefabricated concrete counterfort retaining walls. In addition, a comparison between the proposed system and an existing counterfort cast-in-place concrete retaining wall system was established. The comparison is focused on design, structural efficiency, and structural performance. A parametric study was performed to assess the performance of the proposed system in increasing heights. From the study, the following conclusions can be drawn:

- A prefabricated counterfort retaining wall system is an efficient solution for fast-track construction. It also provides the advantage of minimal energy use, accelerated construction, the use of high-strength construction materials in a consistent and accurate fabrication process, congestion reduction, and safety promotion.
 - A reduction in the counterfort spacing-to-base length ratio from 0.84 for a typical design to 0.35 causes a significant reduction in concrete volume of 57%. This results in cost savings in both materials and time of construction.
 - A counterfort extension-to-heel length ratio of 0.6 is shown to be optimum for the design of the proposed system. It results in a significant reduction in the weight of the components of 54% compared with the existing structure. Simultaneously, it satisfies the code requirements for moment and shear strengths.
 - The tapered design of the shear pocket enhances the resistance of the anchors against pull-out loads. The results show that the grout used is capable of resisting the pull-out and bearing forces and maintaining the integrity between the wall component and the base slab.
 - The headed anchors, which extend from the counterforts to the base slab, are verified to maintain the integrity of the system by resisting the shear forces at the interface between the wall and the base components.
- The proposed system could potentially provide a cost-effective solution and structurally adequate option that can be used in bridge and highway applications due to the reduced amount of materials used while satisfying code requirements.

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Notation

| | | | | | |
|--------------------|---|---------------------------------------------------|-------------------|---|----------------------------------------------------------------------------------------|
| A_b | = | bar area | d_{ha} | = | diameter of anchor head |
| A_{ha} | = | area of anchor head | d_v | = | location of shear critical section |
| $A_{net_bearing}$ | = | net bearing area | DC | = | self-weight of each component |
| $A_{surface}$ | = | loaded area of contact between concrete and grout | D_{pocket_bot} | = | bottom diameter of shear pocket |
| c_l | = | clear cover for precast concrete components | D_{pocket_top} | = | top diameter of shear pocket |
| d_b | = | bar diameter | E_c | = | modulus of elasticity of concrete |
| | | | EH | = | horizontal earth pressure |
| | | | E_s | = | steel modulus of elasticity |
| | | | EV | = | vertical earth pressure |
| | | | f'_c | = | concrete strength |
| | | | f'_{cdt} | = | concrete compressive strength not exceeding 6000 psi (41.3 MPa) |
| | | | f_y | = | steel reinforcement yield strength |
| | | | H | = | height of the wall measured from the bottom of the base slab to the top of the wall |
| | | | k_a | = | coefficient of active earth pressure (AASHTO LRFD specifications section 3.11.5.7.1) |
| | | | l_{dt} | = | development length for headed anchors inside the base slab (ACI 318-11 section 12.6.2) |
| | | | L | = | spacing center to center between two adjacent counterforts |
| | | | L_b | = | total length of the base slab |
| | | | LS | = | horizontal and vertical surcharge load |
| | | | M_r | = | design moment |
| | | | M_u | = | applied factored moment |
| | | | n | = | modular ratio = E_s/E_c per AASHTO LRFD specifications section 5.7.1 |
| | | | n_{ta} | = | number of threads per inch |
| | | | q_{all_prov*} | = | allowable soil bearing resistance provided by geotechnical report |
| | | | q_{u_prov*} | = | factored soil bearing resistance provided by geotechnical report |

| | | |
|------------------|---|--------------------------------------------------------------------------------------------------------------------------|
| R_{ch} | = | counterfort extension-to-heel length ratio |
| S_a | = | spacing between anchors |
| $Slope_{angle}$ | = | angle of the truncated shear pocket sides |
| t_b | = | total thickness of the base slab |
| t_{ha} | = | thickness of anchor head |
| T_u | = | ultimate design tensile axial in the anchor |
| V_n | = | nominal shear strength |
| V_r | = | design shear capacity |
| V_u | = | applied factored shear |
| w_b | = | weight of the base slab |
| w_w | = | weight of the wall, including the stem and the counterforts |
| γ_c | = | unit weight of concrete |
| γ_s | = | dry earth weight |
| θ | = | angle of inclination of shear pocket |
| λ | = | modification factor reflecting the reduced mechanical properties of lightweight concrete (1.0 for normalweight concrete) |
| ρ_a | = | reinforcement ratio of the anchors |
| ρ_c | = | reinforcement ratio of the counterforts |
| ϕ | = | strength reduction factor |
| $\phi_{bearing}$ | = | reduction factor for bearing (ACI 318-11 section 9.3.2.4) |
| ϕ_s | = | angle of internal friction |
| ϕ_{vn} | = | reduction factor for shear (ACI 318-11 section 9.3.2.4) |
| ψ_e | = | modification factor for epoxy-coated bars (ACI 318-11 section 12.6.2) |

About the authors



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Abstract

Counterfort retaining walls are usually constructed with cast-in-place concrete, which can be complicated by site preparation, formwork, and traffic congestion. An alternative is the totally prefabricated concrete counterfort retaining wall, which is composed of a wall component (face panel and counterforts) and a base slab connected on-site through headed anchors. The anchors extend downward from the counterforts and are designed to be embedded in shear pockets in the base slab. While the design of totally prefabricat-

ed concrete counterfort retaining walls shares some features with cast-in-place concrete systems, it also has specific requirements for anchor connections, strength of shear pockets, and counterfort design. The proposed totally prefabricated concrete counterfort retaining wall design was developed according to the 2012 *AASHTO LRFD Bridge Design Specifications* and compared with an existing cast-in-place counterfort system in Chicago, Ill., for both structural and economic performance. The totally prefabricated concrete counterfort retaining wall's design strength (moment and shear) surpassed that of the existing system with an overall reduction in concrete volume reaching 57%. A parametric study identified a counterfort spacing-to-base length ratio of 0.35 and a counterfort extension-to-heel length ratio of 0.6 as optimal values. This geometry achieved the highest economic efficiency while meeting all strength requirements of the AASHTO LRFD specifications.

Keywords

AASHTO, accelerated bridge construction, American Association of State Highway and Transportation Officials, behavior, bridge, counterfort, dapped end, headed anchor, prestressed concrete thin-stemmed member, reinforcement, retaining wall, service load cracking.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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