11.5.1 General

Revise the Paragraph 3 as follows:

Earth retaining structures shall be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the structure. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining wall applications defined as temporary shall be considered to have a service life of 5 years ~~36 months~~ or less.

Add the following new paragraph to the end:

Abutments shall be designed using the SERVICE –I LIMIT STATE loads, as provided in these Specifications, and the Working Stress Design (WSD) method provided in the *Caltrans Bridge Design Specifications (2000)*, dated November 2003.

11.5.2 Service Limit States

Add the following to the end of the first paragraph:

Limit eccentricity under Service Limit State loading to B/6 and B/4 when spread footings are founded on soil and rock, respectively.

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11.5.6 Resistance Factors

Revise the 1st and 2nd Paragraph as follows:

Resistance factors for geotechnical design of foundations are specified in Tables 10.5.5.2.2-1 through ~~10.5.5-3,~~ 10.5.5.2.4-1, and Table 1.

If methods other than those prescribed in these Specifications are used to estimate resistance, the resistance factors chosen shall provide the same reliability as those given in Tables 10.5.5.2.2-1 through ~~10.5.5-3~~ 10.5.5.2.4-1~~, and Table 1~~.

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11.5.6 Resistance Factors

Replace Table 11.5.6-1

Table 11.5.6-1 Resistance Factors For Permanent Retaining Walls

WALL-TYPE AND CONDITION		RESISTANCE FACTOR
Nongravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		1.00
Pullout resistance of anchors(1)	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65(1) 0.70(1) 0.50(1)
Pullout resistance of anchors(2)	Where proof tests are conducted	1.0(2)
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A615 bars) • High strength steel (e.g., ASTM A722 bars) • High strength steel strands (e.g. ASTM A416)	0.90(3) 0.80(3) 0.75(3)
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity Walls		
Bearing resistance	• Gravity and semi-gravity walls • MSE walls	0.55 0.65 ~~Article 10.5 applies~~
Sliding	• Friction • Passive resistance	1.00 0.50 ~~Article 10.5 applies~~
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements(4) • ~~Static loading~~ • ~~Combined static/earthquake loading~~ Grid reinforcements(4)(5) • ~~Static loading~~ • ~~Combined static/earthquake loading~~	0.90 ~~0.75~~ ~~1.00~~ 0.80 ~~0.65~~ ~~0.85~~
Tensile resistance of geosynthetic reinforcement and connectors	• ~~Static loading~~ • ~~Combined static/earthquake loading~~	0.90 ~~1.20~~
Pullout resistance of tensile reinforcement	• ~~Static loading~~ • ~~Combined static/earthquake loading~~	0.90 ~~1.20~~
Abutments and Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

Revise Table 11.5.6-1 Note 3

(3) Apply to maximum proof test load for the anchor. For mild steel apply resistance factor to F_y. For high-strength steel bars and strands apply the resistance factor to guaranteed ultimate tensile strength.

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11.6.1.5.2 Wingwalls

Revise as follows:

Reinforcing bars or suitable rolled sections shall be spaced across the junction between wingwalls and abutments to tie them together. Such bars shall extend into the concrete ~~masonry~~ on each side of the joint far enough to develop the strength of the bar as specified for bar reinforcement, and shall vary in length so as to avoid planes of weakness in the concrete at their ends. If bars are not used, an expansion joint shall be provided and the wingwall shall be keyed into the body of the abutment.

11.6.1.6 Expansion and Contraction Joints

Revise as follows:

Weakened plane ~~Contraction~~ joints should ~~shall~~ be provided at intervals not exceeding 24.0 ~~30.0~~ ft. and expansion joints at intervals not exceeding 90.0 ft. for conventional retaining walls and abutments. All joints shall be filled with approved filling material to ensure the function of the joint. Joints in abutments shall be located approximately midway between the longitudinal members bearing on the abutments.

11.6.2.1 Abutments

Revise as follows:

The provisions of Articles 10.6.2, 10.7.2, 10.8.2 ~~10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4~~, and 11.5.2 shall apply as applicable.

C11.6.2.2

Revise as follows:

For a conventional reinforced concrete retaining wall, experience suggests that differential wall settlements exceeding ~~on the order of~~ 1 in 500 ~~to 1 in 1,000~~ may overstress the wall.

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C11.6.2.3

Add after Paragraph 3

With regard to selection of a resistance factor for evaluation of overall stability of walls, examples of structural elements supported by a wall that may justify the use of the 0.65 resistance factor include a bridge or pipe arch foundation, a building foundation, a pipeline, a critical utility, or another retaining wall. If the structural element is located beyond the failure surface for external stability behind the wall illustrated conceptually in Figure 11.10.2-1, a resistance factor of 0.75 may be used.

Available slope stability programs produce a single factor of safety, *FS*. The specified resistance factors are essentially the inverse of the *FS* that should be targeted in the slope stability program.

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11.6.3.3 Overturning

Revise as follows:

~~For foundations on soil, the location of the resultant of the reaction forces shall be within the middle one-half of the base width.~~

~~For foundations on rock, the location of the resultant of the reaction forces shall be within the middle three-fourths of the base width.~~

The factored gross nominal bearing resistance of the effective base width shall be equal to or greater than the factored gross uniform bearing stress on soil and the gross factored maximum bearing stress on rock.

C11.6.3.3

Revise as follows:

~~The specified criteria for the location of the resultant, coupled with investigation of the bearing pressure, replace the investigation of the ratio of stabilizing moment to overturning moment. Location of the resultant within the middle one-half of the base width for foundations on soil is based on the use of plastic bearing pressure distribution for the limit state.~~

Excessive differential contact stress due to eccentric loading can cause a wall to rotate excessively leading to failure. To prevent rotation, the wall base must be sized to provide adequate factored bearing resistance under the eccentric and vertical load combination that causes the highest equivalent uniform bearing stress.

A bearing resistance check for all potential factored load combinations will ensure the location of the resultant of eccentric loading will not fall outside of the base width. Then neither a check of the ratio of the stabilizing moment to overturning moment nor a limit on the eccentricity is necessary.

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11.10.6.2.1 Maximum Reinforcement Loads

Modify Paragraph 1

Maximum reinforcement loads shall be calculated using the Simplified Method ~~approach.~~ or the Coherent Gravity Method. The Simplified Method shall be considered to apply to both steel and geosynthetic reinforced wall systems. The Coherent Gravity Method shall be applied primarily to steel soil reinforcement systems. For the Simplified Method, ~~For this approach~~, the load in the reinforcements shall be obtained by multiplying the vertical earth pressure at the reinforcement by a lateral earth pressure coefficient, and applying the resulting lateral pressure to the tributary area for the reinforcement. For the Coherent Gravity Method, the load in the reinforcements shall be obtained in the same way as the Simplified Method, except as follows:

- The vertical earth pressure at each reinforcement level shall be computed using an equivalent uniform base pressure distribution over an effective width of reinforced wall mass determined in accordance with the provisions of Articles 11.6.3.1 and 11.6.3.2, and

- For steel reinforced wall systems, the lateral earth pressure coefficient used shall be equal to k_0 at the point of intersection of the theoretical failure surface with the ground surface at or above the wall top, transitioning to k_a at a depth of 20.0 ft below that intersection point, and constant at k_a at depths greater than 20.0 ft. If used for geosynthetic reinforced systems, k_a shall be used throughout the wall height.

All other provisions in this article are applicable to both methods.

C11.10.6.2.1

Add a new paragraph at the beginning and modify Paragraph 1

The development of the Simplified Method for estimating reinforcement loads is provided in Allen, et al. (2001). The Coherent Gravity Method has been used in MSE wall design practice for many years for steel reinforced wall systems. Detailed procedures for the Coherent Gravity Method are provided in Allen, et al. (2001) and in Mitchell and Villet (1987). Its application to geosynthetic soil reinforcement systems results in conservative designs.

The design specifications provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. Research by Allen and Bathurst (2003) and Allen et al. (2003) indicates that reinforcement load is linear with reinforcement spacing to a reinforcement vertical spacing of 2.7 ft or more, though a vertical spacing of this magnitude should not be attempted unless the facing is considered to be adequately stiff to prevent excessive bulging between layers (see Article C11.10.2.3.2).

11.10.6.2.1 Maximum Reinforcement Loads

Modify Paragraph 3

For the Simplified Method, ~~The~~ factored horizontal stress, σ_H, at each reinforcement level shall be determined as:

$$\sigma_H = \gamma_P \left(\sigma_v k_r + \Delta\sigma_H\right) \qquad (11.10.6.2.1\text{-}1)$$

where:

γ_P = the load factor for vertical earth pressure *EV* from Table 3.4.1-2

k_r = horizontal pressure coefficient (dim.)

σ_v = pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (ksf)

$\Delta\sigma_H$ = horizontal stress at reinforcement level resulting from any applicable concentrated horizontal surcharge load as specified in Article 11.10.10.1 (ksf)

Modify Paragraph 4

For the Simplified Method, ~~V~~vertical stress for maximum reinforcement load calculations shall be determined as shown in Figures 1 and 2. For the Coherent Gravity Method, vertical stress shall be calculated at each reinforcement level using an equivalent uniform base pressure that accounts for load eccentricity caused by the lateral earth pressure acting at the back of the reinforced soil mass above the reinforcement level being considered. This base pressure shall be applied over an effective width of reinforced wall mass determined in accordance with the provisions of Articles 11.6.3.1 and 11.6.3.2. As is true for the Simplified Method, live load is not included in the vertical stress calculation to determine T_{max} for assessing pullout loads when using the Coherent Gravity Method.

C11.10.6.2.1

Modify Paragraph 4

The use of *EV* for the load factor in this case for both methods (i.e., the Simplified and Coherent Gravity Methods) should be considered an interim measure until research is completed to quantify load prediction bias and uncertainty.

Add after Paragraph 5

Note that T_{max}, the factored tensile load in the soil reinforcement, must be calculated twice for internal stability design as follows: (1) for checking reinforcement and connection rupture, determine T_{max} with live load surcharge included in the calculation of σ_v; (2) for checking pullout, determine T_{max} with live load surcharge excluded from the calculation of σ_v.

11.10.6.2.1 Maximum Reinforcement Loads

Modify Paragraph 5

For the Simplified Method, ~~T~~the lateral earth pressure coefficient k_r is determined by applying a multiplier to the active earth pressure coefficient, k_a. The k_a multiplier for the Simplified Method shall be determined as shown in Figure 3. For assessment of reinforcement pullout, the Simplified Method multiplier for steel strip walls shall be used for all steel reinforced walls. For reinforcement rupture, the multiplier applicable to the specific type of steel reinforcement shall be used. For the Coherent Gravity Method, the lateral earth pressure coefficient used for internal stability design of steel reinforced MSE wall systems shall be determined as shown in Figure 4. For geosynthetic reinforced wall systems, k_a is used throughout the wall height. For both methods, k_a shall be determined using Eq. 3.11.5.3-1, assuming no wall friction, i.e., $\delta = \beta$. For the Coherent Gravity Method, k_0 shall be determined using Eq. 3.11.5.2-1.

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11.10.6.2.1 Maximum Reinforcement Loads

Add after Figure 11.10.6.2.1-3

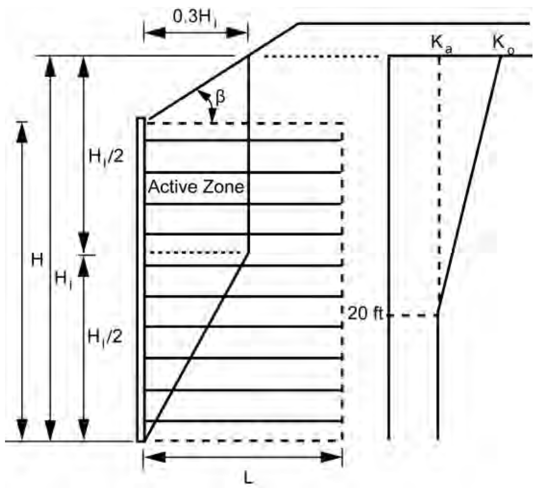

Figure 11.10.6.2.1-4—Determination of Lateral Earth Pressure Coefficients for Internal Stability Design of Steel Reinforced MSE Walls Using the Coherent Gravity Method

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11.10.6.4.2a

Add after Paragraph 3

When soil backfill conforms to the following criteria:

- pH = 5 to 10
- Resistivity ≥2000 ohm-cm
- Chlorides ≤250 ppm
- Sulfates ≤500 ppm
- Organic Content ≤1 percent

Sacrificial thicknesses shall be computed for each exposed surface as follows:

- Loss of galvanizing takes 10 years
- Loss of carbon steel = 1.1 mil./yr. after zinc depletion

C11.10.6.4.2a

Add after Paragraph 4

Considerable data from numerous MSE in California has been gathered for a national research project to develop the resistance and load factors for corrosion in actual field conditions. As a result, the equations, design parameters and construction specifications are under review. This section continues current practice in conjunction with the more aggressive soils permitted in the *Caltrans Standard Special Provisions (2006)* and in the future edition of *Caltrans Standard Specifications (2010)*, until that review is complete.

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Add References:

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Seed, H. B., and R. V. Whitman. 1970. "Design of Earth Retaining Structures for Dynamic Loads." In *Proc., ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures*. American Society of Civil Engineers, NY, pp. 103-147.

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Appendix A11.1.1.1 Mononobe-Okabe Analysis

Revise the 5th paragraph as follows:

The value of h_a , the height at which the resultant of the soil pressure acts on the abutment, may be taken as *H/3* for the static case with no earthquake effects involved. However, it becomes greater as earthquake effects increase. This has been shown empirically by tests. ~~and theoretically by Wood (1973), who~~ The active force, E_{AE}, has been traditionally divided into two components, static soil pressure and dynamic soil pressure. Wood (1973) found that the resultant of the dynamic soil pressure acted approximately at mid height. Seed and Whitman have suggested that h_a could be obtained by assuming that the resultant of the static soil pressure ~~static component of the soil force~~ (computed from Eq. 1 with $\theta = k_v = 0$) acts at *H/3* from the bottom of the abutment, whereas the resultant of the dynamic soil effect ~~additional dynamic effect~~ should be taken to act at a height of 0.6 *H*. ~~For most purposes, it is sufficient to assume h =H/2 with a uniformly distribution pressure.~~ Recent research by Ortiz, et al (1983), Bolton, et al (1984), and Atik, et al (2010) indicated that the resultant of E_{AE} is at about one third wall height from the bottom of the wall. For a gravity, semi-gravity, prefabricated modular retaining wall and a MSE, E_{AE} may be assumed to be distributed in a triangular shape, with h_a taken as *H/3*. (see Figure 1a)

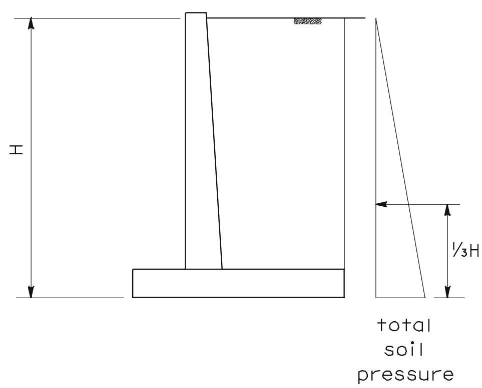

Figure A11.1.1.1-1a Application of total soil pressure from seismic effects

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