SEISMIC DESIGN OF REINFORCED EARTH RETAINING WALLS AND BRIDGE ABUTMENTS

AASHTO Design Method
For
Reinforced Earth Structures
Subject to Seismic Forces

Technical Bulletin: MSE 9

PART A: RETAINING WALLS

PART B: BRIDGE ABUTMENTS

JANUARY 1995
FORWARD

This report is made up of two distinct parts: the first (Part A) is devoted to retaining walls, the second (Part B) to bridge abutments.

Nevertheless, the second part devoted to “seismic” design of bridge abutments calls for concepts and definitions presented for retaining walls; it is therefore necessary to read the report in its entirety.
## INDEX

<table>
<thead>
<tr>
<th>PART A</th>
<th>RETAINING WALLS:</th>
<th>SHEET NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>Introduction</td>
<td>1</td>
</tr>
<tr>
<td>2.0</td>
<td>General</td>
<td>4</td>
</tr>
<tr>
<td>2.1</td>
<td>Forward</td>
<td>4</td>
</tr>
<tr>
<td>2.2</td>
<td>Dynamic Forces - Definitions</td>
<td>4</td>
</tr>
<tr>
<td>2.2.1</td>
<td>External Stability</td>
<td>4</td>
</tr>
<tr>
<td>2.2.2</td>
<td>Internal Stability</td>
<td>5</td>
</tr>
<tr>
<td>2.3</td>
<td>The Accelerations to be Taken Into Account</td>
<td>5</td>
</tr>
<tr>
<td>2.4</td>
<td>Load Combination</td>
<td>8</td>
</tr>
<tr>
<td>2.5</td>
<td>Factors of Safety and Allowable Stress</td>
<td>9</td>
</tr>
<tr>
<td>3.0</td>
<td>External Stability</td>
<td>9</td>
</tr>
<tr>
<td>3.1</td>
<td>Seismic Coefficients</td>
<td>9</td>
</tr>
<tr>
<td>3.2</td>
<td>Determining the Additional Horizontal Thrust $P_{se}$</td>
<td>10</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Vertical Wall With Horizontal Backfill</td>
<td>12</td>
</tr>
<tr>
<td>3.2.2</td>
<td>Vertical Wall With Sloping Backfill</td>
<td>13</td>
</tr>
<tr>
<td>3.3</td>
<td>Effective Inertia Force, $P_r$</td>
<td>13</td>
</tr>
<tr>
<td>3.4</td>
<td>Performing the External Stability Calculations</td>
<td>14</td>
</tr>
<tr>
<td>4.0</td>
<td>Internal Stability</td>
<td>16</td>
</tr>
<tr>
<td>4.1</td>
<td>The Internal Dynamic Load, $P_i$</td>
<td>16</td>
</tr>
<tr>
<td>4.2</td>
<td>Distribution of The Dynamic Load, $P_i$ Among the Reinforcing Strips</td>
<td>16</td>
</tr>
<tr>
<td>4.3</td>
<td>Comparison of Calculated Dynamic Increment of Tensile Loads With F.E.M. Results</td>
<td>19</td>
</tr>
<tr>
<td>4.4</td>
<td>Tension at The R.S. Connection to the Facing</td>
<td>20</td>
</tr>
<tr>
<td>4.5</td>
<td>R.S. Pull-Out Resistance During Earthquakes</td>
<td>21</td>
</tr>
<tr>
<td>PART B</td>
<td>BRIDGE ABUTMENTS:</td>
<td>SHEET NO.</td>
</tr>
<tr>
<td>--------</td>
<td>-------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>5.0</td>
<td>Introduction</td>
<td>23</td>
</tr>
<tr>
<td>6.0</td>
<td>General</td>
<td>24</td>
</tr>
<tr>
<td>6.1</td>
<td>Forward</td>
<td>24</td>
</tr>
<tr>
<td>6.2</td>
<td>Accelerations to be Taken Into Account</td>
<td>24</td>
</tr>
<tr>
<td>6.3</td>
<td>Combined Loads and Safety Factors</td>
<td>25</td>
</tr>
<tr>
<td>7.0</td>
<td>External Stability</td>
<td>25</td>
</tr>
<tr>
<td>7.1</td>
<td>Method of Calculation</td>
<td>25</td>
</tr>
<tr>
<td>7.2</td>
<td>Beam Seat Stability</td>
<td>25</td>
</tr>
<tr>
<td>7.2.1</td>
<td>Loads Transmitted From the Bridge Deck</td>
<td>25</td>
</tr>
<tr>
<td>7.2.2</td>
<td>Forces Arising From The Beam Seat Itself</td>
<td>26</td>
</tr>
<tr>
<td>7.2.3</td>
<td>Forces Transmitted From The Backfill</td>
<td>26</td>
</tr>
<tr>
<td>7.3</td>
<td>External Stability of The Reinforced Earth Mass</td>
<td>26</td>
</tr>
<tr>
<td>7.3.1</td>
<td>Forces Transmitted From The Deck</td>
<td>26</td>
</tr>
<tr>
<td>7.3.2</td>
<td>Forces Transmitted From The Bridge Seat itself</td>
<td>27</td>
</tr>
<tr>
<td>7.3.3</td>
<td>Inertia Forces of the Reinforced Earth Volume</td>
<td>27</td>
</tr>
<tr>
<td>7.3.4</td>
<td>Forces Transmitted From the Backfill Behind the Structure</td>
<td>27</td>
</tr>
<tr>
<td>7.3.5</td>
<td>External Stability Calculations</td>
<td>28</td>
</tr>
<tr>
<td>8.0</td>
<td>Internal Stability</td>
<td>30</td>
</tr>
<tr>
<td>8.1</td>
<td>Method of Calculation</td>
<td>30</td>
</tr>
<tr>
<td>8.2</td>
<td>Loads Considered in the Calculation of P</td>
<td>30</td>
</tr>
<tr>
<td>8.3</td>
<td>Verification of Internal Stability</td>
<td>32</td>
</tr>
<tr>
<td>9.0</td>
<td>Conclusion</td>
<td>32</td>
</tr>
<tr>
<td>References</td>
<td></td>
<td>33</td>
</tr>
<tr>
<td>Appendix</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FIGURE</td>
<td>SHEET NO</td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>External Stability, Supplementary Forces</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>Internal Stability Supplementary Force</td>
<td>6</td>
</tr>
<tr>
<td>3a</td>
<td>Maximum Accelerations Within and Behind the Reinforced Earth Volume, 19.7 ft. wall</td>
<td>7</td>
</tr>
<tr>
<td>3b</td>
<td>Maximum Accelerations Within and Behind the Reinforced Earth Volume, 34.5 ft. wall</td>
<td>7</td>
</tr>
<tr>
<td>4</td>
<td>Average Maximum Acceleration $a_m$, Depending on the &quot;Free Field&quot; Acceleration, $a_o$</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>External Stability - Level Surcharge Condition</td>
<td>12</td>
</tr>
<tr>
<td>6a</td>
<td>External Stability - Infinite Slope Condition</td>
<td>15</td>
</tr>
<tr>
<td>6b</td>
<td>External Stability - Broken Back Slope Condition</td>
<td>15</td>
</tr>
<tr>
<td>7a</td>
<td>Internal Stability - Sloping Condition</td>
<td>17</td>
</tr>
<tr>
<td>7b</td>
<td>Internal Stability - Level Condition</td>
<td>17</td>
</tr>
<tr>
<td>8a</td>
<td>Internal Stability - Loads Included in the Calculation of $T_m$</td>
<td>18</td>
</tr>
<tr>
<td>8b</td>
<td>Distribution of Dynamic Load Among the Strips</td>
<td>18</td>
</tr>
<tr>
<td>8c</td>
<td>Maximum Dynamic Increment of Tensile Loads, 19.7 Ft. Wall</td>
<td>22</td>
</tr>
<tr>
<td>8d</td>
<td>Maximum Dynamic Increment of Tensile Loads, 34.5 Ft. Wall</td>
<td>22</td>
</tr>
<tr>
<td>9</td>
<td>Stability of Beam Seat</td>
<td>28</td>
</tr>
<tr>
<td>10</td>
<td>External Stability of Abutment</td>
<td>29</td>
</tr>
<tr>
<td>11</td>
<td>Calculating Internal Dynamic Force, $P_n$ in Different Cases</td>
<td>31</td>
</tr>
</tbody>
</table>
PART A - RETAINING WALLS

1. INTRODUCTION

It is generally agreed that the stability of retaining walls exposed to earthquakes is not a matter for real concern.

In a paper delivered in 1970 at the ASCE Specialty Conference, Professors H. Bolton Seed and Robert V. Whitman said:

"Few cases of retaining wall movement or collapse of walls located above the water table have been reported in the literature on earthquake damage. (...) it seems likely that the small number of accounts of retaining wall performance is not necessarily indicative of the lack of occurrence of wall movements: this type of damage is not particularly dramatic compared with other forms of earthquake damage and thus may often be considered of minor significance."

The same authors find confirmation of their view that the stability of retaining walls is not crucial based on the scant attention accorded to such structures in the construction codes:

"While all investigators have concluded that the dynamic lateral pressures developed during earthquakes exceed the static pressures on earth retaining structures, a survey of a number of engineering companies highway departments and port authorities in California shows that (...) it is general practice to make no special allowance for increased lateral pressures on retaining walls (...) due to earthquake effects. This also appears to be the case in many other countries."

It is interesting to note that habits have not changed much over the last twenty years. Having recently done a survey similar to that of Seed and Whitman we note:
The seismic design of cantilever retaining walls is a subject on which there is not many guidelines. In fact, most highway departments do not design cantilever retaining walls for seismic loads. Instead they assume, based on previous performance, that static design is adequate. Conversations with the California Department of Transportation confirms this.

In fact, even the most detailed seismic design codes, such as the recommendations from the French Association for Seismic Engineering published in 1990, contain a few rather simplistic rules for standard retaining walls, together with extremely complex design methods for building structures.

In their ASCE communications, Seed and Whitman explained why the stability of retaining walls during earthquakes was a problem which very often resolved itself. Considering the order of magnitude of the additional stresses caused by the effects of "normal" earth tremors, and the usual values of safety coefficients, they state:

"It should be noted that the factor of safety provided in the design of walls for static pressures may be adequate to prevent damage or detrimental movements during many earthquakes. (...) Thus where backfill and foundation soils remain stable, it is only in areas where very strong ground motions might be expected, for walls with sloping backfills or heavy surcharge pressures and for structures which are very sensitive to wall movements, that special seismic design provisions for lateral pressure effects may be necessary."

Such considerations of a very general nature obviously also apply to Reinforced Earth structures which, better than any other type of structure, are known to be able to withstand deformation without damage. Their performance record provides ample proof of this. Many Reinforced Earth structures have been built in seismic zones, usually without any special precautions or extra reinforcement for earthquakes. Some have already been tested by an actual earthquake and have been unaffected.

In Friuli, Italy, four small Reinforced Earth walls 15 to 20 feet in height were at the epicenter of the 1976 earthquake (6.4 Richter magnitude). The design of these walls was based on the minimum requirements for static conditions only. There was no additional reinforcement density or length provided, yet no damage occurred to these walls.

In Japan, most structures are located in a seismic zone; design calculations include a check for earthquake effects, but the final design will, in practice, be based on the routine static approach. In 1983, a serious 7.7 Richter magnitude earthquake occurred in the Akita area, causing considerable damage to buildings, bridges, and port
installations. None of the 24 local Reinforced Earth structures suffered any damage. (Report available).

In 1989, the Loma Prieta Earthquake, a severe 7.1 Richter magnitude event, shook the San Francisco area, causing serious damage to bridges and buildings. Only three privately owned walls out of the 20 Reinforced Earth structures located in the area were designed for earthquake loading conditions. The remaining Reinforced Earth structures, with the exception of one, are owned by Caltrans who has no earthquake design requirements for retaining walls. All 20 of the Reinforced Earth structures whether designed for earthquake resistance or not, performed without any damage. (Report available)

In 1994, the Northridge earthquake, a severe 6.7 Richter magnitude event, shook the densely populated San Fernando Valley, 20 miles northwest of Los Angeles. Severe damage occurred to buildings, bridges and freeways. Twenty-one Reinforced Earth walls and 2 Reinforced Earth bridge abutments were located within the effected area. One-half of the walls and the two bridge abutments, were designed for seismic loads; the others were not. The Reinforced Earth structures performed extremely well, with only superficial damage to one wall, whether specifically designed for earthquake loads or not. (Report available)

These observations confirm that, since no particular provisions for earthquake effects are normally required when designing conventional retaining structures, they may be even less necessary for Reinforced Earth retaining structures due to their outstanding performance record, inherent strength, flexibility, and high degree of damping. And yet, we have always applied special design rules to Reinforced Earth structures built in recognized seismic zones. The practical design method presented in this report, and adopted by the AASHTO technical committee in 1992, is the result of research carried out over fifteen years with the assistance of leading experts. Tests on reduced-scale models, measurements in full-scale test structures subjected to vibration, research led by specialists, such as the late Professor Seed, assembling and processing the research results, and finally, in 1989, a series of dynamic finite element computations enabled us to further refine our seismic design method. The practical design method presented in this report explains in detail, the method outlined in the 1994 AASHTO interim specifications for highway bridges.

1 The late Professor H. Bolton Seed of the University of California at Berkeley is frequently cited in this report. It was the review and evaluation he performed together with Professor James K. Mitchell which helped us develop an understanding for how a Reinforced Earth structure will react to seismic motion. On the basis of his great experience and sure instincts, Professor Seed proposed a number of simple rules in this synthesis; our finite element models have since provided resounding confirmation of their validity.
It should be noted that it is rare for seismic design calculations to result in a significant increase in reinforcements in a Reinforced Earth structure. However, this design method allows us to make such decisions, where advisable, for particularly earthquake-prone regions with high acceleration coefficients, or in the case of structures with special geometry or loading conditions.

2. GENERAL

2.1 Forward

As is customary, the design method distinguishes between the verification of safety factors for external stability and those relating to internal stability. Verification of safety factors with respect to sliding and overturning for external stability will follow relevant rules and regulations set forth in the 1994 AASHTO Interim Specifications for design of highway bridges.

The method for calculating internal stability, also outlined in the 1994 AASHTO Interim Specifications, is based on a specific analysis of the behavior of Reinforced Earth structures exposed to seismic forces. It must therefore be strictly adhered to, totally disregarding calculation methods developed for other types of structures.

2.2 Dynamic forces - Definitions

Dynamic forces, or more accurately, pseudo-static forces play a role in these calculations. The type of pseudo-static force to be considered depends on whether one is concerned with external stability or internal stability.

2.2.1 External Stability (Figure 1)

From the applied horizontal seismic accelerations, two supplementary horizontal forces develop:

\[ P_{ae} = \text{an increase in pressure from the earth retained by the structure.} \]

\[ P_{ir} = \text{an overall inertia load, proportional to the weight of the effective Reinforced Earth mass.} \]

An upward or downward variation in the weight of the structure is possible due to vertical accelerations. However, the vertical accelerations are considered secondary compared to the horizontal accelerations and are therefore generally ignored.
In the paper delivered in 1970 at the ASCE Specialty Conference, Professors H. Bolton Seed and Robert V. Whitman stated:

"Since for most earthquakes the horizontal acceleration components are considerably greater than the vertical acceleration components, it seems reasonable to conclude that in such cases the influence of the vertical acceleration component $K_v$ can be neglected for practical purposes."

2.2.2 Internal Stability (Figure 2)

Only one supplementary horizontal force is included:

$P_i$ = an internal dynamic force, the sum, in fact, of the additional tensile forces occurring in the reinforcing strips which is simply equal to the inertia of the active zone.

2.3 The Accelerations to be Taken Into Account

- The dynamic or pseudo-static forces are functions of $A_m$, the average maximum horizontal acceleration occurring in the Reinforced Earth structure and the ground behind the structure (the term "maximum" is with respect to time, while "average" relates to the height of the structure).

- The acceleration $A_m$ is related to the maximum horizontal acceleration $A$ which is presumed to occur at the level of the free surface of the natural ground at the site, for a given earthquake and class of risk.

This acceleration $A$ (known as the "free field" acceleration), having been somewhat influenced by the presence of the Reinforced Earth structure on the site, becomes gradually greater towards the surface of the reinforced backfill (Figures 3a and 3b). On average, the greater the acceleration $A$ the less pronounced the amplification with height. In practical terms, for any site where:

$$0.05 < A < 0.45$$

the average maximum horizontal acceleration, $A_m$, in the Reinforced Earth structure and the ground behind can be taken as:

$$A_m = (1.45 - A)A$$ (Figure 4)
The free field acceleration "A" is a function of the structure's location with respect to an active fault and the nature of the foundation soils. If the value of "A" is not indicated by the owner or their agent, the value can be assumed as the acceleration coefficient "A" obtained from figure 1-5 of the 1991 AASHTO interim specifications for highway bridges (See appendix). Note, the accelerations given on the contour map are expressed as percent of gravity. Therefore, these values must be divided by 100 to obtain the decimal percent acceleration to be used in the design calculations.

Figure 1: External Stability, Supplementary Forces

Figure 2: Internal Stability, Supplementary Force
Figure 3a: Maximum accelerations within and behind the Reinforced Earth volume, 19.7 ft. wall (Superflush)

Figure 3b: Maximum accelerations within and behind the Reinforced Earth volume, 34.5 ft. wall (Superflush)
2.4 Load Combination

Seismic loads are generally considered to be accidental in nature, with a single degree of aggressiveness and no load factor. The combined loads to be taken into account when verifying the stability of the structure, both externally and internally, fall under AASHTO service load group VII. Group VII considers dead load, earth pressure, buoyancy, stream flow pressure, and the earthquake forces. Live loads are not considered in a seismic analysis.

Table 3.22.1a and the applicable text, of the AASHTO standard specifications for highway bridges are presented in the appendix for reference.

\[ A_n = (1.45 - \lambda)A \]

![Graph showing \( A_n \) vs. \( \lambda \)]

Figure 4: Average maximum acceleration, \( A_n \), depending on the "free field" acceleration, \( \lambda \)
2.5 Factors of Safety and Allowable Stress

Increased allowable stress and reduced factors of safety are acceptable during seismic events due to the temporary nature of the loading condition. It is generally acceptable to allow 133% of the allowable static stresses and 75% of the required static safety factors for dynamic conditions associated with an earthquake event.

<table>
<thead>
<tr>
<th>External Stability</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>F.S. with respect to base sliding:</td>
<td>1.5</td>
<td>1.1</td>
</tr>
<tr>
<td>F.S. with respect to overturning:</td>
<td>2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>F.S. with respect to bearing capacity:</td>
<td>2.0</td>
<td>Note 1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Internal Stability</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcement Tensile Stress:</td>
<td>0.55 $F_y$</td>
<td>0.73 $F_y$</td>
</tr>
<tr>
<td>(see note 2)</td>
<td>(36 ksi)</td>
<td>(48 ksi)</td>
</tr>
<tr>
<td>F.S. with respect to bond of Reinforcing Strips:</td>
<td>1.5</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Note 1: A factor of safety of 2.0 with respect to foundation bearing capacity is considered acceptable for static conditions. Eccentricity of the structure and applied bearing pressure are not determined during a seismic event due to the temporary and transient nature of the loading condition. Bearing pressure at the toe of the structure during a seismic event should not vary appreciably from the static case. However, this commentary shall serve as a reminder that it may be necessary to check that an earthquake will not alter the inherent strength characteristics of the foundation soils.

Note 2: The reinforcement tensile stress presented above is the allowable reinforcement tensile stress at the end of the design service life. At time zero, the allowable tensile stress is considerably less to allow for a minimum sacrificial reinforcement thickness of 1.42mm for a 75 year service life and 1.77mm for a 100 year service life.

3. EXTERNAL STABILITY

3.1 Seismic coefficients

Two "seismic coefficients", $K_h$ and $K_v$, must be defined before the dynamic horizontal thrust, $P_{ae}$, and the structure's inertia load, $P_{ir}$ can be calculated. These coefficients are applied simultaneously and uniformly to all parts of the structure, i.e. to the retaining structure itself and to the ground behind the structure.
For gravity structures such as Reinforced Earth, the values assigned to these coefficients are:

\[ K_h = A_m \]
\[ K_v = 0.5K_h = 0.5A_m \]

The value selected for the seismic coefficient, \( k_h \), equal to the average maximum horizontal acceleration, \( A_m \), should be conservative. The use of one-half the dynamic thrust, \( 0.5P_{ae} \), as shown in Figure 1 takes into account the fact that particle acceleration is not at its maximum everywhere at the same moment, either in the wall, or in the ground it retains, and that some small horizontal displacement leading to stress release is acceptable. This is consistent with the recommendations of Professors Seed and Mitchell in their report, *Earthquake Resistant Design of Reinforced Earth Walls*, dated December 1981.

### 3.2 Determining the Dynamic Horizontal Thrust, \( P_{ae} \)

The additional dynamic horizontal thrust, \( P_{ae} \), has the effect of increasing the static force, \( P \). Stability computations shall be made by considering, in addition to static forces, the horizontal inertial force (\( P_{ir} \)) acting simultaneously with 50 percent of the dynamic horizontal thrust (\( 0.5P_{ae} \)). The dynamic horizontal thrust \( P_{ae} \) shall be evaluated using the pseudo-static Mononabe-Okabe method and shall be applied to the vertical rear boundary of the effective reinforced earth mass at a height of 0.6\( H \) from the base and the horizontal inertial force shall be applied at mid-height of the structure.

To find \( P_{ae} \), we use the Mononabe-Okabe formula:

\[ P_{ae} = \frac{1}{2} \cdot 2\cdot (K_{ae} - K_a) \]

where:

\[ ?K_{ae} = (1-K_v)K_{ae} - K_a \]

\( K_{ae} \) is a total earth pressure coefficient, including the seismic effect, and \( K_a \) is the static earth pressure coefficient. By subtracting \( K_a \) from \( K_{ae} \), we obtain \(?K_{ae}\) which represents the incremental increase in the earth pressure due to the earthquake event.

Calculation of the total earth pressure coefficient, \( K_{ae} \), for a vertical wall, using the Mononabe-Okabe equation is as follows:

\[ K_{ae} = \frac{\cos^2 (\phi - \theta)}{\cos \theta \cos (\delta + \theta)} \left[ 1 + \frac{\sin (\phi + \delta) \sin (\phi - \theta - i)}{\cos (\delta + \theta) \cos i} \right]^{-2} \]
If \( i > (\phi - \theta) \) then \((\phi - \theta - i)\) is assumed to be zero. The above relationship becomes:

\[
K_{ae} = \frac{\cos^2(\phi - \theta)}{\cos \theta \cos(\delta + \theta)}
\]

Calculation of the static earth pressure coefficient, \( K_a \), for any backfill slope angle, \( i \), is:

\[
K_a = \cos i \left[ \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} \right]
\]

where:

\( \phi \) = Angle of internal friction of the soil
\( \theta \) = \( \arctan \left( \frac{K_h}{1 - K_v} \right) \)
\( d \) = Angle of friction between soil and structure
(Note for standard RE design, \( d = i \))
\( k_h \) = horizontal seismic coefficient
\( k_v \) = vertical seismic coefficient
\( i \) = backfill slope angle

Neglecting vertical accelerations in accordance with section 2.2.1

\( \theta = \arctan \frac{K_h}{K_v} = \arctan A_m \)

and \( \theta \) \( K_{ae} = K_{ae} - K_a \)
3.2.1 **Vertical Wall With Horizontal Backfill (figure 5)**

For a vertical wall, with a horizontal backfill having an angle of internal friction of 30°, a free field acceleration equal to 0.4g, the value of $P_{ae}$ may be calculated as follows:

$$P_{ae} = 0.375 \times H^2 A_m$$

For other accelerations or for materials of differing shear strength, the value of $P_{ae}$ may be calculated by computing the difference between $K_{ae}$ and $K_a$ to determine the seismic earth pressure coefficient, $\delta K_{ae}$. Therefore, the value of $P_{ae}$ may be calculated as follows:

$$P_{ae} = \frac{1}{2} \times H^2 \times K_{ae} = \frac{1}{2} \times H^2 (K_{ae} - K_a)$$

In either case, one-half of the resultant dynamic thrust, $0.5P_{ae}$, is applied horizontally at 0.6H above the base of wall as shown in figure 5.

![Figure 5: External stability - level surcharge condition](image-url)
3.2.2 Vertical Wall With Sloping Backfill (Figures 6a and 6b)

For vertical walls with sloping backfill, the resultant seismic force, \( P_{ae} \), is always calculated by working out the difference between \( K_{ae} \) and \( K_a \) to determine the seismic earth pressure coefficient, \( ?K_{ae} \). The procedure allows for the actual shear strength and slope angle of the soil being retained.

One-half of the resultant seismic force, \( 0.5P_{ae} \), is applied at \( 0.6H^2 \) above the base of wall, acting parallel to the actual infinite slope or equivalent infinite slope at an angle of \( i \) with respect to the horizontal.

3.3 Effective Inertia Force \( P_{ir} \)

The effective inertia force, \( P_{ir} \), is a horizontal load acting at the center of gravity of the effective mass. For a horizontal backfill condition (Figure 5), with \( W \) being the total weight of the effective mass, the effective inertia force is equal to:

\[
P_{ir} = K_h W = 0.5 \times H^2 A_m
\]

For a sloping surcharge condition (Figures 6a and 6b), the supplementary inertia force, \( P_{is} \), caused by any soil situated above the effective mass shall be included in the computation. Therefore, the total inertia force becomes:

\[
P_{ir} + P_{is} = K_h(W + W_s) = 0.5 \times H_2 A_m [H_1 + 0.5 (H_2 - H_1)]
\]

where:

\[
H_2 = H_f + \frac{0.5H_2 \tan i}{1 - 0.5 \tan i}
\]

In either case, the weight of the facing panels is omitted from the calculations as in the case for routine static stability calculations.
3.4 Performing the External Stability Calculations

The static stability of the structure is determined as normal, utilizing the minimum reinforcement lengths necessary to satisfy the required factors of safety for sliding, overturning and bearing, including a check of structure eccentricity (see section 2.5). In addition, the minimum reinforcement length for static stability should satisfy the minimum reinforcement length requirements of the project specifications.

The static thrust, \( P \), is applied to the imaginary vertical rear boundary at the end of the reinforcements as shown in figures 5, 6a and 6b. Next, it is necessary to determine the geometry of the effective mass of the structure for the dynamic condition, which extends a distance of 0.5 \( H_2 \) behind the wall facing. Then, one-half of the dynamic thrust, \( 0.5P_{ae} \), is applied to the imaginary vertical rear boundary at 0.5 \( H_2 \) behind the wall facing acting simultaneously with the inertia of the effective mass, \( P_{ir} \) and \( P_{is} \), if applicable. The dynamic forces are in addition to the static force used to determine the minimum reinforcement length required for static stability. See figures 5, 6a and 6b.

If the reinforcement length is required to be increased for adequate stability during the dynamic condition, the applied thrusts, \( P \), \( 0.5P_{ae} \), \( P_{ir} \) and \( P_{is} \) are **NOT** changed. Only the resistance of the reinforced mass is increased as required to achieve the required stability safety. This procedure is logical since there is no reason for the applied thrusts from the embankment to increase just because the reinforcements get lengthened.
Figure 6a: External Stability - Infinite Slope Condition

*NOTE:* The reinforcement length, \( L \), may need to be increased for stability, however the applied thrusts do *NOT* increase and remain applied to their respective imaginary vertical boundaries as shown.

Figure 6b: External Stability - Broken Back Slope Condition
4. INTERNAL STABILITY

4.1 The Internal Dynamic Load, $P_i$

The internal dynamic load, $P_i$, which is distributed among the reinforcing strips and is added to the static tensile forces, is equal to the weight of the actual active zone (not the bilinear approximation), including any additional soil surcharge on top, multiplied by the average maximum horizontal acceleration, $A_m$.

Since calculations are generally performed using the bilinear envelope (Figure 7a) and not the actual active zone consisting of soil located inside the actual line of maximum tension (potential failure surface), a correction factor of 0.67 is required to adjust the volume of the active zone in the calculations.

For example, let $W_a$ be the weight of fill in the bilinear active zone envelope (figure 7a), the internal dynamic load, $P_i$, becomes:

$$P_i = 0.67W_a A_m$$

The geometry of the actual active zone, as verified by the dynamic F.E.M. results, is identical to that for static calculations. In the case of a basic structure with no additional soil surcharge load, the active zone envelope volume, $V_a$, is as shown in figure 7b and is equal to:

$$V_a = 0.75 (0.3H \times H) = 0.225H^2$$

Therefore, the internal dynamic load, $P_i$, becomes:

$$P_i = 0.67 \times 0.225H^2 \times A_m = 0.15H^2A_m$$

4.2 Distribution of Dynamic Load $P_i$ Among the Reinforcing Strips

The dynamic load, $P_i$, is added to the maximum tensile forces, $T_m$, induced in the reinforcing strips by static loads, i.e.: the structure's own weight, applied static earth pressure, and the supplementary loads and pressures due to any dead load surcharge. The other loads of dynamic origin, $0.5P_{ae}$, or $P_{ir}$, are not taken into account in the calculation of the maximum tensile force $T_m$ (figure 8a).

The dynamic load, $P_i$, is distributed among the individual reinforcing strips in proportion to their "resistant area", obtained by multiplying their width times their embedment length in the resistant zone.
Figure 7a: Internal Stability - Sloping Condition

\[ W_a = 0.225 \gamma H'^2 - 0.15H'h \gamma \]
\[ P_1 = 0.67 W_a A_a \]

--- line of maximum tension
--- active zone envelope

--- line of maximum tension
--- active zone envelope

Figure 7b: Internal Stability - Level Condition

\[ W_a = 0.225 \gamma H^2 \]
\[ P_1 = 0.15 \gamma H^2 A_a \]
Figure 8a: Internal Stability - Loads Included in the Calculation of $T_2$

Figure 8b: Distribution of Dynamic Load Among the Strips
Thus in layer J (figure 8b), a reinforcing strip of width $b_j$, having a resistant length $L_{aj}$, the static tensile force, $T_m$, will be increased by an increment of the total dynamic load, $\Delta T_d$ equal to:

$$
\Delta T_d = \left[ \frac{b_j L_{aj}}{i = N} \right] \sum_{i=1}^{N} n_i b_i L_{ai} \times 9.84 \text{'}
$$

Where $n$ is the number of reinforcing strips across two columns of panels (9.84') in layer $i$, and $N$ is the total number of layers of reinforcing strips in the section of structure under investigation (figure 8b).

Therefore, the maximum tensile force in a reinforcing strip during the dynamic event becomes:

$$
T_{dm} = T_m + \Delta T_d
$$

4.3 Comparison of Calculated Dynamic Increment of Tensile Loads With F.E.M. Results

Figures 8c and 8d present a comparison of the maximum dynamic increment of tensile loads calculated by the above procedure with those determined in the dynamic finite element study. The 19.7 foot high wall (Figure 8c) and the 34.5 foot high wall (figure 8d) consist of vertical walls founded on rock subjected to the 1957 Golden Gate Accelerogram. Three peak rock accelerations, 0.1g, 0.2g and 0.4g were examined.

The Reinforced Earth Backfill material was assigned a unit weight of 125 pcf, a shear strength of 36 degrees and no cohesion. The random backfill material being retained by the Reinforced Earth structure was assigned a unit weight of 125 pcf, a shear strength of 30 degrees and no cohesion.

The facing panels consisted of 7 inch thick, discrete facing panels, 4.92 feet in height, with a unit weight and strength representative of reinforced concrete.

The maximum dynamic increment of tensile loads, as determined utilizing the following equation, is conservative with respect to the F.E.M. results:

$$
P_i = 0.15 ? H^2 A_m
$$
The calculation procedure, which takes into account only the inertia of the soil within the actual active zone is compared to finite element results which include the inertia of the facing panels. Therefore, based on this conservatism, there is no need to include the facing panel weight in the calculations.

Also note in figures 8c and 8d that the level of conservatism of the calculated dynamic increment with respect to the F.E.M. results increases with increasing peak foundation acceleration. In other words, increased conservatism will be provided in structures located in more seismically active areas of the country, having higher acceleration coefficients.

4.4 **Tension at the Reinforcing Strip Connection to the Facing**

The magnitude of tension at the reinforcing strip connection to the facing is a function of the maximum reinforcement tension at the potential failure surface and the facing type.

We know from previous studies that if the facing consists of flexible steel elements, or wire, for example, the static tension at the connection, $T_{o}$, is equal to 75 percent of the maximum reinforcement tension, $T_m$, over the full height of wall.

When discrete concrete facing panels, approximately 5 foot by 5 foot in dimension are used, the ratio of $T_{o}/T_m$ is 85 percent from the top of the wall to a depth of 60 percent of the wall height and then increases linearly to 100% at the toe of wall.

When full height facing panels are used, the static tension at the connection is equal to the maximum tension over the full wall height.

We have learned from dynamic finite element studies that the dynamic increment of tensile force is also less at the connection in comparison to the maximum dynamic increment, $\Delta T_d$.

Therefore, at the facing, if the static tensile force at the connection is $T_o$ and the maximum tensile force is $T_m$, we can calculate the total force at the connection including the superimposed dynamic load, $\Delta T_d$, as follows:

$$T_{do} = T_o \left(\frac{T_m}{T_m} + \Delta T_d\right)$$

Since the connection of the reinforcement to the facing is specifically designed to be stronger than the gross section of the reinforcement (with allowance for sacrificial metal thickness), it will **NOT** control the number of reinforcements needed in the wall. The maximum reinforcement tension occurring at the line of maximum tension (or potential failure surface) will be compared with the allowable reinforcement tension for the static and dynamic condition.
Therefore $T_m + ?T_d$ must be less than or equal to 73 percent of the yield stress of the steel times the reduced cross sectional area of the reinforcement (section 2.5).

$$T_m + ?T_d \leq 0.73 \ F_y \times A_{rs}$$

4.5 **Reinforcing Strip Pull-out Resistance During Earthquakes**

A series of pullout tests were performed on a full scale test wall subjected to vibrations. The vibrations were induced by vibratory compaction equipment placed in a cradle at the top of wall.

Several pullout tests were performed in the presence of vertical vibrations more severe than an earthquake would impose. Vertical accelerations ranged from 0.2g to 1.2g during the pullout tests. The test results show a maximum 20 percent reduction in the pullout resistance, $R$, of the reinforcing strips for vertical accelerations that may be considered typical for earthquake events. This reduced pullout resistance is not due to a reduction in the friction coefficient between the reinforcing strips and soil, but, is due to reduced vertical stress (overburden) on the strips caused by the vertical accelerations.

Therefore, for convenience in the analysis of Reinforced Earth structures considering earthquake effects, a 20 percent reduction of the calculated static pullout resistance of the reinforcing strips will be used for the dynamic pullout resistance to conservatively take into account any reduced vertical stress on the strips due to vertical accelerations inherent in earthquake events.

$$R_{seismic} = 0.8 \ R_{static}$$

As we have already seen, the width of the active zone is not dependent on $A_m$. Therefore, for each reinforcing strip level, adherence is checked over the usual length as in the static condition. The calculated factor of safety with respect to bond is compared with the allowable safety factor for the seismic condition (Section 2.5).
Figure 8c: Maximum dynamic increment of tensile loads 19.7 ft. wall (Superflush)

Figure 8d: Maximum dynamic increment of tensile loads 34.5 ft. wall (Superflush)
PART B - BRIDGE ABUTMENTS

5. INTRODUCTION

In general, there has not been much research done to investigate the performance of bridge abutments subjected to earthquake loads. Actual in-service performance data is also scarce due to the fact that fewer abutments have experienced earthquakes.

A stub abutment constructed on Reinforced Earth will perform in a similar manner to a stub abutment supported on an embankment. The Reinforced Earth walls provide vertical boundaries to replace the slopes that are required for embankment construction. Case histories of Reinforced Earth bridge abutments subjected to earthquakes confirm this analogy.

In 1983, an unexpected 5.0 Richter magnitude earthquake struck the Belgian town of Liege. The epicenter was located at a shallow depth of only 2 to 3 miles and therefore, the earthquake developed extremely destructive forces within the local area. Many homes and shops within 7 miles of the epicenter were destroyed or damaged by the quake. There are two small Reinforced Earth walls approximately 1/2 mile from the epicenter where ground accelerations reached 0.15 to 0.20g. No damage or deformation occurred in either wall. Two miles from the epicenter, in an area where many houses were damaged, a large wall with a total surface area of 180,000 sq. ft., which includes three Reinforced Earth bridge abutment sections, was not damaged.

In 1987, a 6.3 Richter magnitude earthquake occurred in the Plenty North Bay of New Zealand. Many homes and buildings were severely damaged in the nearby town. In Maniatutu, less than 20 miles from the epicenter, a Reinforced Earth bridge abutment was nearing completion. The prefabricated elements of the deck, already placed but not yet secured, began "dancing" on their supports causing the workers to flee. Backfilling of the Reinforced Earth abutments had not reached its final level and therefore only limited reinforcing strip adherence was available to support the bridge structure. Nevertheless there was no damage or noticeable deformation of the Reinforced earth abutments.

There two case histories confirm that Reinforced Earth abutments, like Reinforced Earth retaining walls, perform well during earthquakes!
6. **GENERAL**

6.1 **Forward**

Research to date with respect to the behavior of Reinforced Earth structures subject to earthquakes has been limited to retaining walls.

The design method presented in the following sections for Reinforced Earth bridge abutments has been developed as an extension of the design method for retaining walls.

6.2 **Accelerations to be Taken Into Account**

Whether verifying external or internal stability, dynamic forces connected with the Reinforced Earth mass, those relating to the beam seat, and those coming from the bridge superstructure, must be accounted for separately.

The "dynamic" loads from the deck are calculated and supplied by the bridge designer, along with the static bridge loadings. In practice they are expressed in terms of $A$, the site's maximum "free field" acceleration for the earthquake and class of risk under consideration.

Obviously the abutment walls, beam seats, and deck together form a single structure, the bridge. Therefore, there is no reason to infer that abutments be assigned a higher class of risk, or have a greater reference acceleration than the bridge deck itself.

As a result, the average maximum acceleration assigned to the Reinforced Earth volume supporting the beam seat continues to be a function of the free-field acceleration, $A$, as follows:

$$A_m = (1.45 - A) A$$

For the beam seat, its role in the analysis depends on whether we are concerned with its own inherent stability, or with the fact that it is included in the overall stability of the abutment.

With respect to its own stability, the beam seat should be treated as a gravity wall, being assigned seismic coefficients $K_h$ and $K_v$.

Given, however, that little is known of the actual accelerations reaching the beam seat at the top of the structure, we shall verify its stability using the free field acceleration, $A$.

With respect to overall stability of the abutment, the beam seat is considered an integral part of the Reinforced Earth abutment structure and will be analyzed using the same assumptions as the Reinforced Earth volume.
6.3 Combined loads and safety factors

The "dynamic" bridge loads from the superstructure must be divided into supplementary loads (vertical and Horizontal), due to dead loads and due to traffic loads. In accordance with AASHTO Standard Specifications for Highway Bridges, live loads are omitted in the analysis of seismic stability. However, it is probable that there will be live load on the bridge during an earthquake. For example, it was rush hour in San Francisco during the Loma Prieta earthquake in October 1989. It is unlikely, however, that the maximum live load condition (fully loaded trucks) will coincide with the seismic event. Therefore, we will assume that 50% of the maximum live load is applied during the seismic event. This will conservatively represent live load conditions associated with rush hour automobile traffic.

The effect of thermal shrinkage of the deck is not considered in addition to the earthquake effects.

The safety factors and allowable stress levels for the abutment structure during the seismic event are the same as for retaining walls (Section 2.5).

7. EXTERNAL STABILITY

7.1 Method of Calculation

Verifying external stability is a two-stage operation. The first stage examines the stability of the beam seat with respect to forward sliding and overturning. the second stage verifies the stability of the Reinforced Earth volume. Since the forces involved are not exactly identical, the two calculation processes are presented separately in the following sections.

7.2 Beam Seat Stability (Figure 9)

7.2.1 Loads Transmitted From the Bridge Deck

For beam seat stability calculations, the safety factor with respect to sliding and overturning is checked considering only the dead load, Q_d, of the bridge and the horizontal inertia of the deadload, F_d. the inertia of the deadload, F_d, is calculated as follows and is applied at the location of bearing.

\[ F_d = Q_d A \]

For the beam seat bearing pressure calculation and surcharge effect for internal stability, the dead load, Q_d, plus 50% of the live load, 0.5Q_l, are applied vertically, while the inertia of the dead load and live load are applied horizontally. The inertia of the deadload plus live load, F_{d+l}, is calculated as follows:
\[ F_{d+l} = (Q_d + 0.5Q_l)A \]
7.2.2 Forces Arising From the Beam Seat Itself

The beam seat, including its backwall and the backfill over its heel, has a total weight, $W_s$. The resultant inertia force from the beam seat weight is:

$$P_{is} = W_s A$$

7.2.3 Forced Transmitted From the Backfill

When considering the stability of the beam seat, the static and dynamic pressures exerted directly on the seat and its backwall, by the backfill overlying the Reinforced Earth volume, are taken into account. The value of the dynamic force is calculated for the acceleration $A$.

These forces include:

- the static earth pressure $P_2$ (horizontal);
- the static pressure $P_{2q}$ due to the (reduced) surcharge;
- the pseudo-static pressure, $P_{aes}$, calculated using the Mononobe-Okabe formula;

$$P_{aes} = \frac{1}{2} \alpha H_s^2 (K_{ae} - K_a)$$

Where $K_{ae}$ is calculated using:

$$\alpha = \arctan A \quad \text{(See Section 3.2)}$$

The reduced traffic surcharge must also be incorporated into the total dynamic earth pressure. Therefore, the total dynamic earth pressure applied at $0.6H_s$ above the base of the beam seat is:

$$P_{arc} \left[ 1 + \frac{P_{2q}}{P_2} \right]$$

7.3 External Stability of the Reinforced Earth Mass (Figure 10)

7.3.1 Forces Transmitted From the Deck

Only dead load, $Q_d$, and the inertia of the dead load, $F_{id}$, are considered in the external stability calculation. Live loads, if included in the calculation, would have a tendency to increase the safety factor with respect to sliding of the mass and would have little or no effect on overturning.
The inertia of the deck dead load, $F_d$, is calculated based on the free field acceleration, $A$.

$$F_d = Q_d A$$

### 7.3.2 Forces Transmitted From the Bridge Seat Itself:

When computing the overall stability of the Reinforced Earth abutment, the beam seat, including its backwall and the backfill over the heel, are considered an integral part of the Reinforced Earth structure. Therefore, as for the reinforced volume, the inertia of the beam seat is calculated using the acceleration $A_m$, where,

$$A_m = (1.45 - A)A$$

Thus the inertia of the beam seat for the stability calculation of the Reinforced Earth mass becomes:

$$P_{is} = K_h W_s = A_m W_s$$

### 7.3.3 Inertia Forces of the Reinforced Earth Volume:

As in the case of retaining structures, if $W$ denotes the effective weight of the reinforced earth mass and $W2$ the effective weight of the overlying fill, then we assume an inertia force at the center of gravity of each weight equal to:

$$P_{ir} = K_h W = A_m W \quad \text{(Reinforced Earth Mass)}$$

and

$$P_{i2} = K_h W_2 = A_m W_2 \quad \text{(Overlying Fill)}$$

For mass stability calculations, the live load is once again omitted because if included the safety factor with respect to sliding and overturning would increase.

### 7.3.4 Forces Transmitted From the Backfill Behind the Structure:

As in a Reinforced Earth retaining wall, the static earth pressure, $P$, is increased by one-half of the horizontal dynamic pressure, $0.5P_{ae}$ exerted at a level, $0.6H$ above the base.
Given the usually shallow slope of abutment approach embankments, a horizontal backfill condition may be assumed for the calculation of the dynamic earth pressure, $P_{ae}$, in accordance with Section 3.2.1, with:

$$A_m = (1.45 - A)A$$

7.3.5 External Stability Calculations

The verification of safety factors with respect to sliding and overturning are then carried out following the same principles that apply to static conditions.

As in the case of retaining walls, the eccentricity and bearing pressure under the Reinforced Earth mass is not calculated because a seismic event is a temporary and transient loading condition on a very flexible system. Therefore, bearing pressures at the foundation level should not increase appreciably during the seismic event (Section 2.5).
Figure 10: External Stability of Abutment
8. INTERNAL STABILITY

8.1 Method of Calculation

As in the case of retaining walls, internal stability calculations are done in two stages.

The first stage involves calculating, by the usual methods, the tensile forces resulting from the application of static loads alone.

In the second stage, an overall internal dynamic load, $P_i$, connected with both the reinforced mass itself and the concentrated load transmitted by the beam seat is calculated. The load, $P_i$, is then distributed among the strips in proportion to their resistant area, and added to the tensile load calculated in the static case.

8.2 Loads considered in the Calculation of $P_i$ (Figure 11)

The dynamic force, $P_i$, is directly connected with the "active zone" of the Reinforced Earth volume, through its own weight and the load it carries. As in the case of retaining walls, to take into account the weight of the actual active zone, the weight of the idealized (bilinear) active zone envelope must be multiplied by the coefficient 0.67 to obtain the weight of the actual active zone. The applied load from the bridge seat is directly added to obtain the total vertical load. The total vertical load is then multiplied by the acceleration $A_m$ to obtain the dynamic Force, $P_i$, to be distributed among the reinforcing strips. The weight of the active zone envelope is a function of the geometry of the structure and the beam seat.

The diagrams in figure 11 show the three main configurations that the active zone envelope may resemble. The active zone may include part of the fill behind the beam seat, in which case a reduced surcharge, $Q'_1$, acting at the roadway surface, over the width concerned, shall be taken into account.

The load borne by the active zone is a combination of the vertical bridge loads, consisting of the dead load, $Q_d$, and 50% of the live load, $0.5Q_l$, and the weight of the beam seat, $W_s$, which includes the backwall and soil above the heel.

Thus:

$$P_i = [(0.67W_a + Q'_1) + (Q_d + 0.5Q_l + W_s)] A_m$$
line of maximum tension

active zone envelope

Figure 11: calculating Internal Dynamic force, $P_I$, in Different Cases
8.3 Verification of Internal Stability

The load, $P_i$, is distributed among the strips in proportion to their resistant area, and added to the tensile load calculated in the static case. Reinforcement tensile stresses and adherence safety factors are calculated by conventional procedures and checked against the allowable values presented in Section 2.5 for retaining walls.

9. CONCLUSION

The practical design method presented in this report is the result of synthesizing research conducted by The Reinforced Earth Company with the assistance of leading experts in the field of earthquake engineering.

The design method with respect to external stability is in accordance with recommendations set forth in the 1994 AASHTO Interim Specifications for highway bridges, utilizing the widely accepted Mononobe-Okabe method of analysis.

The design method with respect to internal stability follows simple rules proposed by the late Professor Seed which are specific to Reinforced Earth technology. The validity of these rules have been confirmed by dynamic F.E.M. analyses conducted in 1989.

This document presents the state of the art design method recommended by The Reinforced Earth Company for the design of Reinforced Earth walls and bridge abutments in seismically active areas.
REFERENCES


reinforcements. The upper two rows of reinforcement shall be designed for an additional horizontal load of 300 pounds per linear foot of wall.

5.8.10 Seismic Design

5.8.10.1 External Stability

Stability computations shall be made by considering, in addition to static forces, the horizontal inertial force (P_{nx}) acting simultaneously with 50 percent of the dynamic horizontal thrust (P_{nx}). The dynamic horizontal thrust P_{nx} shall be evaluated using the pseudo-static Mononobe-Okabe method and shall be applied to the back surface of the reinforced fill at a height of 0.6H from the base and the horizontal inertial force at the mid-height of the structure. Values of P_{nx} and P_{in} for structures with horizontal backfill, may be determined using the following:

\[ A_{nx} = (1.45 - A)A \]  
(5.8.10.1.1)  

\[ P_{nx} = 0.375A_{nx}H^2 \]  
(5.8.10.1.2)  

\[ P_{in} = 0.5A_{nx}H^2 \]  
(5.8.10.1.3)

For structures with sloping backfills, the inertial force (P_{nx}) shall be based on an effective mass having height \( H_2 \) and a base width equal to 0.5 \( H_2 \) determined as follows:

\[ H_2 = H + \frac{\text{Tan} \beta \times 0.5 \ H}{(1 - 0.5 \ \text{Tan} \beta)} \]  
(5.8.10.1.4)

The inertial force (P_{nx}) shall be taken to act simultaneously with one-half the dynamic horizontal thrust P_{nx}, computed using the pseudo-static Mononobe-Okabe method, and applied at 0.6 \( H_2 \) above the base on the back surface of the effective mass.

Factors of safety against sliding and overturning failure under combined loading may be reduced to 75 percent of the factors of safety defined in Article 5.5.5.

5.8.10.2 Internal Stability

Reinforcements shall be designed to withstand horizontal forces generated by the internal inertia force (P_{i}) in addition to the static forces. The total inertial force P_{i} per unit length of structure shall be considered equal to the mass of the active zone times the maximum wall acceleration coefficient \( A_{nx} \). This inertial force shall be distributed to the reinforcements proportionally to their resistant areas as follows:

\[ T_{nx} = P_{i} \left( \frac{b_1L_{oz}(S_{oz})}{\sum b_1L_{oz}(S_{oz})} \right) \]  
(5.8.10.2.1)

For seismic loading conditions, values of \( I^* \), \( N_a \) and \( f_a \) may be reduced up to 80 percent of the values used for static design. Factors of safety under combined static and seismic loads for pullout and breakage of reinforcement may be reduced to 75 percent of the factors of safety used for static loading.

5.8.11 Structural Requirements

Panels shall be designed to resist the horizontal forces calculated according to Articles 5.8.4.1 or 5.8.4.2. Reinforcement shall be provided to resist the average loading conditions for each panel. As a minimum, temperature and shrinkage steel shall be provided. Epoxy coating for corrosion protection of panel reinforcement where salt spray is anticipated is recommended.

5.9 PREFABRICATED MODULAR WALL DESIGN

5.9.1 Structure Dimensions

Prefabricated modular walls shall be dimensioned to ensure that the applicable factors of safety outlined in Article 5.5.5 are satisfied.

Minimum embedment and scour protection shall satisfy the requirements of Article 5.8.1.

5.9.2 External Stability

Stability computations shall be made by assuming that the system acts as a rigid body.

Lateral pressures shall be computed by wedge theory using a plane surface of sliding (Coulomb theory). Where the rear of the prefabricated modular systems forms an irregular surface (stepped modules), pressures shall be computed on an average plane surface drawn from the lower back heel of the lowest module to the upper rear heel of the top module, as shown in Figures 5.9.2A and 5.9.2B.

The following wall friction angles, \( \delta \), shall be used unless more exact coefficients are demonstrated:

<table>
<thead>
<tr>
<th>Case</th>
<th>Wall Friction Angle (( \delta ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Significant vibrations of backfill or modules settling more than backfill</td>
<td>0</td>
</tr>
<tr>
<td>(b) Continuous pressure surface of precast concrete (uniform width modules)</td>
<td>( 1/2 \delta )</td>
</tr>
<tr>
<td>(c) Averaged pressure surface (stepped modules)</td>
<td>( 3/4 \delta )</td>
</tr>
</tbody>
</table>

Computations for stability shall be made at every module level. At each level, the required factors of safety with respect to overturning shall be provided. The value of \( K_s \) used to compute the lateral thrust resulting from the random backfill and other loads shall be computed...
modified by the framing factor, $F$, may be used as the design response spectrum.

### 3.21.3 Special Cases

Structures adjacent to active faults, sites with unusual geologic conditions, unusual structures, and structures having a fundamental period greater than 3.5 seconds will be considered special cases. These structures will be required to be designed using current seismicity, soil response, and dynamic analysis techniques.

### 3.21.4 Design of Restraining Features

3.21.4.1 Restraining features to limit the displacement of the superstructure—i.e., hinge ties, shear blocks, etc.—shall be designed for the following force:

$$
EQ = 0.25 \times \text{contribution of DL minus column shears due to EQ}
$$

(3-9)

3.21.4.2 Contributing DL is determined by examining the entire frame. For example, a simple span fixed at one end and sliding at the other will have the entire superstructure as the contributing DL for longitudinal forces at the fixed abutment, while one half of the superstructure DL will act at each abutment for transverse forces.

3.21.4.3 For a frame, such as a 2-span structure, the full length of the bridge should be used as the contribution length in the longitudinal direction. The resulting force can be reduced by deducting the shear in the column due to earthquake.

3.21.4.4 For hinge restraints use $0.25 \times DL$ of the smaller of the 2 frames and deduct the column shear due to EQ.

### Part B

#### COMBINATIONS OF LOADS

### 3.22 COMBINATIONS OF LOADS

3.22.1 The following Groups represent various combinations of loads and forces to which a structure may be subjected. Each component of the structure, or the foundation on which it rests, shall be proportioned to withstand safely all group combinations of these forces that are applicable to the particular site or type. Group loading combinations for Service Load Design and Load Factor Design are given by:

$$
\text{Group (N) = } \gamma \beta_D \cdot D + \beta_L \cdot (L + I) + \beta_C \cdot CF + \beta_E \cdot E + \beta_B \cdot B + \beta_S \cdot SF + \beta_W \cdot W + \beta_W \cdot WL + \beta_L \cdot LF + \beta_R \cdot (R + S + T) + \beta_EQ \cdot EQ + \beta_{ICE} \cdot ICE
$$

(3-10)

where

- $N$ = group number;
- $\gamma$ = load factor, see Table 3.22.1A;
- $\beta$ = coefficient, see Table 3.22.1A;
- $D$ = dead load;
- $L$ = live load;
- $I$ = live load impact;
- $E$ = earth pressure;
- $B$ = buoyancy;
- $W$ = wind load on structure;
- $WL$ = wind load on live load—100 pounds per linear foot;
- $LF$ = longitudinal force from live load;
- $CF$ = centrifugal force;
- $R$ = rib shortening;
- $S$ = shrinkage;
- $T$ = temperature;
- $EQ$ = earthquake;
- $SF$ = stream flow pressure;
- $ICE$ = ice pressure.

3.22.2 For service load design, the percentage of the basic unit stress for the various groups is given in Table 3.22.1A.

The loads and forces in each group shall be taken as appropriate from Articles 3.3 to 3.21. The maximum section required shall be used.

3.22.3 For load factor design, the gamma and beta factors given in Table 3.22.1A are only intended for designing structural members by the load factor concept. The actual loads should not be increased by the factors given in the table when designing foundations (soil pressure, pile loads, etc.). The load factors are also not intended to be used when checking the foundation stability (safety factors against overturning, sliding, etc.) of a structure.

3.22.4 When long span structures are being designed by load factor design, the gamma and beta factors specified for Load Factor Design represent general conditions and should be increased if, in the Engineer's judgment, expected loads, service conditions, or materials of
Table 3.22.1A  Table of Coefficients $\gamma$ and $\beta$

<table>
<thead>
<tr>
<th>COL No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROUP</td>
<td>$\gamma$</td>
<td>$D$</td>
<td>$(L=1)_H$</td>
<td>$(L=1)_H$</td>
<td>CF</td>
<td>$E$</td>
<td>$B$</td>
<td>$SF$</td>
<td>$W$</td>
<td>$WL$</td>
<td>$LF$</td>
<td>$R=2+T$</td>
<td>EQ</td>
<td>ICE</td>
</tr>
<tr>
<td>I</td>
<td>1.0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>$D_E$</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>IA</td>
<td>1.0</td>
<td>1</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>IB</td>
<td>1.0</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>II</td>
<td>1.0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>III</td>
<td>1.0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>IV</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>V</td>
<td>1.0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VI</td>
<td>1.0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VII</td>
<td>1.0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>VIII</td>
<td>1.0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>IX</td>
<td>1.0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>X</td>
<td>1.0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
| **Percentage = $\frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$**

**For Service Load Design**

% (Column 14) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

$\beta_E = 1.00$ for vertical and lateral loads on all other structures.

For culvert loading specifications, see Article 6.2.

$\beta_E = 1.00$ and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20.

**For Load Factor Design**

$\beta_E = 1.3$ for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts.

$\beta_E = 0.5$ for lateral earth pressure when checking positive moments in rigid frames. This complies with Article 3.20.

$\beta_D = 1.0$ for vertical earth pressure

$\beta_D = 0.75$ when checking member for minimum axial load and maximum moment or maximum eccentricity. For

$\beta_D = 1.0$ when checking member for maximum axial load and minimum moment. 

For Group X loading (culverts) the $\beta_E$ factor shall be applied to vertical and horizontal loads.

$L + L_H$ - Live load plus impact for AASHTO Highway H or HS loading

$L + L_H$ - Live load plus impact consistent with the overload criteria of the operation agency.

* 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.
FIGURE 1-5

Map of Horizontal Acceleration, A (expressed as percent of gravity) in Rock with 90 Percent Probability of Not Being Exceeded in 50 Years. Variability in Attenuation of Loga Ground Motion \((O_2 = 0.62)\) and Log\(\text{10}\) Fault Rupture Length \((O_2 = 0.52)\) are Included in the Calculation.

Hawaiian and Puerto Rico are derived from Commentary Section 1.4.1, Figure C1-4.
