DISPLACEMENT BASED SEISMIC DESIGN OF RETAINING WALLS

Shamsher Prakash¹, Vijay K. Puri² and Sanjeev Kumar³

ABSTRACT

The safe design of retaining walls requires adequate factor of safety against sliding, overturning and bearing capacity failure. The lateral earth pressures are generally calculated by using Coulomb’s or Rankine’s method for the static case and Mononobe-Okabe’s method for the seismic case. In addition, the displacement of the retaining wall becomes another important consideration in earthquake prone areas as retaining wall have been observed to fail by displacement away from the backfill in several past earthquakes. The paper presents a simple but realistic 2-D model to calculate displacement of rigid retaining walls. The model accounts non-linear soil behavior by incorporating strain dependent stiffness and damping in soil. This model has been used to calculate the displacement of a typical retaining wall for several combinations of backfill and foundation soil conditions for Northridge and Loma-Prieta earthquakes.

Keywords: Walls, Retaining, Design, Seismic, Displacement.

INTRODUCTION

There are two approaches for design of retaining walls for earthquake loading, namely (a) the pseudo-static approach and (b) the displacement approach. The pseudo-static approach is a simple method of calculating the safety of the retaining wall. The lateral earth pressure for static case is commonly determined using either Rankine’s or Coulomb’s method. The earthquake induced forces on the retaining wall are generally computed using the modified Coulomb’s approach in which the earthquake force on the backfill is replaced by an equivalent static force. This is known as Mononobe-Okabe method. The safety factors against overturning, sliding and bearing capacity failure are then calculated for the static as well as for the case of earthquake loading. The design is considered safe if the safety factors satisfy the design criteria for the safety factors.

A solution for determination of static and dynamic active earth pressure for a c-φ soils was developed by Puri and Prakash (2011). The method includes the effect of cohesion in the soil, adhesion between the retaining wall and backfill, the inclination of the backfill, horizontal and vertical seismic coefficients, surcharge on the backfill, and the inclination of the wall face and the backfill.

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Although the Mononobe–Okabe method has been commonly used in practice, there are questions about the calculated values of active and passive pressures. Terzaghi (1943) had shown that the for active earth pressure, assumption of plane failure surfaces matches with experimental observations. For the case of passive pressure when the angle of wall friction ‘δ’ is more than $\phi/3$, the assumption of plane failure surfaces results in an overestimation of passive pressure. Morison and Ebeling (1995), Soubra (2000) and Kumar (2001) used curved failure surfaces in their analysis for dynamic passive pressures. Choudhury (2002) has provided solution for passive pressure for seismic conditions for a c-$\phi$ soil. Gazetas et al., (2005) have observed that Mononobe-Okabe gives conservative estimates of active earth pressure.

Atik and Sitar (2010) conducted a series of centrifuge tests to study the seismic behavior of cantilever type retaining walls and concluded that additional seismic earth pressure on cantilever retaining walls can be disregarded for ground accelerations below 0.4g. They also observed that the dynamic earth pressure and inertia force do not act simultaneously. This is a significant observation and in our opinion needs further investigation.

Choudhury et al (2004) presented a comparison of seismic earth pressures by various methods for a typical 6 m high retaining wall (for, $\phi = 34^\circ$, $c = 0$, $\delta = 17$, $\gamma = 17.3$ kN/m$^3$, $\alpha_h = 0.3$ and $\alpha_v = 0.3$) which is shown in Table 1. It is observed from this table 1 that the displacement analysis results in smaller design earth pressures compared to the pseudo-static case.

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<thead>
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<tbody>
<tr>
<td>Seismic active earth pressure (kN/m)</td>
<td>145.16</td>
<td>--</td>
<td>141.24</td>
<td>100.92</td>
<td>100.92</td>
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<tr>
<td>P.O.A of active force from base (m)</td>
<td>2.72</td>
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<td>2.0</td>
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<td>Seismic passive earth pressure (kN/m)</td>
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<td>P.O.A of passive force from base (m)</td>
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<td>Displacement (mm)</td>
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<td>--</td>
<td>100</td>
<td>100</td>
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<tr>
<td>Remarks</td>
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<td>D</td>
<td>D</td>
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P.O.A. – Point of Application, F – Force Based (Pseudo-Static), D- Displacement Based

**DISPLACEMENT OF RETAINING WALLS**

Retaining walls have failed during earthquakes by sliding away from the backfill or due to combined action of sliding and rocking displacements. Performance based design of the retaining walls in seismic areas must account for the likely displacements, the retaining wall may experience during an earthquake in addition to calculating the usual factors of safety against failure in bearing capacity, sliding and overturning. An important question that comes to mind is ‘how much should be the permissible displacement’? There is no definite answer to this question though some guidelines given below available based on experience or judgment (Huang, 2005).

**Eurocode (1994)**
Permissible horizontal displacement = 300.$a_{max}$ (mm),  $a_{max}$ = maximum horizontal design acceleration

**AASHTO (2002)**
Permissible horizontal displacement = 250.$a_{max}$ (mm)
Wu and Prakash (1996)
Permissible horizontal displacement = 0.02H, \(H\) = height of retaining wall
Failure horizontal displacement = 0.1H

JRTRI (1996)
Permissible differential settlement = 0.1-0.2m, (damage needing minor retrofit measures)
Severe differential settlement = > 0.2m (damage needing long term retrofit measures)

It may be noted from the above that Eurocode 8 (1994) and Wu and Prakash (1996) recommend using specified horizontal displacements of the retaining wall for evaluating its seismic performance. The Japanese Railway Technical Research Institute (1999) suggests the use of vertical differential settlement as the performance criterion which seems reasonable for traffic accessibility and retrofit purposes after the earthquake.

**MODEL FOR DISPLACEMENT CALCULATIONS**

A realistic model for estimating the dynamic displacement must account for the combined action of sliding and rocking vibrations and considering 1) Soil stiffness in sliding and rocking 2) Geometrical and material damping in sliding and rocking, and 3) non-linear coupling effects for stiffness and damping.

Rafnsson and Prakash (1991) developed a model for simulating the response of rigid retaining walls subjected to seismic loading. This model consisted of a rigid wall resting on the foundation soil and subjected to a horizontal ground motion and analyzed the problem as a case of combined sliding and rocking vibrations including the effect of various important parameters such as soil stiffness in sliding, soil stiffness in rocking, geometrical damping in sliding, geometrical damping in rocking, material damping in sliding, and material damping in rocking.

![Figure: 1. Force diagram of forced vibration of rigid retaining wall with submerged pervious backfill](Wu and Prakash, 2001)
Only dry backfill was considered and seismic ground motion was represented by an equivalent sinusoidal motion. This model was further modified to accommodate both (Fig. 1) the dry and submerged backfills (Wu, 1999, Wu and Prakash, 2001)

Several cases of 6.0 m high retaining walls were analyzed for typical cases of foundation soil condition varying from well graded gravel (GW) to silt (ML) and the backfill soil varying from silty gravel (GM) to poorly graded sand (SP). Ground motions corresponding to El Centro, Loma-Prieta and Northridge earthquakes were used in the analysis. Typical case of a reference retaining wall, 6.0 m high, with nine different inclination angles of the wall face in contact with the backfill ‘α’ (0°, 1.25°, 2.5°, 3.75°, +5°, -1.25°, -2.5°, -3.75°, and -5°) subjected to Northridge earthquake is used for illustration. The negative angle at the back of the wall is the case of the wall resting on the backfill. Figure 2 shows the cumulative displacements of the retaining wall away from the backfill due to combined sliding and rocking effects for α = -5°, 0° and +5° for a base width of 3.57 m. The foundation soil for this case was well graded sand (SW) and the backfill consisted of submerged silt gravel (GM). It can be observed from Fig. 2 that the negative values of α result in somewhat smaller cumulative displacements compared to the case of vertical wall face (α = 0) or for positive value of α within the range considered.

Another typical plot of cumulative displacement of a rigid retaining wall of 6.0 m height and having a base width of 4.61m and subjected to Northridge earthquake motion is shown in Fig. 3. The foundation and backfill soils in this case were silt of low compressibility and silty gravel respectively. The trend of the results in Fig. 3 is similar that in Fig: 2.Similar results were observed for other cases (Wu, 1996).

Table 2 shows a summary of new base widths and computed displacements for various inclinations. The computed cumulative sliding, rocking and total displacements are also shown in this table. The base widths decreased from 3.57m to 3.38m as the inclination changed from 0° to -5°, since the active earth forces decrease with negative inclination. Therefore, the base width was somewhat smaller for a wall with a negative inclination.

Table 2. Cumulative displacement for several angles of inclination of the back of the wall subjected to Northridge earthquake condition (B=3.57m) (Wu, 1996)

<table>
<thead>
<tr>
<th>Inclination-on angle (degree)</th>
<th>Base width-h (m)</th>
<th>Cumulative Displacement by Fixed Base Width (3.57m)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sliding (m)</td>
</tr>
<tr>
<td>+5.00°</td>
<td>3.81</td>
<td>0.0820</td>
</tr>
<tr>
<td>+3.75°</td>
<td>3.76</td>
<td>0.0820</td>
</tr>
<tr>
<td>+2.50°</td>
<td>3.70</td>
<td>0.0815</td>
</tr>
<tr>
<td>+1.25°</td>
<td>3.63</td>
<td>0.0808</td>
</tr>
<tr>
<td>0.00°</td>
<td>3.57</td>
<td>0.0808</td>
</tr>
<tr>
<td>-1.25°</td>
<td>3.50</td>
<td>0.0806</td>
</tr>
<tr>
<td>-2.50°</td>
<td>3.43</td>
<td>0.0805</td>
</tr>
<tr>
<td>-3.75°</td>
<td>3.35</td>
<td>0.0803</td>
</tr>
<tr>
<td>-5.00°</td>
<td>3.38</td>
<td>0.0801</td>
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</table>
Fig: 2. Cumulative displacements of walls (B1-F3) with different inclinations with the vertical.

Fig: 3. Cumulative displacements of walls (B1-F6) with different inclinations with the vertical.
The angular rotation in rocking (Table 2) decreased from 1.29° (α=0°) to 1.25° (α=-5°), and the total displacements decreased slightly from 0.2155m to 0.2112m. The cumulative displacements for these walls will not be significantly altered by changing the inclination at the back of the wall.

For the wall built as a leaning-type rigid retaining wall with α=5° lying on the backfill, the wall experienced a rocking movement of 1.25° during the Northridge earthquake. Therefore, when the wall was subjected to the same earthquake event up to 3 or 4 times, the wall experienced a total rocking close to 5°. At this time, the wall may become vertical.

Further analysis was conducted for 21 backfill and foundations soil combinations for a typical reference wall 6m high, subjected to three earthquakes. The backfill soil was varied from silty gravel to poorly graded sand, and the foundation soil varied from well graded gravel to silt of low compressibility. The results generally indicated that the design widths of foundations for 21 cases of backfill – foundation soil combinations used in analysis generally reduced with values of α from 0° α= -5°. This may results in saving of 8 -10 % in the material cost.

CONCLUSIONS

The following conclusions are drawn:

1. A realistic displacement model for rigid retaining walls under earthquake condition has been developed.
2. This model considers non-linear soil properties and any water condition behind the wall.
3. The available information on permissible displacements of retaining walls has been summarized but more research needed in this direction.

REFERENCES