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Comparison Between Finite Element Predictions and Results from Dynamic Centrifuge Tests on Tilting Gravity Wall Retaining Dry Sand

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SYNOPSIS An analytical model is developed to analyze the seismic response of gravity walls retaining and founded on dry sand, with special emphasis on tilting behavior. A well verified two-dimensional finite element code is used for this purpose. The analytical model is verified comparing predictions to results from three dynamic centrifuge tests, with satisfactory agreement. Moreover, sensitivity analyses are carried out for one of the centrifuge test conditions to understand how the results would change if the boundary conditions and rotational stiffness of the wall were changed.

INTRODUCTION

The behavior of earth retaining structures during earthquakes is considered an important design problem in seismic regions. One such structure is the gravity retaining wall, which uses its mass for stability against failure.

Field observations indicate that, where there has been significant movements of gravity retaining walls during earthquakes, rotational displacement (or tilting) of these walls has been important. The dynamic response of gravity walls that experience tilting and the effect of tilting on the overall displacement of these walls has received little study. Most of the available models in the literature were not successful in predicting qualitatively and quantitatively the field observations of gravity walls response during earthquakes and the results from the experimental tests on physical models of such walls.

A PROPOSED MODEL FOR EVALUATING TILT OF GRAVITY RETAINING WALLS DURING EARTHQUAKES

An analytical model was developed by Al-Homoud (1990) analyze the seismic response of gravity retaining walls with special emphasis on tilting behavior. An already existing and well verified finite element code named FLEX (Vaughan and Richardson, 1989) was used for this purpose. The proposed model by Al-Homoud (1990) has the following characteristics (Figure 1 shows the different features of the proposed model):

1) The soil (dry sand in this study) was modeled by a two dimensional finite element grid.

2) The gravity retaining wall is modeled as a rigid substructure.

3) The strength and deformation of the soil are modeled using the viscous cap constitutive model. This model consists of a failure surface and hardening cap together with an associated flow rule (see Figure 2). The cap surface is activated only for the soil under the wall to represent compaction during wall rocking. In addition, visco-elastic behavior is provided for representing the hysteretic-like damping of soil during dynamic loading . (For more details on this constitutive model, see work by Isenberg, Vaughan, and Sandler, 1980; Sandler and Rubin, 1979; and Vaughan and Isenberg, 1982).

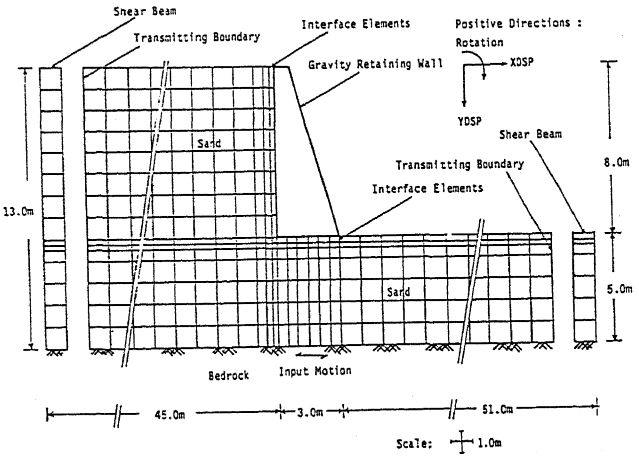


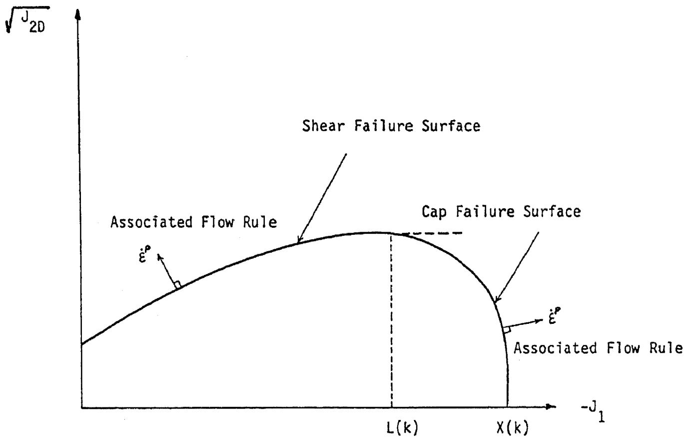
Figure 1 Proposed 2-D finite element grid for gravity retaining wall problems which shows the different features of the proposed model in this study.

4) Interface elements are used between the wall and the soil (at the back face of the wall and under its base)

to allow for sliding and for debonding/ recontact behavior.

5) The finite element grid is truncated by using an absorbing boundary approximation developed by Lysmer and Kuhlemeyer (1969). Using this boundary at both sides of the grid simulates the radiation of energy scattered from the wall and the excavation. Shear beams are placed adjacent to the lateral boundaries from each side which give the far-field ground motion, for comparison with those computed adjacent to the boundaries.

The procedure for carrying out the analysis was presented in detail by Al-Homoud (1990). The results of the different quantities from the analyses are obtained and presented in the from of time histories.



$$J_1 = \sigma_1 + \sigma_2 + \sigma_3$$

$$J_{2D} = \frac{1}{6}\,[(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2]$$

where in these equations σ_1, σ_2 and σ_3 are the principal stresses.

The associated flow rule requires the plastic strain rate vector $\dot{\varepsilon}^P$ (which is assumed to be the difference between the total strain rate and the elastic strain rate) to be normal to the yield surface.

Figure 2 Typical yield surface and hardening cap in the cap constitutive model

COMPARISON BETWEEN MODEL PREDICTIONS AND RESULTS FROM CENTRIFUGE TESTS ON TILTING GRAVITY WALL RETAINING DRY SAND

The porposed model is used in the current study in analyzing three "prototype" dynamic centrifuge tests on a tilting wall model conducted by Andersen et al. (1987). These tests were carried out at about 80g. Figure 3 shows a side view of tilting gravity retaining wall centrifuge test arrangement and plan view of experimental package. The soil was 14/25 Leighton Buzzard sand. Figure 4 shows the 2-D finite element grid of the "prototype" tilting retaining wall centrifuge test set-up as used in the analysis which illustrates the modeling features of the proposed model. Table 1 summarizes the model quantities measured in these tests. Table 2 summarizes the "Prototype" tilting wall parameters as used in the analysis. Table 3 gives the "Prototype" input motion characteristics. The sand used in the analysis is 120/200 Leighton Buzzard dry sand at a relative density of about 80% due to lack of laboratory test results on the cyclic shear strength of 14/25 Leighton Buzzard sand. The angle of friction for this sand at this density is about 40 degrees and its dry density is about 1530 $\mathrm{Kg/m^3}$.

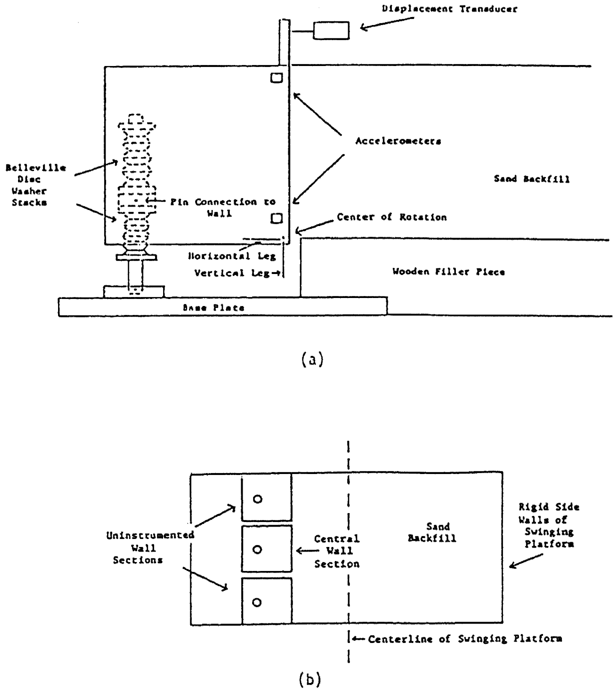


Figure 3 (a) Side view of tilting gravity retaining wall centrifuge test arrangement, (b) Plan view of experimental package. (from Andersen et al., 1987)

The input parameters of the viscous cap constitutive model are evaluated from monotonic compression tests on 120/200 Leighton Buzzard sand conducted by Gately et al. (1985) and cyclic triaxial compression tests on the same sand conducted by Pahwa et al. (1986). The estimated input parameters of the viscous cap constitutive model for 120/200 Leighton Buzzard sand are omitted from this paper for the sake of briefness. The shear and bulk moduli are chosen to vary with depth as a function of the initial effective stress, and correspond to the levels of strains expected in the dynamic analysis. A damping ratio of 8.5% is used in the analysis using the proposed model.

In comparing the proposed model predictions to the results from the "Prototype" centrifuge tilting wall tests, the model proved to be successful both quantitatively and qualitatively. A summary of the main predicted and measured "prototype" dynamic quantities for the three centrifuge tests is given in Table 4.

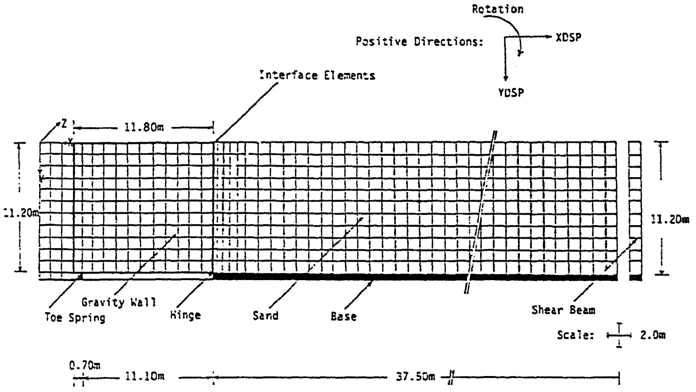


Figure 4 A 2-D finite element grid of the "Prototype" tiling gravity retaining wall centrifuge test

Table 2 "Protoype" tilting wall parameters

Parameter	"Prototype" Value
Dept of Sand Backfill above Hinge Point	11.181 m
Width of Wall	11.796 m
Length of Wall (in the Z Direction)	11.796 m
Location of Wall Center of Gravity	
To Left of Hinge	4.659 m
Above Hinge	5.033 m
Location of Spring Assembly Pin:	
To Left of Hinge	11.099 m
Above Hinge	1.966 m
Backfill Length behind wall	37.353 m
Total Mass of Wall	1388346.920 kg
Mass of Wall/LM(1) of Wall	117696.420 kg/LM
Total Mass Moment of Inertia of Wall about Hinge.	111193142.1 kg-m^2
Mass Moment of Inertia of Wall about Hinge/LM of Wall.	9426343.0 kg-m^2/LM
Total Mass Moment of Inertia of Wall about Center of Gravity.	14477724.0 kg-m^2
Mass Moment of Inertia of Wall about Center of Gravity/LM	1227341.8 kg-m2/LM
Spring Stiffness/LM of Wall	
Test GA3EQ1 (Soft)	2298.206 kN/m/LM
Test GA6EQ1 (Medium)	4596.412 kN/m/LM
Test GA5EQ1 (Stiff)	13789.235 kN/m/LM
Rotational Stiffness/LM of Wall	
Test GA3EQ1 (Soft)	283110.943 kN-m/rad/LM
Test GA6EQ1 (Medium)	566221.640 kN-m/rad/LM
Test GA5EQ1 (Stiff)	1698666.153 kN-m/rad/LM

(1) LM stands for linear meter of wall length in the z-direction.

Table 1 Model quantities measured in tests GA3EQ1, GA6EQ1 and GA5EQ1 (from Andersen et al., 1987)

Quantity	Test GA3EQ1			Test GA6EQ1			Test GA5EQ1		
	Minimum	Maximum	Residual	Minimum	Maximum	Residual	Minimum	Maximum	Residual
Horizontal Pin Acceleration (ft/sec^2)	-283	+272		-345	+349		-223	+194	
Angular Wall Acceleration (rad/sec^2)	-514	+820		-416	+555		-360	+459	
Initial Displacement at Top of Wall (in)(1)			(0.117)			(0.068)			(0.021)
Displacement at Top of Wall (in)(2)	0.127	0.143	0.142	0.066	0.076	0.074	0.021	0.024	0.023
Initial Earth Force (lbs)			(107)			(104)			(109)
Earth Force (lbs)	94	198	154	93	192	146	91	173	137
Initial Resultant Height as % of Total Height			(30.6)			(40.6)			(25.9)
Resultant Height as % of Total Height	21	54	38.4	31	57	43.4	19	42	33.7
Initial Wall Friction Angle (Degrees)			(17.6)			(16.5)			(9.1)
Wall Friction Angle (Degrees)	5	27	6.7	-3	33	11.3	-9	17	3.7

(1) Initial displacment refers to those at the end of gravity spin-up
(2) These " measured" displacments have been deduced from the spring force data and spring constants. This is becuase measurments from LVDT's were suspect.

Table 3 "Prototype' input motion characteristics

Quantity	Test GA3EQ1	Test GA6EQ1	Test GA5EQ1
Average Amplitude			
(+ve) (1) (m/s^2)	+0.889	+1.005	+0.724
(-ve) (m/s^2)	-0.938	-1.100	-0.791
Predominant Frequency(Hz)	1.46	1.48	1.43
Peak Amplitude Among All Cycles			
(+ve) (m/s^2)	+1.167	+1.451	+1.037
(-ve) (m/s^2)	-1.259	-1.588	-1.104

(1) Sign convention for input motion is as follows:
+ve toward the backfill
-ve away from the backfill

The phasing relations between the different quantities in the problem are found to be the same in both the results from the dynamic analysis using the proposed model and the measurements from the centrifuge tests. These are summarized below:

1. The maximum earth pressure behind the wall occurs when the wall is at its maximum displacement towards the backfill, which occurs also at the time of a maximum outward horizontal acceleration at the base.

Table 4 Comparison between measured and predicted values of different "Prototype" dynamic quantities of tilting wall centrifuge tests

Quantity	Test GA3EQ1 "Soft"			Test GA6EQ1 "Medium"			Test GA5EQ1 "Stiff"			Average Absolute Error (%)
	Measured	Predicted	% Error	Measured	Predicted	% Error	Measured	Predicted	% Error	
Displacement at Top of Wall:										
Peak Outward (m)	5.1×10^{-2}	4.07×10^{-2}	-20%	1.57×10^{-2}	2.30×10^{-2}	+45%	0.59×10^{-2}	0.56×10^{-2}	-5%	23%
Residual Outward (m)	4.92×10^{-2}	3.21×10^{-2}	-35%	1.18×10^{-2}	1.47×10^{-2}	+25%	0.39×10^{-2}	0.23×10^{-2}	-40%	33%
Increase in Horizontal Earth Force:										
Peak (N)	2.43×10^{6}	3.16×10^{6}	+30%	2.35×10^{6}	3.77×10^{6}	+60	1.71×10^{6}	1.61×10^{6}	-6%	32%
Residual (N)	1.25×10^{6}	0.70×10^{6}	-44%	1.21×10^{6}	+13%	0.75¥106	0.23×10^{6}	0.23×10^{6}	-69%	42%
Increase in Spring Force:										
Peak (N)	8.00×10^{5}	10.94×10^{5}	+37%	9.33×10^{5}	12.6×10^{5}	+35%	7.55×10^{5}	8.89×10^{5}	+18%	30%
Residual (N)	6.89×10^{5}	8.13×10^{5}	+18%	6.25×10^{5}	8.0×10^{5}	+28%	6.01×10^{5}	3.70×10^{5}	-38%	28%
Horizontal Acceleration at:										
* 2.5 m behind wall 1.0 m below backfill surface (%g)$^{(2)}$	31.9	35.5	+11%	26.7	27.4	+3%	16.3	15.2	-7%	7%
* 20.0 m behind wall and 1.0 m below backfill surface (% g)	22.2	23.6	+6%	26.1	33.1	+27%	20.5	24.6	+20%	18%
* 20 m behind wall and 5.0 m below backfill surface (% g)	20.2	26.4	+31%	20.5	23.0	+12%	15.9	+16.5	+4%	16%
Height of Residual Resultant Earth Force (Static + Dynamic) (% H)$^{(3)}$	38.4	62.3	+62%	43.4	50.0	+15%	33.7	41.4	+23%	33%
									Average	26%

Note:
(1) Quantities reported here are due to dynamic loading
(2) g is the gravitational acceleration
(3) H is the wall height

2. The minimum earth pressure occurs when the wall is at its maximum displacement away from the backfill, which also occurs at the time of a maximum inward horizontal acceleration at the base.

3. The peak accelerations at the top of the far field and at top of the wall lag those at the base. The amount of lag is found to be dependent on the ratio of excitation frequency to the fundamental frequency of the backfill layer (i. e. f/f_1 ratio). For example, for an f/f_1 ratio of 1.06, this lag is found to be about 130 degrees, while for an f/f_1 ratio of o.53, the lag is about 60 degrees.

4. The highest location of the resultant earth force above the bottom of the wall occurs at the time of maximum earth pressure, while the lowest location occur at the time of minimum earth pressure.

It is important to emphasize that the phasing relations in (1) and (2) above are just the opposite of the result reached using the Mononobe-Okabe (1929) approach, assuming active conditions at all times, and the result observed during shaking table tests such as those by Sherif et al. (1981).

Sensitivity analyses were carried out for one of the centrifuge test conditions to understand how the results would change if the boundary conditions and rotational stiffness of the wall were changed. These are:

1. Replacing the toe spring (which was used in the tilting wall tests to provide resistance to tilting) by an elastic foundation to obtain the same magnitude of permanent outward tilt. Figure 5 shows a 2-D finite element grid of the "prototype" tilting gravity retaining wall centrifuge test modified (compared to that of Figure 4) to include a 5.0 m foundation as a replacement of the toe spring.

2. Replacing the toe spring by a dry sand foundation (modeled by finite elements in which the sand behavior is represented by a viscous cap constitutive model with the cap surface active) and allowing free rocking and sliding of the wall by removing the hinge (note that in the tilting wall tests by Andersen et al., the wall is hinged at the heel) results in small amount of sliding compared to tilting. These results emphasize the importance of tilting of gravity retaining walls during earthquakes.

3. Varying the stiffness of the toe spring to cover values of spring stiffness other than those in the tests. The

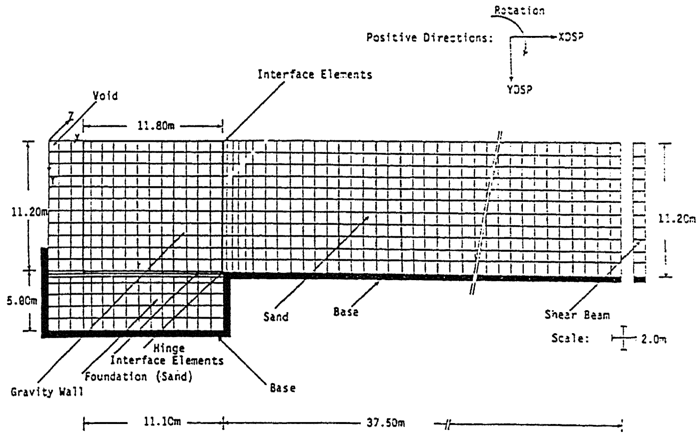


Figure 5 A 2-D finite element grid of the "Prototype" tiling gravity retaining wall centrifuge test modified to include a 5.0m foundation as a replacement of the toe spring.

relation between the predicted tilt and the spring stiffness (from all cases analyzed) is shown in Figure 6.

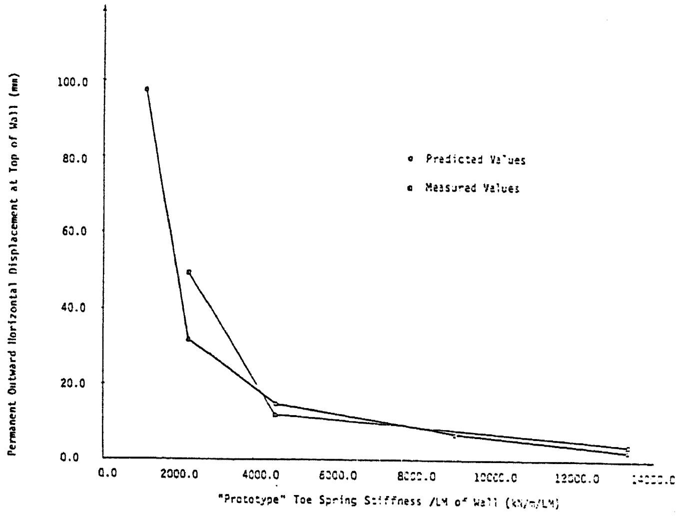


Figure 6 Predicted and measured "Prototype" permanent outward horizontal displacement at top of wall for different values of "Prototype" toe spring stiffness

Overall evaluation of the comparison

As shown in Table 4, the comparison between the main predicted and measured "Prototype" dynamic quantities for the three analyzed centrifuge tests resulted in an overall average absolute error of about 26%. Indeed this reflects successful predictions knowing that there are some inaccuracies and difficulties encountered in the tests and approximations and drawbacks in the proposed model. The inaccuracies encountered in carrying out the centrifuge tests were given by Andersen et al. (1987). Examples are:

(1) nonuniform acceleration field applied in the centrifuge tests; and (2) some problems with the load sensing devices in the high gravity environment and the reflected uncertainty on the value of the wall displacement as this displacement is computed from the spring force instead of direct measurements due to problems with the displacement transducers.

The proposed model is by itself an approximation to the real problem. Moreover, there are some drawbacks in certain aspects of this model. One of these is the inability of the viscous cap constitutive model (used to represent the behavior of the sand) to include the hysteretic volumetric strains which develop in the sand during dynamic loading (e.g. Stamatopoulos, 1989). In the analysis using the proposed model, the deformations in the backfill behind the tilting wall are mainly due to shear strains. As a result of the wall-backfill interaction during dynamic loading at the base, the wall ended with a permanent outward tilt. This permanent tilt is due to the permanent increase in the horizontal stresses and shear strains behind the wall (mainly the upper 2/3 of it). The authors believe that if the hysteretic volumetric strains are modeled during dynamic loading on top of the modeling capabilities of the viscous cap model as discussed above, vertical downward deformation (i. e. compaction) will be superimposed on the deformations resulted from using the viscous cap model alone. This will cause an increase in the horizontal stresses mainly near the bottom of the wall (because of larger shear stresses) causing a downward shift in the resultant horizontal earth force. In fact, it is difficult without carrying out the analysis (with this new feature) to quantify the effect on the wall tilt. However, the effect may be negligible in the centrifuge tilting wall problem analyzed in this chapter due to the following reasons: (1) The downward shift in the resultant horizontal earth force is accompanied with an increase in the magnitude of this force and depending on the magnitude of these changes, the moment which causes the permanent tilt may not change; and (2) the foundation under the wall is just a linear elastic spring compared to the situation of a real foundation which experiences compaction due to the added feature.

The second drawback in the proposed model is the inability to model the nonlinearty in the soil behavior within the failure surface (i. e. nonlinear elastic behavior). This behavior is approximated by choosing the shear and bulk moduli to be compatible with the levels of strains expected in the dynamic response of the backfill soils in the current analysis.

Finally, the approximation of the actual coarse 14/25 Leighton Buzzard sand by the properties for fine 120/200 Leighton Buzzard sand. Indeed, it is not possible without having the necessary test results to quantify the error in this approximation. However, as discussed by Al-Homoud (1990) this approximation is very reasonable.

SUMMARY AND CONCLUSIONS

On the basis of the discussions and results of the current study, the following summary and conclusions can be made:

1. There is a wide use of Mononobe-Okabe (1929) formula or the Seed-Whitman (1970) simplified equation for computing the maximum dynamic earth force on a retaining structure for design purposes. However, the accuracy of these equations is still in debate. Moreover, there is confusion in the literature on the location of this force.
2. An analytical model is developed in this study to analyze the seismic response of gravity walls retaining and founded on dry sand, with special emphasis on tilting behavior. The model considers all aspects of the dynamic gravity retaining wall problem.
3. The results from the current study showed that the Seed-Whitman (1970) simplified equation is conservative while the location of the maximum dynamic earth force is higher than 0.6 H above the base, which is the value suggested by Seed and Whitman (1970).
4. In applying the proposed model to three "prototype" tilting gravity retaining wall dynamic centrifuge tests by Andersen et al. (1987), we observe:
    (a) The proposed model predictions are in an excellent qualitative agreement (i.e. regarding phasing relations) and in good quantitative agreement (e.g. magnitude of wall tilt, dynamic earth force, etc.) with the measurements from the studied tests.
    (b) Replacing the toe spring (which was used in the tilting wall tests to provide resistance to tilting) by a dry sand foundation (modeled by finite elements in which the sand behavior is represented by a viscous cap constitutive model with the cap surface active) and allowing free rocking and sliding of the wall by removing the hinge result in small amount of siding compared to tilting. These results emphasize the importance of tilting of gravity retaining walls during earthquakes.

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