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SEISMIC PERFORMANCE ANALYSES OF GEOSYNTHETIC REINFORCED SLOPES

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ABSTRACT

In an effort to improve understanding of seismic behaviour of geosynthetic reinforced soil slopes, the authors developed a numerical model which is capable of predicting seismic performance of geosynthetic reinforced slopes. Test measurements of large scale geosynthetic reinforced slope shaking table tests were first used to verify the developed model. It was found that both internal and external movement of test slopes were successfully modelled to a certain extent. In addition to verification utilizing laboratory test data, the developed model was also employed to simulate performance of a GRS slope that failed during the 1999 Chi-Chi earthquake in Taiwan. The developed numerical model was able to describe seismic behaviour such as acceleration history, reinforcement strain distribution, as well as failure mechanism in reasonable range. In this paper, details of model development and results of the model verifications are presented. Moreover, influence of important design factors such as vertical spacing and embedded length of reinforcement on seismic performance of GRS slopes are carefully examined as well. Progress of offered study is hoped to be helpful in gaining understanding of seismic performance of geosynthetic reinforced structures.

Keywords: Geosynthetic Reinforced Slopes, Seismic Performance Analysis, Numerical Modeling, Shaking Table Test

INTRODUCTION

Because of the well performance in sustaining seismic loading, geosynthetics reinforced soil (GRS) slopes have gradually become one of important applications in earthquake prone area such as Taiwan and Japan. However, present seismic design of GRS slopes are still based on pseudo-static limited equilibrium concept, or Newmark type deformation analysis for few cases of research purpose. Performance information such as internal deformation and reinforcement strain distribution is not yet being well studied in the past. Reasons for little progress was made on analyzing such seismic performance of GRS slopes in the past are limited performance data available as well as lack of calibrated numerical models that is capable of reproducing seismic behaviour of GRS slopes.

In an effort to improve understanding of seismic behaviour of GRS slopes, the authors developed a numerical model which is capable of simulating seismic performance of GRS slopes. Test measurements of large scale shake table tests of GRS slopes, that were conducted in University of Washington, United States, were first used to verify the developed models. It was found that the developed model was able to reproduce the shaking table test results within a reasonable agreement. Both internal and external post shaking displacement of test slopes were successfully modeled to a

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certain extent. In addition to verification utilizing laboratory test data, the developed model was also employed to simulate performance of a GRS slope that failed during the 1999 Chi-Chi earthquake in Taiwan. The developed numerical model was able to describe seismic behaviour such as acceleration history, reinforcement strain distribution, as well as failure mechanism in realistic manner. Feasibility of applying the developed model to analyze seismic behaviour of GRS slopes was further confirmed.

In this paper, details of model development and results of the model verifications are presented. Moreover, influence of important design factors such as vertical spacing and embedded length of reinforcement on seismic performance of GRS slopes are carefully examined as well. Progress of offered study is hoped to be helpful in gaining understanding of seismic performance of GRS structures.

NUMERICAL MODEL

Model Composition

Figure 1 shows the schematic drawing of typical cross section of the developed numerical model. The developed models were modified from the numerical models that were developed by Lee (2000) for analyzing performance of GRS retaining structures. They were built using the finite difference method computer program FLAC. In the developed models, backfill materials were modeled using Mohr-Coulomb material elements coded with hyperbolic soil modulus model. Geosynthetic reinforcements were modeled using the structural cable elements (Lee, 2000). In order to avoid computational instability of many interfaces under seismic loading condition, the authors further updated Lee's model into the "rigid cable box" (RCB) model. In the RCB model, complicated facing systems of GRS structures such as modular block system was simplified as a Mohr-Coulomb material element surrounded with structural cable elements tied to the element node points.

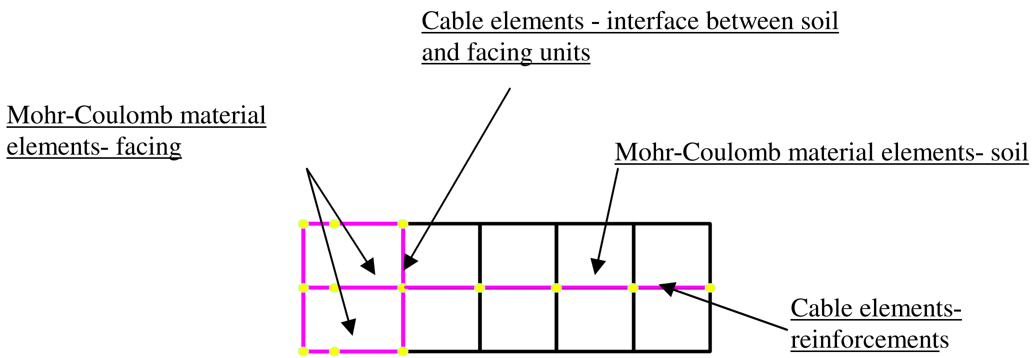


Figure 1. Schematic Drawing of Typical Cross Section of The Developed Model.

Input Property Determination

Input values of soil moduli were calculated based on the in-situ SPT-N values, and coefficients of hyperbolic soil modulus model were adapted from triaxial test results of similar material reported by Duncan et al (1980). Input values of geosynthetic reinforcement were determined following the similar procedures suggested by Lee (2000). For complicated facing system such as modular block, properties of the gravel infill of the modular block facing system were used as the inputs of the Mohr-Coulomb element except the unit weight, and unit weight of concrete were used as the input unit weight. Strength properties of the masonry block were assigned as the input properties of the cable elements. Failure or yielding criteria of the modular block facing system could be defined by setting upper bound stress/strain values of the RCB model. By using the RCB model, iteration process of the numerical analysis was not interrupted even the modular block facing system failed; moreover, stress-strain distribution of critical states inside the backfill could be observed.

MODEL VERIFICATION I- UW SHAKING TABLE TESTS

UW Shaking Table Tests

The developed numerical models were first verified using shaking table test results of GRS slopes. These tests were large scale GRS slope tests that were conducted at University of Washington, Seattle, United States, in a purpose to investigate seismic performance of GRS slopes (Perez, 1999). Figure 2 shows the test facilities. The shaking table was in a size of 1.2m(W)x1.2m(H)x2.4m(L). It was equipped with single degree shaking actuator. Capacity of the shaking table could reach 1.0g level of acceleration, and only sine wave motion was adapted as the input motion in these tests. Side walls of the shaking table box were installed with ply wood boards. Two layers of plastic sheets were added to reduce and average the side wall friction. Figure 3 shows the typical instrumentation layouts of the shaking table tests. Performance measurements including face displacement, acceleration, and interior movement were recorded.

Figure 2. Shaking Table Facility at University of Washington. (Perez, 1999)

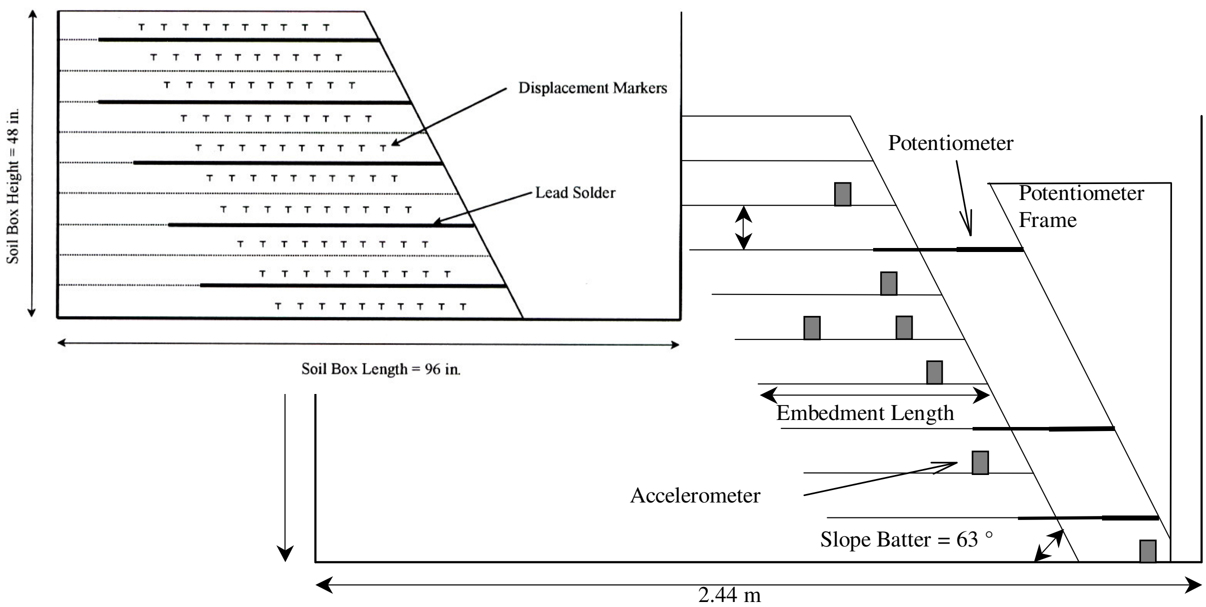


Figure 3. Sketch Example of Instrumentation Layouts of Shaking Table Test. (Perez, 1999)

Verification Model

Total 6 tests were replicated in the reported study. All 6 tests were designed with a 63 deg slope angle and a height of 1.2m. Table 1 summarized the basic design information and test results of analyzed shaking table tests. All selected models were installed with 10 layers of geosynthetic reinforcement, except Test No. 14 and 15. Test No. 14 is installed with 20 layers of reinforcement and Test No. 15 is installed with only 6 layers of reinforcement, in an effort to investigate the effect of vertical spacing. Figure 4 shows an example model utilized in simulating the test results. In these models, cable elements were used as the geosynthetic reinforcement and Mohr-Coulomb material elements were used as backfill material for both soil in reinforced zone and unreinforced zone. The input motion is directly applied at base of the model. Rear boundary of the model was first modeled with rigid boundary to simplify modeling process; yet effects of this simplification were also investigated with different boundary conditions.

Table 1. Design Information and Test Result Summary of Analyzed Shaking Table Tests

Test No.	Le/H*	J/Sv, kN/m/m**	Applied Max. Acceleration, g	Seismic Loading Duration, sec	Max. Face Deflection, d_max (mm)	Yield Acceleration, a_y (g)
7	0.31	9.7	0.72	37	41	0.44
8	0.25	9.7	0.7	30	56	0.35
9	0.21	9.7	0.5	26	49	0.28
10	0.5	2.08	0.6	81	26	0.38
13	0.25	2.08	0.5	44	49	0.31
14	0.31	4.17	0.7	83.5	27	0.62
15	0.31	1.25	0.4	29.5	35	0.27

*: Le- embedded length of geosynthetic reinforcement, H- height of slope.
**: J- stiffness of geosynthetic reinforcement at peak, Sv: vertical spacing.

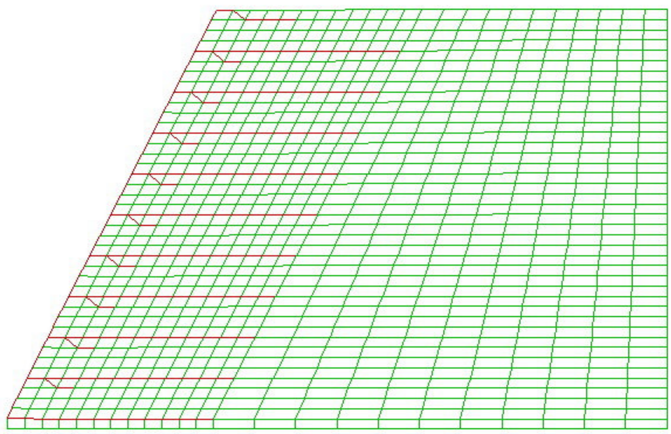


Figure 4. Example of FLAC model

Side wall friction reduction was applied by superposing horizontal back draw acceleration onto the model itself. The back draw acceleration was obtained by calculation the inertial forces of soil body acting on the side walls, and further converting the friction forces into accelerations.

The developed models were able to simulate the construction stages, and record stress and strain occurred before seismic loading was applied. To predict the test measurements, modeling results were obtained by subtracting the static performance during construction stage from the result of dynamic analysis.

Modeling Result

Figure 5 to Figure 7 shows the typical examples of modeling results. Figure 5 shows the face displacement of Models No. 8 and 13 at end of tests. The developed models were able to reproduce the deformation of the test slopes within a reasonable range. Figure 6 shows the internal deformation distribution of Models No. 13 and 15 at end of tests. And Figure 7 shows the internal stress distribution of Models No. 13 and 15 at end of tests. The developed models have successfully predicted the internal deformation distribution of modeled tests, as well as to provide valuable information of stress distribution inside the GRS slope model tests. As shown in Figures 6 and 7, the boundaries of sliding areas inside the slopes were clearly identified. For GRS slope with smaller embedded reinforcement length over height ratio (Le/H), yet higher reinforcement stiffness over vertical spacing ratio (J/Sv), such as model 13, has a "yielding surface" with flatter angle from toe of the slope, and extends a further distance into the unreinforced zone. In comparison to Model 13, Model 15, which has a higher Le/H ratio, and a much smaller J/Sv value, exerts a steeper yielding surface along the reinforced zone and suffers higher stress level at location near slope face. This observation is found to be consistent with the comments made by Perez (Perez, 1999) at University of Washington.

To examine the influence of design factors such as Le/H and J/Sv on seismic performance of tested slopes, modeling results was further summarized in Figures 8 and 9. As indicated in the figures, GRS slopes with longer embedded length of reinforcement, smaller vertical reinforcement spacing, and higher reinforcement stiffness will posses higher yield acceleration, i.e., better resistance to seismic loading. However, sensitivity of such design factor on seismic performance of GRS slopes needs more research efforts.

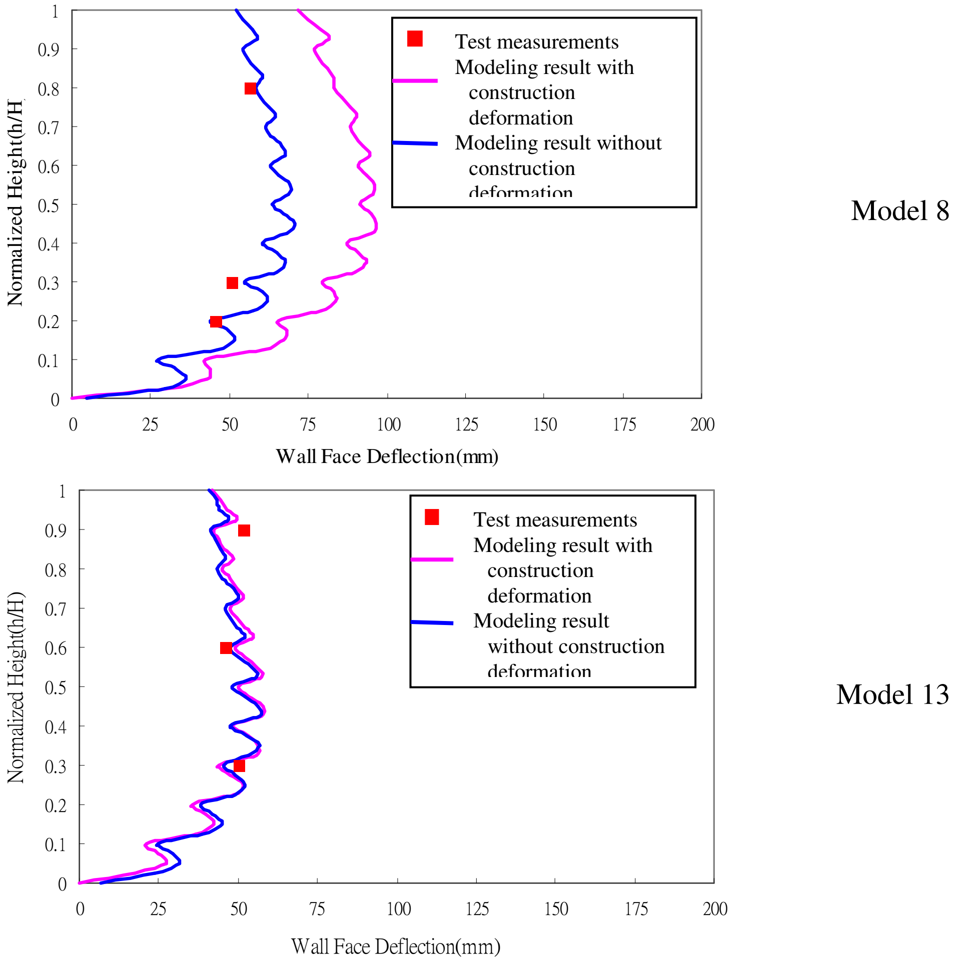
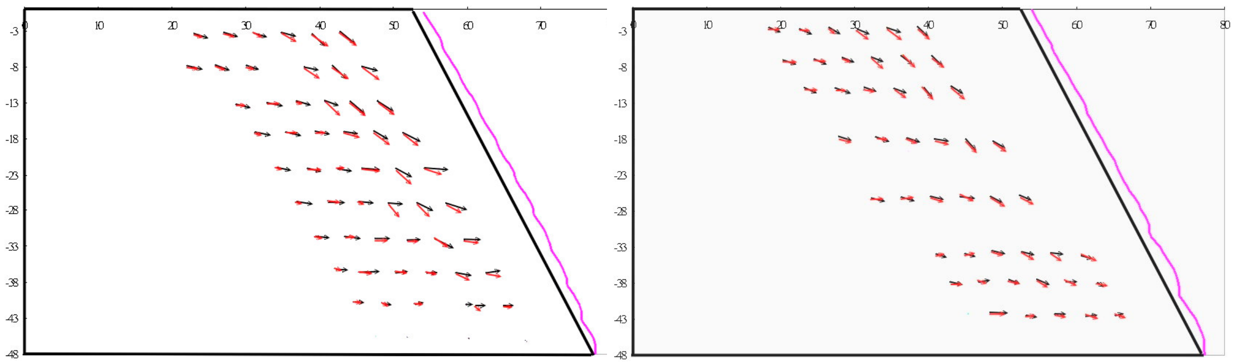
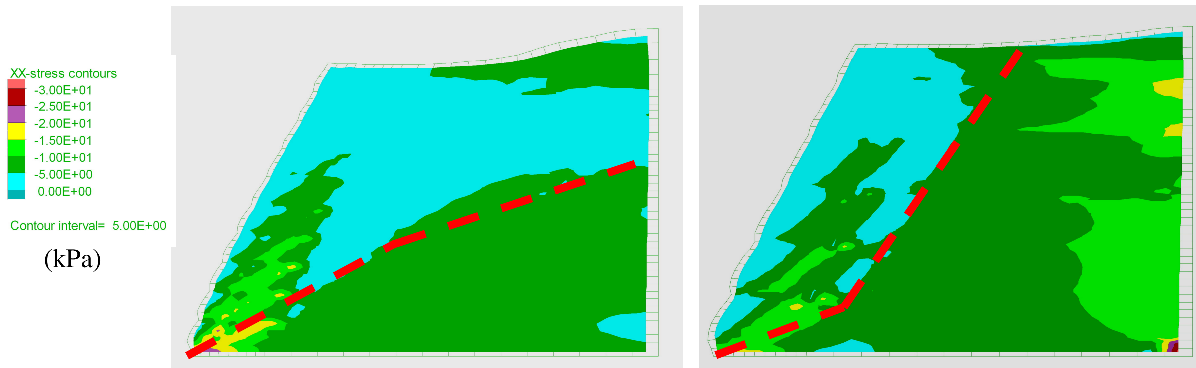


Figure 5. Face Displacement of Models No. 8 And 13 at End of Tests



Model 13 Model 15

Figure 6. Internal Deformation Distribution of Models No. 13 and 15 at End of Tests



Model 13 Model 15

Figure 7. Internal Stress Distribution of Models No. 13 and 15 at End of tests

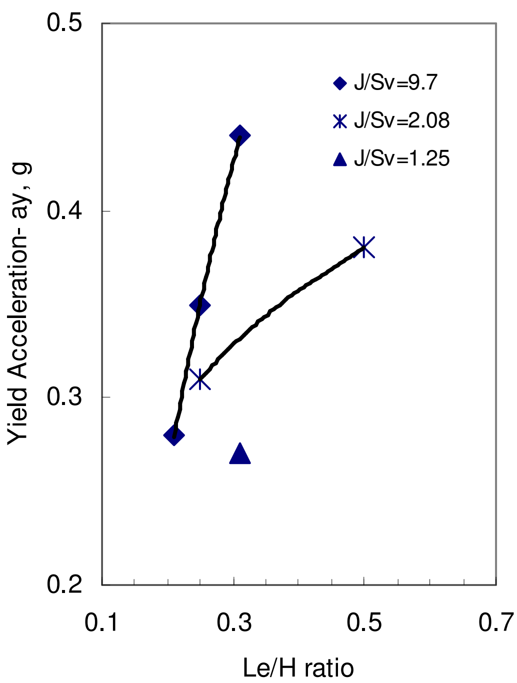


Figure 8. Yield Acceleration versus Le/H Ratio for Modeled Shaking Tests

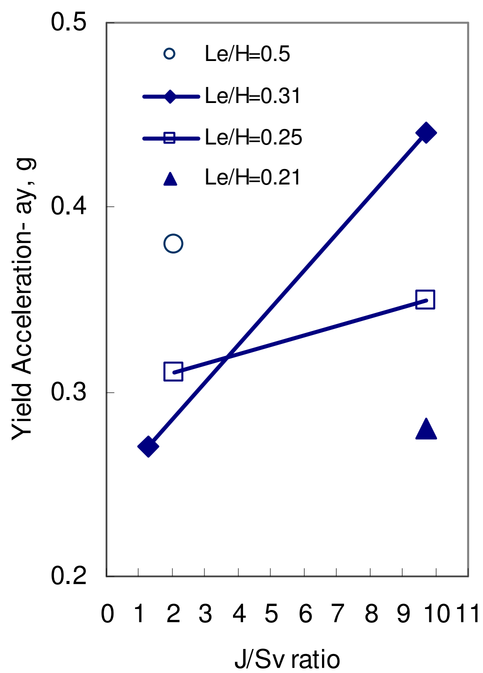


Figure 9. Yield Acceleration versus J/Sv Value for Modeled Shaking Tests

MODEL VERIFICATION II- MODULAR BLOCK FACED GRS WALL FAILURE

In order to further verify the developed model, field performance of a modular block faced GRS wall during the 1999 Chi-Chi earthquake was studied by the authors. Seismic performance during earthquake as well as final failure mechanism was successfully simulated by utilizing the developed model.

Da-Kung Wall

The studied wall was located in the Da-Kung Village, Taichung County, Taiwan. Surface scarp of the major fault movement of Chi-Chi earthquake was located about 5km west of the site. The wall was constructed in the east-west direction with modular block faces toward north. The wall was about 80m long with a height of 3.2m. The geogrid reinforcement was installed with vertical spacing of 0.8m. The embedded lengths of reinforcement were 2.7m for the bottom two layers and 3.0m for the top layer (Lee et al., 2001). Figure 10 shows the failure photo and schematic failure drawing (after Huang and Tatsuoka, 2001) of the studied wall. As shown in the figure, the wall collapsed with reinforcement ruptures and face block dislocations.

Study Result

In order to avoid instability of many interfaces between different materials of modular block wall system, the "rigid cable box" model described in previous sections of this paper was used to develop the numerical model. Real earthquake motion records were used as the input ground motion in this study. Figure 11 shows both input ground acceleration and the simulated face displacement history of the studied wall. As presented in the figure, the wall started to exert distinguished deformation about 12 seconds after the earthquake started. Figure 12 shows the internal displacement vectors, axial force distribution of reinforcement (cable element), and shear strain distribution inside the studied wall at 12 seconds after the earthquake started. As shown in Figure 12, the modular block facing was bulged at about 1/3 to 1/2 of the wall height; and facing blocks located at lower half of the wall posses higher stress level than upper half blocks. Moreover, all three layers of reinforcements and facing blocks had suffered serious stress concentration near the wall face, especially at the lower layers. This observation agrees to the field observation of wall failures.

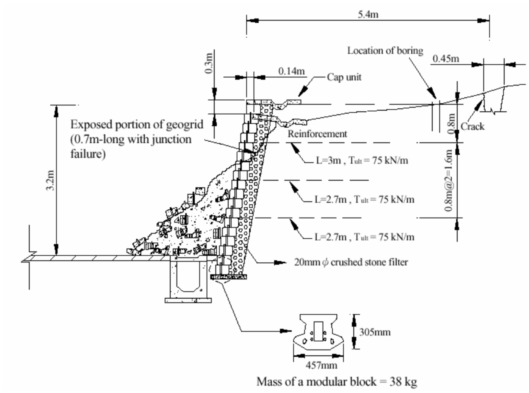
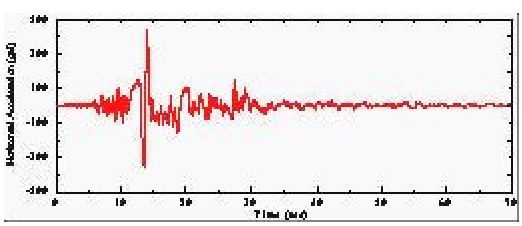


Figure 10. Failure Photo and Schematic Drawing of Da-Kung Wall (after Huang and Tatsuoka, 2001).



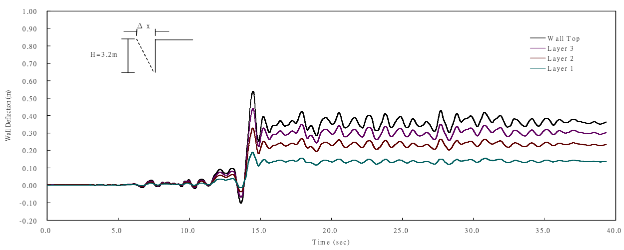


Figure 11. Input Ground Acceleration and Simulated Face Displacement of Da-Kung Wall.

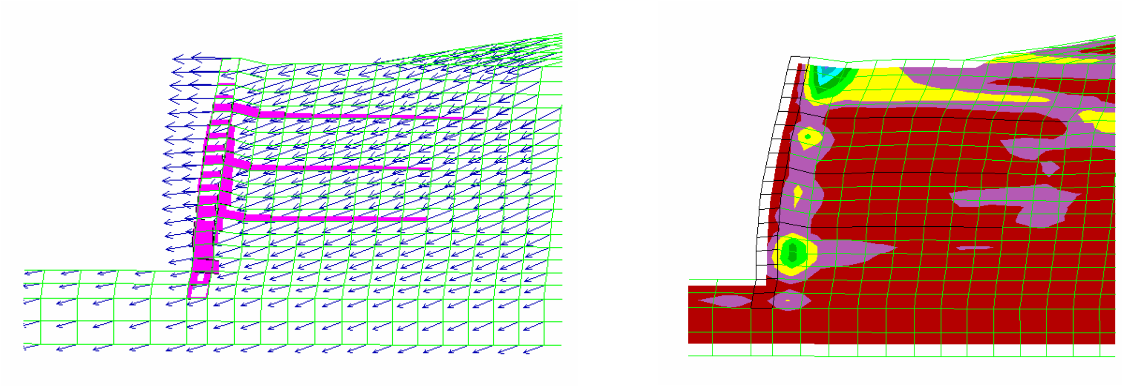


Figure 12. Yield Acceleration versus J/Sv Value for Modeled Shaking Tests

CONCLUSION

In this paper, capability of the developed numerical models were verified by successfully reproducing laboratory shaking table measurements as well as field performance of GRS slopes during real earthquake. Feasibility of utilizing the simplified "rigid cable box" model to simulate difference seismic performance of the GRS structures, as well as to predict possible failure mechanisms were verified in this study. Influence of design factors such as embedded length and vertical spacing of reinforcement on seismic performance of GRS slopes were examined as well.

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