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Reducing seismic displacement of gravity-type highway bridge abutments using soil reinforcement

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ABSTRACT: Seismic displacements of a typical gravity-type highway bridge abutments used in Taiwan are examined using input ground accelerations suggested by new and old aseismic design codes. A pseudo-static-based multi-wedge method is used in conjunction with the Newmark's sliding block theory to evaluate seismic displacement of the bridge abutment. It is found that gravity bridge abutments located in some seismically active areas may displace far beyond allowable values when subjected to level 2 and 3 intensities of shaking specified in the new code. Test results of a series of shaking table tests on model soil retaining walls show that large-diameter soil nailing and geosynthetic-reinforcement are effective in reducing seismic displacement of gravity-type abutments to mitigate possible catastrophic failures in seismically active zones.

1 INTRODUCTION

Highway bridge abutments constitute an important link between the approach embankment and the deck of bridge. A possible measure to ensure the integrity of bridge abutments under strong ground shaking is to limit the relative horizontal displacement of the abutment. This requires an accurate evaluation on the seismic displacement of the bridge abutments in the aseismic design. In current aseismic design guidelines for highway-related structures in the North America, force-based design of abutments and soil retaining walls still prevails, displacement-based design is not mandatory even for an essential abutment located in a seismically active area (namely, a design category 'D', the highest priority among four design categories, see AASHTO, 2002)

In Japan, after the shock of the 1995 Hyogoken-Nambu earthquake ($M_L = 7.2$) in Kobe area, the railway authority first adopted a two-level input earthquake intensities in the aseismic design of reinforced and unreinforced structures, including earth retaining walls, bridge abutments and earth embankments (Tatsuoka et al., 1996, 1998 and JRTRI, 1999). Displacement-based damage levels were specified for soil structures under corresponding input intensities of ground shaking. A pseudo-static method called 'multi-wedge method' incorporating with Newmark's sliding block theory is used in the present study to investigate the seismic displacement of a typical gravity-type bridge abutment. This method was developed by Huang et al. (2003) and Huang and Chen (2004), and was validated by analyzing four geosynthetic-reinforced modular block walls in 1999 Chi-Chi earthquake (Huang et al. 2003), two leaning-type soil retaining walls situated on slopes (Huang and Chen, 2004 and Huang, 2005) and a geosynthetic-reinforced railway embankments survived the 1995 Hyogoken-Nambu earthquake (Huang and Wang, 2005b).

2 COMPARISONS BETWEEN OLD AND NEW DESIGN CODES IN TAIWAN

Old seismic design codes (SDC) for highways and bridges in Taiwan was issued in 2000 by the Construction and Planning Agency, Ministry of the Interior, (CPAMI, 2000). In which, a maximum earthquake with a 475-year return period was used and different design values of horizontal peak ground acceleration ($HPGA_{design}$) were assigned for two seismic zones, i.e., $HPGA_{design}$ = 0.23 g for Taipei and Kaohsiung areas; 0.33 g for other areas. (CPAMI, 2000, see Fig. 1). A new SDC was enforced in July 2005 (CPAMI, 2005). Three levels of design HPGA and safety requirements, namely, Level 1: the structure should remain in elastic earthquake reinforcement for the poorly-behaviored condition under a medium/ small earthquakes of 30-abutments should be considered. year return period, Level 2: the structure should remain in allowable ductile state under a design earthquake of 475-year return period and Level 3: the structure should be kept from reaching ultimate ductility state (or reaching ultimate collapase state) under a design earthquake of 2500-year return period. For earth retaining walls including bridge abutment, these design guideline suggested a design horizontal seismic coefficient (k_h) equal to (0.5°D HPGA)/g (g: gravitational acceleration). Seismic displacement calculation was not required in these design codes.

Figure 1 also shows the seismic zones with respective spectral acceleration coefficient (D_S^D) in new SDC. The $HPGA_{design}$ for major bridges in six representative zones (Fig. 2) in west Taiwan are shown in Table 1. This table shows that the $HPGA_{design}$ in new SDC are generally 18%~63% greater than those in old SDC. Seismic displacement of many bridge abutments designed and constructed based on the old version of SDC must be examined using new seismic loadings specified in the new version of SDC. Pre-earthquake reinforcement for the poorly-behaviored abutments should be considered.

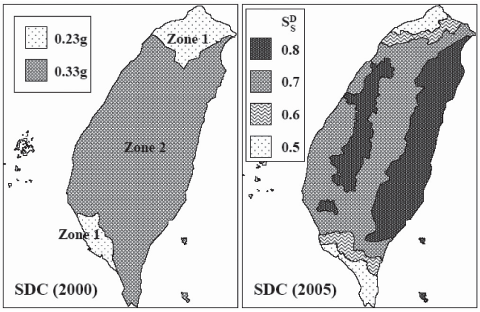

Figure 1. Seismic zones of Taiwan suggested in old and new versions of SDC.

3 SEISMIC STABILITY ANALYSIS

A pseudo-static-based multi-wedge method which was developed by Huang et al. (2003) is used in the following analyses. The multi-wedge failure mechanism, as shown in Fig. 3, includes a two-wedge failure (wedge F & wedge B) behind the wall, a sliding failure along the base of wall and a passive failure in front of the wall (Wedge P). Based on the limit equilibrium for all wedges, a factor of safety against horizontal sliding at the wall base, Fs, can be derived:

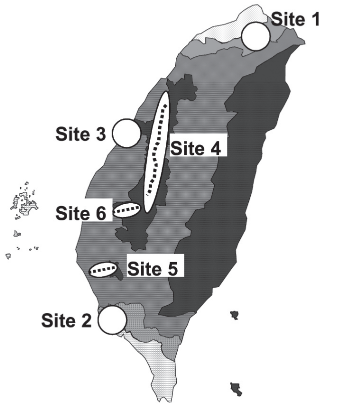

Figure 2. Six representative sites in new version of SDC.

$$F_s = (S_f + P_{PH})/(P + W_W \_ k_h) \quad (1)$$

Where,

- k_h : Horizontal seismic coefficient (= a_h/g, a_h : horizontal ground acceleration, g: gravitational acceleration)
- S_f : Ultimate shear resistance of soils beneath the wall ($S_f = P_{bv} \times \tan \phi_b + c \times B$, ϕ_b: soil friction angle at the base of the wall, c : cohesion of soil, B: width at the base of the wall base, P_{bv}: normal force acting on the wall base, P_{bv}:
- P_{PH} : Horizontal component of dynamic passive earth resistance (P_p) in front of the wall based on limit equilibrium of wedge 'P' using an input seismic coefficient, k_h.
- P_{FH} : Horizontal component of seismic active earth pressure behind the wall (P_F) based on limit equilibrium of wedges 'B' and 'F' using an input seismic coefficient, k_h.
- W_W: Weight of retaining wall.

The seismic active earth pressure and seismic passive earth pressure calculated above are verified using the Mononobe – Okabe theory (Mononobe, 1924 and Okabe, 1924).

Table 1. Comparisons of different levels of design ground accelerations specified in old and new SDC for major bridges.

Site	Area	Old SDC (g)		New SDC (g)			New/Old (%)		
		$30^{(1)}$ years	$475^{(2)}$ years	$30^{(3)}$ years	$475^{(4)}$ years	$2500^{(5)}$ years	(3)/(1)	(4)/(2)	(5)/(2)
1	Taipei Basin	0.054	0.276	0.069	0.288	0.384	128	104	139
2	Kaohsiung City	0.054	0.276	0.088	0.370	0.475	163	134	172
3	Taichung City	0.077	0.396	0.091	0.384	0.480	118	97	121
4	Che-Lung-Pu Fault	0.077	0.396	0.112	0.472	0.600	145	119	151
5	Hsin-Hwa Fault	0.077	0.396	0.112	0.472	0.619	145	119	156
6	Mei-Shan Fault	0.077	0.396	0.125	0.526	0.624	162	133	158

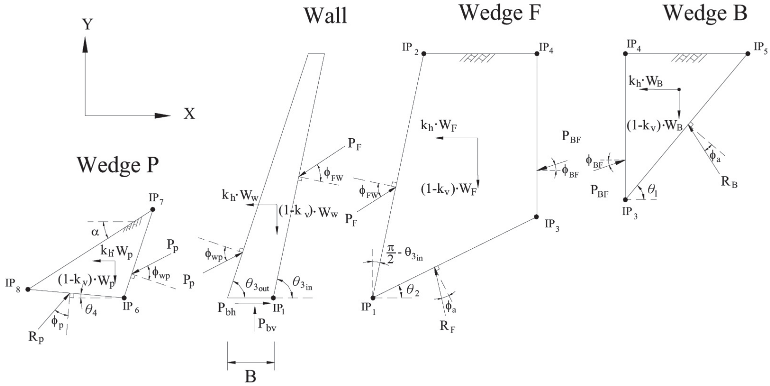

Figure 3. Forces used in the active and passive failure analyses.

4 SEISMIC DISPLACEMENT OF GRAVITY-TYPE BRIDGE ABUTMENTS

Table 2 summarizes safety factors against sliding and overturning under static and seismic ($k_h = 0.2$ and k_v $\frac{2}{3} = k_h = 0.13$) conditions for the gravity-type bridge abutments as shown in Fig. 4. For seismic loading condition, the instability of the abutment is triggered by the sliding failure and this abutment is only marginally stable for the seismic condition when the passive resistance in front of the wall is not taken into account. The seismic displacement calculations are based on the value of k_{h_c} obtained in Fig. 5, in conjunction with the sliding-block theory proposed by Newmark (1965). Four ground acceleration records obtained in 1999 Chi-Chi, 1995 Hyogoken-Nambu, 1989 Loma Prieta and 1940 EI-Centro earthquakes are used. It can be seen in Fig. 5 that the influence of passive resistance to the values of k_{h_c} is significant (k_{h_c} = 0.265 vs. k_{h_c} = 0.152). Importance of the passive resistance to the seismic displacement of soil retaining wall has also been pointed out by Huang and Chen (2004) and Huang (2005) in post-earthquake investigation on two collapsed leaning-type soil retaining walls used to support highway embankments

Table 2. Results of static and seismic stability analyses for the gravity-type bridge abutment.

	Static condition* (DL = 0, LL = 0, q > 0)*		Seismic condition** (LL = 0, DL > 0, q = 0) *	
	$P_p = 0$	$P_p > 0$	$P_p = 0$	$P_p > 0$
F_s against horizontal Sliding	1.7	4.6	1.1	1.3
Gravity-type F_s against overturning	4.1	4.7	1.6	1.6

* $\delta_{s\text{-}s} = \phi_s$ and $\delta_{s\text{-}c} = \frac{\phi_s}{2}$ ($\delta_{s\text{-}s}$: friction angle on the vertical line above the heel,
$\delta_{s\text{-}c}$: friction angle on the soil-concrete interface, ϕ_s : internal friction angle of soil)
** $\delta_{s\text{-}s} = \phi_s$ and $\phi_{s\text{-}c} = \delta_s/2$
• DL and LL : dead load and live load, respectively applied at the seat of bridge abutment; q : uniform surcharge applied on the top of approach embankment

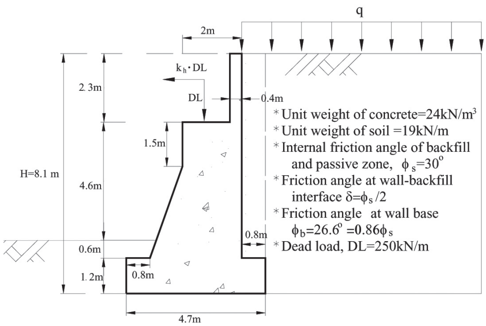

Figure 4. Cross-section of the gravity abutment analyzed in the present study.

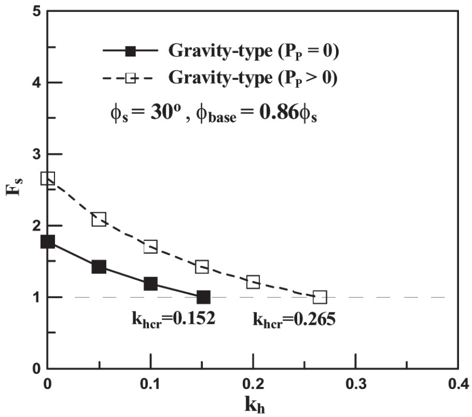

Figure 5. F_s vs. k_h relationships for the gravity-type abutment.

in slope areas. In practical design of soil retaining structures, the passive resistance at the toe of the wall is usually ignored. This may be misleading that the passive resistance to the stability of the wall is not important and a regular examination on the integrity of the passive zone is not necessary. However, the great influence of passive resistance to the values of khc and also to the seismic displacements to be discussed later indicates that the insurance of the integrity of the passive zone is vital to the reduction of seismic displacement of free-standing gravitytype bridge abutments.

It is also noted that in the following displacement calculations a single value of internal friction angle of soil, i.e., $\phi_s = 30°$ is used. This value of ϕ_s may represent residual strength of the soil, inferring that the displacements of the walls calculated in the following are conservative.

Figure 6 and Table 3 show the calculated seismic displacements of the gravity-type abutment. Significant differences in the δ_h when using different earthquake events can be observed despite that maximum horizontal ground acceleration (a_{max}) in each record has been scaled to the same $HPGA_{design}$. Table 3 shows that a maximum δ_h = 636 mm is obtained for Site 6 using the Chi-Chi record under $HPGA_{design}$ = 0.526 g for the level 2 earthquake. This value is much greater than the permissible displacements suggested by Wu and Prakash (1996), i.e., δ_h = 162 mm (= 2% × H, H = total height of wall). Table 3 also shows that δ_h = 1096 mm is obtained at Site 6 for $HPGA_{design}$ = 0.624 g for level 3 earthquake. This value greatly exceed the failure limit (δ_h/H = 10%) suggested by Wu and Prakash (1996), indicating possible damage in a major earthquake for the gravity – type abutments when passive resistance is not available could be catastrophic. Significant differences in δ_h for $P_p = 0$ and $P_p > 0$ conditions in Table 3 suggest that a routine integrity examination for the passive zone of gravity-type bridge abutment is important in mitigating possible disaster in a major earthquake.

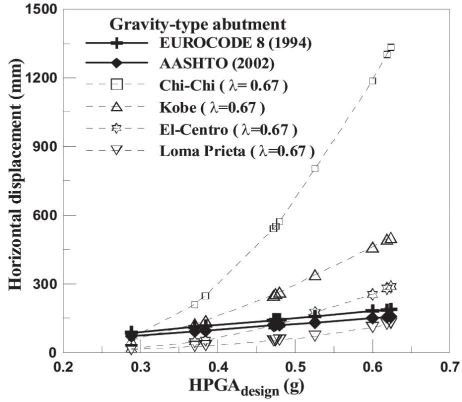

Figure 6. Horizontal seismic displacement of gravity-type abutment subject to level 3 $HPGA_{design}$ of Chi-Chi earthquake with different λ values.

5 SEISMIC BEHAVIOR OF REINFORCED WALLS

Figures 7(a)-7(c) show schematically the test results of a series of shaking table tests on a leaning-type wall (LW), a leaning-type wall reinforced with large-diameter soil nails (R-LW) and a geosynthetic reinforced soil wall with full-height rigid panel facing (RW). The model test RW was used to simulated a geosynthetic-reinforced wall which survied the 1995 Hyogoken-Nambu earthquake with a small

Table 3. Calculated seismic horizontal displacements of gravity-type Bridge Abutments**.

SDC	Sites No.	Sites	$HPGA_{design}$ (g)	Calculated seismic horizontal displacement, δ_h(mm)			
				Chi-Chi*	Hyogoken-Nambu*	El-Centro*	Loma-Prieta*
Old SDC	–	Taipei & Kaohsiung	0.276	0~29	0~29	0~8	0~8
475 years return period	–	Other Sites	0.396	13~213	14~128	4~47	6~27
	1	Taipei Basin	0.288	0~39	0~37	0~11	0~10
	2	Kaohsiung City	0.37	7~147	8~99	2~34	4~21
New SDC	3	Taichung City	0.384	9~176	11~113	3~40	5~24
475 years return period	4	Che-Lung-Pu Fault	0.472	44~422	46~214	13~90	13~42
	5	Hsin-Hwa Fault	0.472	44~422	46~214	13~90	13~42
	6	Mei-Shan Fault	0.526	92~636	81~292	23~138	20~59
New SDC	1	Taipei Basin	0.384	7~176	11~113	3~39	5~24
2500 years return	2	Kaohsiung City	0.475	46~432	47~218	13~92	13~43
period	3	Taichung City	0.48	49~449	50~224	14~97	14~44
	4	Che-Lung-Pu Fault	0.6	179~967	134~400	43~209	30~85
	5	Hsin-Hwa Fault	0.619	209~1069	150~430	50~230	33~94
	6	Mei-Shan Fault	0.624	218~1096	154~443	51~235	34~96

* a_{max} in the horizontal ground acceleration record was scaled to $HPGA_{design}$
** Calculations based on $\phi_s = 30°$, $\phi_{base} = 0.86\ \phi_s$, $k_{h_{cr}} = 0.152$, $\lambda = 0$, $P_P = 0$

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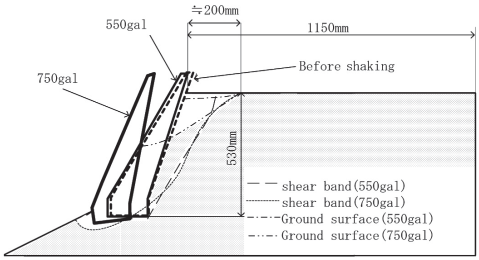

Figure 7. (a) Displacements and shear bands observed in the shaking test on a gravity wall (LW).

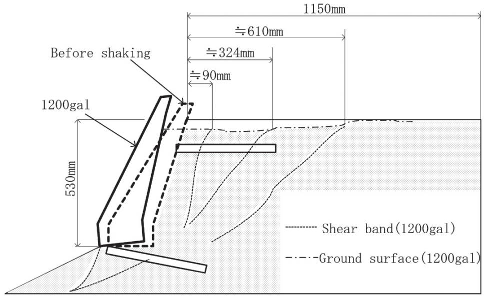

Figure 7. (b) Displacements and shear bands observed in the shaking test on a gravity wall reinforced using large-diameter soil nails (R-LW).

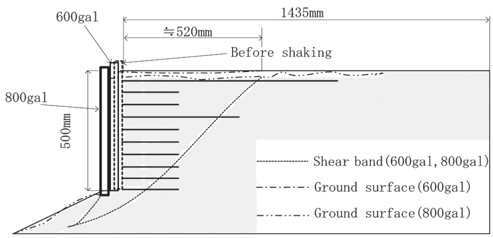

Figure 7. (c) Displacements and shear bands observed in the shaking test on a geosynthetic-reinforced wall with full-height panel facing (RW).

displacement (Tatsuoka et al., 1996, 1998). This series of tests were performed at JRTRI and the details of the tests have been reported by Huang et al. (2000) and Kato (2001). A scaled horizontal ground acceleration record obtained at Kobe Ocean Meteorogical Observatory during the 1995 Hyogoken-Nambu earthquake was used as the input base motions in the step shaking tests (Fig. 8). Comparing Figs. 7(a)-7(c), it is clear that the formations of shear bands in the soil were significantly impeded by using soil reinforcement technique. Table 4 compares measured horizontal displacement of the walls at the end of every step-shaking. It can be seen that for a similar value of δ_h/H = 3.8-5.2%, the required peak ground accelerations (a_{max}) were substantially increased by soil nailing and geosynthetic reinforcement methods.

Table 4. Lateral displacements measured at the top of wall at the end of various shaking stages.

a_{max}(g) and shaking stage	δ_h, (δ_h/H) LW	δ_h, (δ_h/H) RW	δ_h, (δ_h/H) R-LW
≅ 0.56-0.61 (4th stage)	26 mm, (0.052)*	9 mm, (0.018)	6 mm, (0.012)
≅ 0.76 (6th stage)	152 mm, (0.304)	20 mm, (0.04)*	13 mm, (0.026)
≅ 0.92 (8th stage)	–	46 mm, (0.092)	19 mm, (0.038)
≅ 1.12 (10th stage)	–	–	30 mm, (0.06)
≅ 1.22 (11th stage)	–	–	42 mm, (0.084)
≅ 1.33(12th stage)	–	–	73 mm, (0.146)

* : shear bands in the backfill and the foundation appear at this stage of shaking

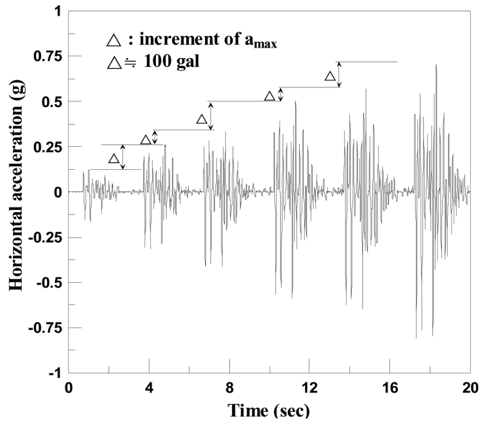

Figure 8. A scaled input horizontal ground acceleration used in the step shaking tests

This test result strongly suggests the use of soil nailing for reinforcing existing gravity-type retaining walls and bridge abutments which have unallowable displacements under the expected levels of ground shaking. Table 4 also shows that for an input a_{max} ≒ 0.56-0.61 g which approximates a level 2 earthquake in new SDC, the values of δ_h/H for RW and R-LW are 23%–35% of that for LW. For an input a_{max} ≒ 0.76 g which may represent a level 3 earthquake in new SDC, the values of δ_h/H for RW and R-LW are only 8%-13% of that for L/W, and the values of δ_h/H for RW and R-LW are far below the failure criterion (δ_h/H = 10%) suggested by Wu and Prakash (1996). On the contrary, the value of δ_h/H = 0.304 for LW is far beyond the failure criterion mentioned above, indicating the effectiveness of soil reinforcement techniques in reducing the seismic displacement of soil retaining walls. This result also suggests the use of geosynthetic reinforcement for new bridge abutments to be built in seismically active areas.

6 CONCLUSIONS

A multi-wedge method is used to calculated seismic displacement of a typical gravity-type bridge abutment based on design ground accelerations specified in old and new versions of seismic design codes in Taiwan. A series of shaking table tests on various soil retaining model walls is also investigated. The following conclusions can be drawn:

(1) Results of a comparative study on old and new aseismic design codes in Taiwan show that 18%-63% increases of design horizontal ground accelerations is used in the new code. This suggests that many free-standing gravity-type bridge abutments designed based on the old guidelines may require pre-earthquake reinforcement for reducing possible unallowable displacement in level 2 and 3 earthquakes.
(2) Based on the displacement calculations for some areas with geographical importance using four acceleration records with peak acceleration scaled to the level 2 horizontal ground acceleration, the maximum horizontal seismic displacement calculated for the gravity-type bridge abutment is greater than the permissible range proposed in the literatures; for the level 3 horizontal ground acceleration, the maximum calculated displacement is far beyond the failure criterion suggested in the literature.
(3) Both large-diameter soil nailing and geosynthetic-reinforcement techniques can be used for existing and/or newly built gravity-type abutments to reduce possible damage induced by level 2 and level 3 earthquakes in seismically active areas.

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