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Gravity Retaining Walls: Reinvented

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ABSTRACT

Gravity retaining walls and in particular quay walls have repeatedly experienced large displacements during past earthquakes, leading to substantial damages to the facilities and infrastructure built on their backfill. As the majority of the damaged gravity walls have had a vertical back-face, the seismic performance of gravity walls with broken-back geometries are explored in this paper. Pseudo-static limit equilibrium analyses are used to calculate the lateral earth pressures on broken-back retaining walls for evaluating their stability in sliding and overturning. A cost reduction analysis is subsequently conducted for comparing the external stability and the efficiency of broken-back walls with those of vertical-back walls. The results indicate that a broken-back wall could be designed at a significantly reduced cost while maintaining sliding and overturning stability of a wall. These characteristics can be used to design seismically-resistant gravity retaining walls and mitigate earthquake damages.

Introduction

Gravity retaining walls are indispensable elements of most important infrastructures. However, many of these structures have experienced large displacements during past earthquakes, resulting in damages to the structures built on their backfill. For example, severe damage incurred to the quay-walls during the 2003 Lefkada, Greece earthquake (Gazetas, et al., 2005), or following the 1995 Kobe, Japan earthquake in which displacement and overturning of waterfront quay-walls caused significant backfill settlement and damage to offshore facilities (e.g., cranes and rail tracks). Recently, the massive March 2011 Tohoku earthquake led to large lateral displacement of many seawalls at northern Japan. These walls were backfilled with coarse rock fill material and no liquefaction occurred in their backfill (Takahashi, et al., 2012). Improving the seismic performance and stability of gravity retaining walls is vital for reducing such damages associated with wall displacement, particularly with the more frequent occurrence of larger magnitude earthquakes.

Lateral movement of a retaining wall primarily results from the increased seismic earth pressure applied on the wall (Dakoulas and Gazetas, 2008, Pitilakis and Moutsakis, 1989, Seed and Whitman, 1970). One particular approach for reducing lateral earth pressures is to minimize the size of the failure wedge developed behind a wall. This can be simply accomplished by modifying the back-face shape of a wall. As illustrated in Figure 1, compared to a vertical-back wall (Fig. 1b), a landward-leaning wall (with a negative batter) is subject to a smaller backfill failure wedge and therefore a smaller lateral thrust. On the other hand, a larger failure wedge and lateral thrust develop behind an outward-leaning (battered) wall. Compared to typical vertical-

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back walls, a large landward-leaning gravity wall can be considerably more expensive as it would require large volumes of materials (e.g., concrete and steel reinforcement) to build, and would attract greater inertial forces during earthquakes due to its larger mass. A combination of landward- (in Fig. 1a) and outward- (in Fig. 1c) leaning rear-face segments as in a broken-back wall can alleviate these undesirable aspects. While taking advantage of the reduced lateral earth pressure on the landward-leaning section in a broken-back wall, material volume and wall weight are reduced by using an outward-leaning rear-face at shallow depths where lateral earth pressures are less significant.

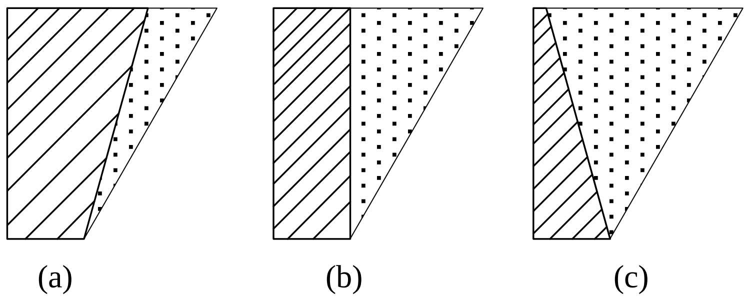

Figure 1. Wall shape and the corresponding backfill failure wedge for (a) landward-leaning wall, (b) vertical-back wall, and (c) outward leaning wall (hatched area: wall material; grey area: backfill failure wedge)

Broken-back gravity retaining walls are easily constructed by dry-stacking of segmental concrete blocks. The concrete blocks are secured through the interface frictional forces and shear keys among them. Despite the construction of these retaining walls, a systematic investigation on the potential advantages of broken-back walls for improved external stability and seismic design is missing. To the authors knowledge, only Sadrekarimi et al. (2008) have investigated the seismic performances of two types of broken-back retaining walls using a number of reduced-scale 1g shaking table model experiments. Figure 2 shows the schematics of these model walls. The total height of each wall was 44 cm and they were backfilled with fine gravel-sized crushed limestone particles at a unit weight (γ) of 18.7 kN/m^3.

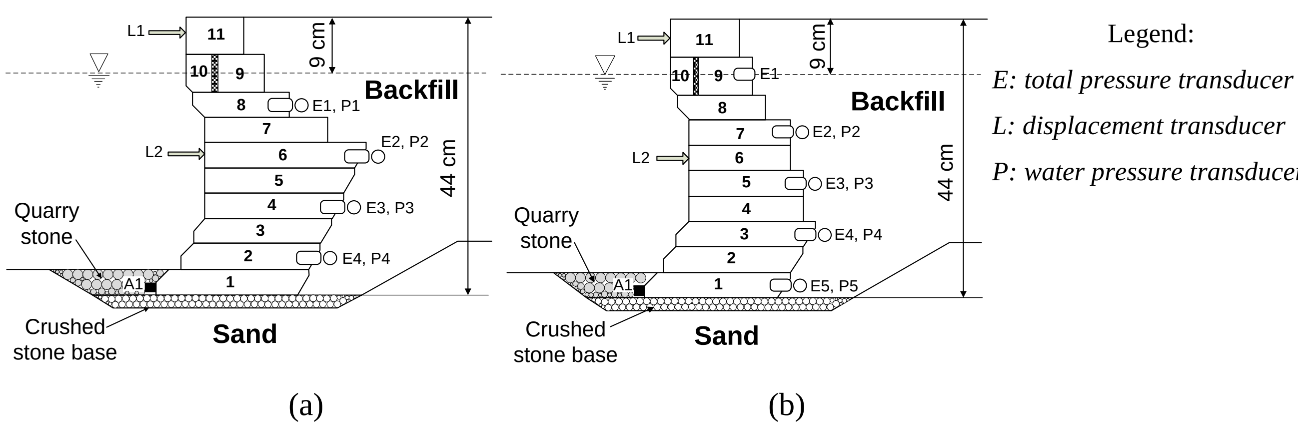

Figure 2: Schematics of the reduced-scale broken-back wall models: (a) type I and (b) type II tested by Sadrekarimi et al. (2008)

This paper presents a parametric study carried out using limit equilibrium analysis to investigate the effects of wall back-face geometry on seismic lateral earth thrust and overturning moment. The results are then employed for comparing sliding and overturning stability of broken-back and vertical-back walls.

Limit Equilibrium Parametric Analyses

Design of a gravity retaining wall requires ensuring stability against sliding, and overturning modes of failure. The approach used to evaluate each criterion is discussed in the following paragraphs. The pseudo-static limit equilibrium analysis of Mononobe-Okabe (Mononobe and Matsuo, 1929, Okabe, 1924) - referred to as the M-O method hereafter - is often used to estimate seismic lateral earth pressures (p_{AE}). In this method, the dynamic force is considered as an equivalent pseudo-static inertial force by applying the earthquake load as a uniform coefficient of the weight of the active failure wedge. Despite some limitations (Kramer, 1996, Nakamura, 2006), the M–O method is widely used in building codes and engineering guidelines (Anderson, et al., 2008, ASCE, 2000, ATC, 1978, CSA, 1998, EAU 1996, 2000, IBC, 2006, International Navigation Association, 2001) and it is the standard practice in the seismic design of retaining structures (Mylonakis, et al., 2007). The extensive application of this analytical method is often because of its simplicity and the reasonable prediction of seismic earth pressures (Dakoulas and Gazetas, 2008, Mononobe and Matsuo, 1929, Pitilakis and Moutsakis, 1989, Seed and Whitman, 1970, Steedman and Zeng, 1990).

Based on the suggestion of Wu (1976), Sadrekarimi (2010) used the M-O method for calculating pseudo-static lateral earth pressures on broken-back retaining walls by dividing the backfill into a number of horizontal slices and calculating the M–O earth pressure in each of the slices. Therefore, the lateral earth pressure distribution was determined by computing the earth pressures at different depths along the wall height. Sadrekarimi (2010) obtained close agreement between the M-O estimates with those measured in reduced-scale shaking table model tests on broken-back retaining walls (Sadrekarimi, et al., 2008). The comparisons indicated that the M–O method was in general capable of predicting the magnitude and distribution of the horizontal active pressure measured in the model tests. The analytical approach used by Sadrekarimi (2010) is implemented in a spreadsheet here for a parametric study of broken-back walls performance in sliding and overturning stability. The backfill soil is divided into a number of 0.5 meters thick segments, and the lateral earth pressure in each segment is calculated using the M-O method. Similar to the experiments of Sadrekarimi et al. (2008), a granular backfill with γ = 18.7 kN/m^3 and ϕ' = 34^o is used in these analyses. It is assumed that the backfill and foundation soils are homogeneous and isotropic free-draining materials that would not liquefy under seismic loading conditions. Even if liquefaction occurs, the comparisons made in this study for different wall geometries' performance in sliding, and overturning would be still valid. The analysis further considers inertias of the backfill and foundation soils as well as the wall structure with the same coefficient of horizontal acceleration (k_h). Figure 3 presents the schematic of the broken-back wall modeled in this study and the forces acting on the wall. According to this figure, α_2 and Σ are the inclinations of the wall's rear-face and p_{AE} from horizontal, respectively. Parameters RH = h_1/h_2 and α_2 are used to characterize wall shape in this parametric study for different combinations of RH = 0.0, 0.25, 0.5, 1.0, 2.0, and 4.0, and α_2 = 90^o to 150^o.

In order to ensure geometrical compatibility, the angle of the outward-leaning rear-face segment of the wall with horizontal (α_1) is calculated as a function of RH and α_2. Stress and moment calculations are carried out for walls with a base width (B) of 4 m and a total height (H) of 10 m ($H = h_1 + h_2$). Although earthquakes with peak ground accelerations (PGA) larger than 0.30g also occur, from the examination of damage to 129 gravity retaining walls in past earthquakes and due to the transient nature of PGA, the maximum seismic earthquake coefficient (k_h) that represents the effects of an earthquake ground motion on a retaining wall is generally less than 0.25 (Nozu, et al., 2004). Accordingly, the analyses of this study are presented for a peak k_h = 0.25. Limiting $k_h \leq 0.25$ ensures that the backfill failure plane remains steeper than the ground surface and thus the resulting failure wedge does not become infinite. The vertical component of an earthquake record is not considered in this study as it is often much smaller (except at the epicentral area of an earthquake) than the horizontal acceleration component for most earthquakes (Seed and Whitman, 1970), and rarely peaks at the same time as the horizontal ground acceleration.

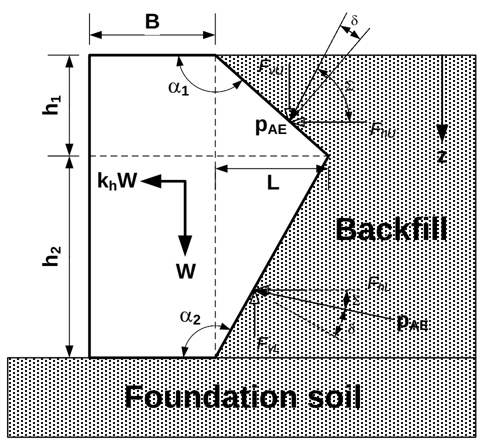

Figure 3: Schematic of the broken-back wall model and applied forces analyzed in this study

Horizontal Thrust

As illustrated in Figure 1, because of the larger and smaller failure wedges developed respectively behind the outward-leaning and landward-leaning (battered) segments of a broken-back wall, lateral earth pressures would become larger than those on a vertical-back wall (α_2 = 90^o) at the upper segment of a broken-back wall where the wall leans outwards, and smaller at the lower segment where the wall leans towards the backfill. The horizontal thrust (F_H) applied by the backfill soil is obtained by calculating the area of the lateral stress distribution diagram. Figure 4a demonstrates the variation of F_H with wall back-face geometry for walls with B = 4 m subject to k_h = 0.25 seismic acceleration. Note that RH = 0 corresponds to a landward-leaning (battered) wall. For any α_2, F_H decreases with increasing the size of the landward-leaning segment (decreasing RH). Except for outward-leaning walls (RH = *inf*), F_H decreases with increasing α_2 as the size of the backfill failure wedge shrinks. This reduction is greater for wall geometries with a larger landward-leaning segment (smaller RH).

Except for landward-leaning walls (RH = 0), α_1 and α_2 increase simultaneously in order to maintain geometrical closure of the wall shape. However, with increasing α_2 beyond a certain angle, the size of the failure wedge on the outward-leaning segment of a broken-back wall grows faster than the shrinking size of the failure wedge behind the landward-leaning section of the wall. Consequently, the total size of the failure wedge and hence F_H increase at larger magnitudes of α_2. For outward-leaning walls (RH = *inf*), α_2 is taken as α_1 and hence the size of the failure wedge and F_H increase with increasing α_2. Based on the analyses of Figure 4, F_H can be minimized by carefully designing the shape (RH and α_2) of a broken-back wall. Figure 4 also presents F_H calculated for the broken-back walls of Figure 2. Among these walls, wall type I provides a greater reduction in F_H compared to a vertical-back wall.

Overturning Moment

The overturning moment produced by the backfill soil (M_H) is obtained by multiplying F_H with the distance of its point of application (centroid of the pressure distribution diagram) from wall's toe. As shown in Figure 4b, M_H slightly increases (except for an outward-leaning wall, RH = *inf*) and then decreases with increasing α_2 irrespective of RH. The initial increase in M_H results from the elevated application point of F_H as the horizontal stress on the upper outward-leaning segment increases compared to that on the lower landward-leaning segment of the wall. This doesn't occur in an outward-leaning wall (RH = *inf*) as there is no landward-leaning segment and F_H is theoretically applied at the lower one-third of the wall. With further increasing of α_2, M_H reduces as F_H and its application point drop. Except for a landward-leaning wall (RH = 0), the stabilizing moment produced by the downward vertical component of p_{AE} on the outward-leaning segment of a wall (i.e. F_{vU} in Fig. 3) increases with increasing α_2 (and thus α_1) which further contributes to M_H reduction. As discussed earlier, given that F_H decreases by expanding the lower landward-leaning segment of a broken-back wall (reducing RH in Fig. 4), M_H also decreases with reducing RH from 4 to 0.25. Note that the uplifting vertical component of p_{AE} on a landward-leaning wall (F_{vL}) produces additional overturning moment which is manifested by the initial increasing of M_H for RH = 0 walls. This becomes particularly significant with increasing backfill inertia at large k_h. Among the broken-back walls presented in Figure 2, only wall type I provides some reduction in M_H compared to a vertical-back wall.

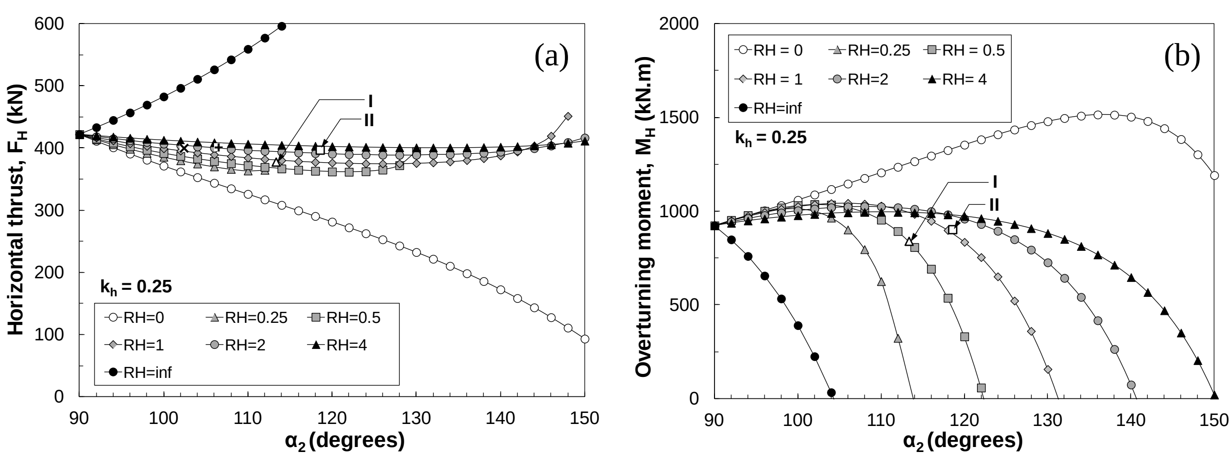

Figure 4: Effect of wall back-face geometry (RH and α_2) on (a) horizontal earth thrust (F_H), and (b) overturning moment (M_H) for k_h = 0.25. I, and II are broken-back walls of Figure 2

Stability Analysis of Broken-back Walls

Sliding failure of a gravity retaining wall is driven by the combined lateral earth thrust (F_H) applied from the backfill soil and wall inertia (F_i) produced by k_h, compared to the sliding resistance mobilized at the wall's base (F_f). Overturning stability of a wall depends on whether M_H and the seismic inertial overturning moment of the wall mass (M_i) could overcome the stabilizing moment of the wall's weight (M_w). Sliding and overturning stability of gravity retaining walls are subsequently compared by evaluating the total driving force ($F_T = F_H + F_i - F_f$) and overturning moment ($M_T = M_H + M_i - M_w$) for H = 10 m reinforced concrete walls (with a unit weight of 24 kN/m^3). For these analyses, the interface friction angle (δ) between the (concrete) wall and the backfill or foundation soil (behind or beneath the wall) is assumed as $0.5\phi'$ (Ichihara and Matsuzawa, 1973, International Navigation Association, 2001). A wall would slide or overturn if $F_T > 0$ or $M_T > 0$, respectively. The effect of passive resistance from the embedment depth in front of a wall is often neglected as a relatively larger wall displacement is required for its mobilization and the overburden soil could also become eroded, excavated, or disturbed. Figure 5 presents the minimum wall widths required to meet sliding ($F_T = 0$), and overturning ($M_T = 0$) stability conditions, respectively for a seismic acceleration of 0.25g. These plots can be used to select the shape (RH and α_2) of a broken-back wall for $k_h = 0.25$ in order to meet either of the stability criteria. According to these plots, a stable broken-back wall requires a smaller B than a vertical-back retaining wall, while (as one would intuitively expect) the minimum B increases with increasing k_h for either wall types. Moreover, sliding ($F_T = 0$) is always the critical stability criterion which requires a larger wall weight and hence B to resist sliding. The trend of minimum B for an outward-leaning wall (RH = *inf*) is particularly interesting. The size of the failure wedge developed behind an outward-leaning wall increases with increasing α_2, particularly when subjected to a high seismic load ($k_h = 0.25$). That's why the minimum width required for sliding stability ($F_T = 0$) of an outward-leaning wall initially increases with increasing α_2 in Figure 5a. However, beyond $\alpha_2 = 122^o$ the base width of the wall and F_{vU} produce adequate sliding resistance (F_f) and thus the required B reduces with further increasing of α_2 (> 122^o).

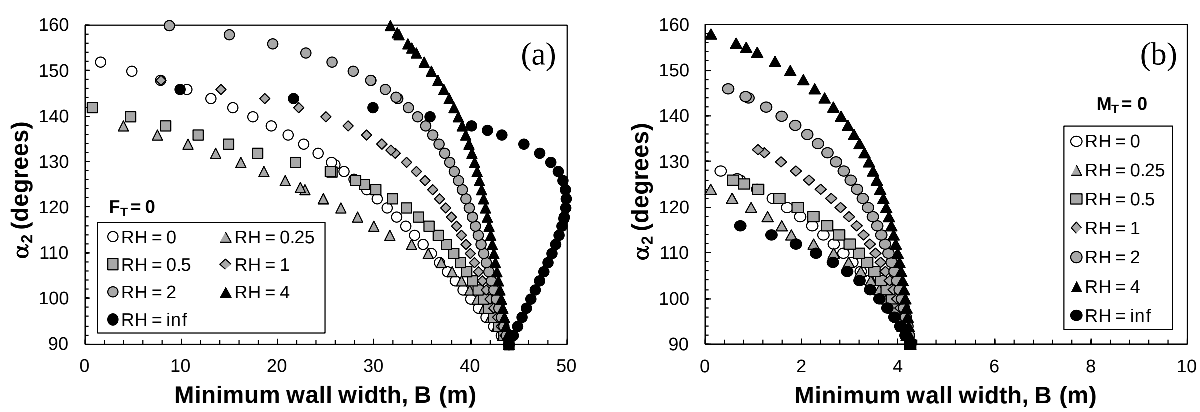

Figure 5: Minimum wall widths required for (a) sliding stability ($F_T = 0$), and (b) overturning stability ($M_T = 0$) at $k_h = 0.25$

Cost-based Evaluation

Material cost (concrete and reinforcing steel) is often a key factor in determining the choice of a soil retaining system. The percentages of cost reductions (per unit wall length) that could be realized by replacing a vertical-back wall with a broken-back wall are presented in Figure 6 for meeting each stability criterion. The amount of cost reduction is estimated based on the difference between the cross sectional areas of broken-back and vertical-back walls. The calculations are carried until a maximum cost reduction or B = 0 is reached. According to these plots, broken-back walls can provide significant material reduction and cost savings compared to vertical-back walls while presenting the same or even improved level of stability. For a certain RH, the size of the backfill failure wedge decreases with increasing α_2 (particularly for walls with smaller RH) resulting in the reduction of the minimum wall area required for each stability criterion and therefore increased cost reduction. For example, a broken-back wall with $\alpha_2 = 128^o$ and RH = 0. 5 can provide a savings of about 36% in construction material costs while ensuring both sliding and overturning stability for a design acceleration of 0.25g. Greatest benefits of broken-back walls are displayed in sliding stability for which cost reductions of up to 90% are obtained. In general, broken-back walls with smaller RH (= 0.25 and 0.5) provide relatively greater cost reductions because of the larger proportion of their landward-leaning segment and the smaller size of the resulting backfill failure wedge. Note that an outward-leaning wall becomes profitable only with relatively large magnitudes of α_1 (> 135^o), creating a wide base width.

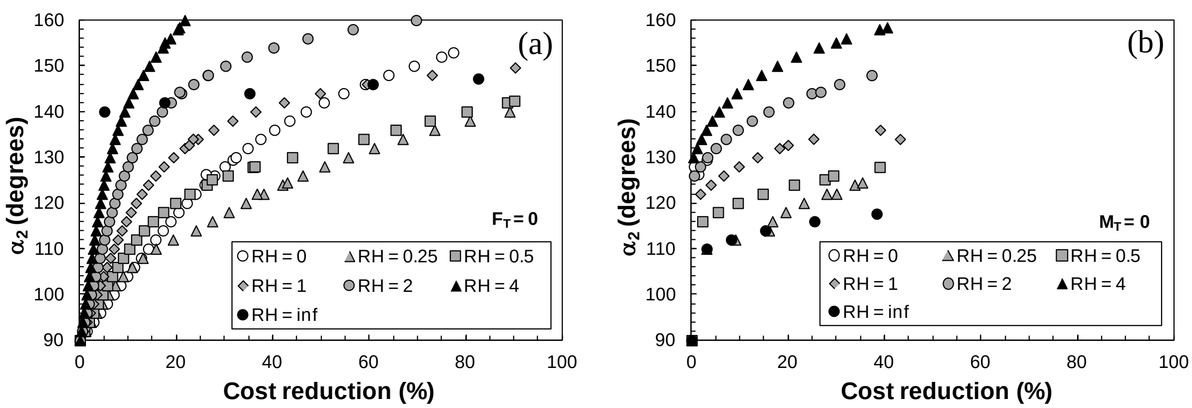

Figure 6: Amounts of cost reductions provided per unit length of broken-back walls for meeting (a) sliding, and (b) overturning stability criteria compared to a vertical-back wall at $k_h = 0.25$

Conclusions

The analyses results presented in this paper indicate that the total horizontal thrust and overturning moment on a broken-back wall decrease with increasing the size of the landward-leaning segment. The overturning moment further tends to slightly increase and then decrease with reducing the slope of the landward-leaning segment. These result in a smaller wall width required for the external stability of a broken-back wall with a flatter landward-leaning rear-face. Seismic stability of a broken-back retaining wall in comparison to that of a vertical-back wall is further improved as its center of gravity is drawn landwards, thereby increasing the stabilizing moment and developing a higher frictional resistance at its base with the foundation soil. Therefore, by merely adjusting wall geometry more stable and less voluminous retaining walls can be developed. As demonstrated in this study, broken-back retaining walls can provide up to about 90% reduction in the volume of construction material and therefore material cost compared to vertical-back walls. A properly designed broken-back retaining wall can provide improved safety and stability in the seismic design of ports, harbours, land development or highway bridge abutments as a more efficient gravity-type retaining wall for mitigating earthquake hazards. The analytical procedure described in this paper can be used to design a broken-back wall shape with optimum external stability while maintaining its economical efficiency.

References

Anderson, D. G., Martin, G. R., Lam, I., and Wang, J. N. (2008). "Seismic analysis and design of retaining walls, buried structures, slopes and embankments." *National Cooperative Highway Research Program Report 611*, Transportation Research Board, Washington, DC.

ASCE (2000). "Seismic analysis of safety related nuclear structures and commentary." American Society of Civil Engineers, Reston, Va.

ATC (1978). "Tentative provisions for the development of seismic regulations for buildings." Applied Technology Council (ATC), Berkeley, CA.

CSA (1998). "Canadian highway bridge design code." Canadian Standards Association, Mississauga, Ontario.

Dakoulas, P., and Gazetas, G. (2008). " Insight into seismic earth and water pressures against caisson quay walls." *Geotechnique*, 58(2), 95 - 111.

EAU 1996 (2000). "Recommendations of the Committee for Waterfront Structures, Harbours, and Waterways.", Ernst & Sohn, Berlin, Germany.

Gazetas, G., Anastasopoulos, I., and Dakoulas, P. "Failure of harbour quay walls in the Lefkada 2003 earthquake." *Proc., Proceedings of the Geotechnical Earthquake Engineering Satellite Conference, Performance Based Design in Earthquake Geotechnical Engineering: Concepts and Research*, Japanese Geotechnical Society, 62 - 69.

IBC (2006). "International Building Code." International Code Council, Country Club Hills, Ill.

Ichihara, M., and Matsuzawa, H. (1973). "Earth pressure during earthquake." *Soils Found*, 13(4), 75 - 86.
International Navigation Association (2001). *Seismic design guidelines for port structures*, A. A. Balkema Publishers, Tokyo.

Kramer, S. L. (1996). *Geotechnical earthquake engineering*, Prentice Hall, Upper Saddle River, N.J.

Mononobe, N., and Matsuo, H. (1929). "On the determination of earth pressure during earthquakes." *World Engineering Conference*Japan, 179 - 187.

Mylonakis, G., Kloukinas, P., and Papatonopoulos, C. (2007). "An alternative to the Mononobe-Okabe equation for seismic earth pressures." *Soil Dyn Earthq Eng*, 27(10), 957 - 969.

Nakamura, S. (2006). "Reexamination of Mononobe-Okabe theory of gravity retaining walls using centrifuge model tests." *Soils Found*, 46(2), 135-146.

Nozu, A., Ichii, K., and Sugano, T. (2004). "Seismic design of port structures." *Journal of Japan Association for Earthquake Engineering*, 4(3), 195 - 208.

Okabe, S. (1924). "General theory on earth pressure and seismic stability of retaining wall and dam." *Journal of Japan Society of Civil Engineers*, 10(6), 1277 - 1323.

Pitilakis, K. D., and Moutsakis, A. (1989). "Seismic analysis and behaviour of gravity retaining walls - the case of Kalamata harbour quay wall." *Soils Found*, 29, 1 - 17.

Sadrekarimi, A. (2010). "Pseudo-static lateral earth pressures on broken-back retaining walls." *Canadian Geotechnical Journal*, 47(11), 1247-1258.

Sadrekarimi, A., Ghalandarzadeh, A., and Sadrekarimi, J. (2008). "Static and dynamic behavior of hunchbacked gravity quay walls." *Soil Dyn Earthq Eng*, 28(2), 99-117.

Seed, H. B., and Whitman, R. V. "Design of earth retaining structures for dynamic loads." *Proc., ASCE Special Conference on Lateral Stress, Ground Displacement and Earth Retaining Structures*, American Society of Civil Engineers, 103 – 147.

Steedman, R. S., and Zeng, X. (1990). "The influence of phase on the calculation of pseudo-static earth pressure on a retaining wall." *Geotechnique*, 40(1), 103 - 112.

Takahashi, H., Oohashi, T., and Endoh, H. (2012). "Earthquake damage investigation of gravity-type quay walls in Ibaraki port." *Proceedings of the International Symposium on Engineering Lessons Learned from the 2011 Great East Japan Earthquake*Tokyo, Japan, 687 - 696.

Wu, T. H. (1976). *Soil mechanics.* , Allyn and Bacon, Boston, Mass.