# Optimized performance of MSE retaining walls subjected to extreme ground shaking

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ABSTRACT

The need to shift geotechnical design from a "factor of safety" to a "performance based" design (PBD) approach has increased rapidly in the past years. According to the PBD philosophy, designers seek for sustainable engineered systems that may result in more rational and less conservative solutions. In the case of retaining walls, the concept of mechanically stabilized earth (MSE) systems has received considerable attention for being a system that combines the above attributes while demonstrating a remarkable performance when subjected to extreme ground shaking. Indeed, MSE walls have performed very satisfactorily during catastrophic recent earthquakes that have caused conventional concrete walls to fail.

In this paper the authors attempt a quantification of the factors affecting the superiority of MSE systems when subjected to severe ground shaking. Emphasis is on the role of dilatancy as the main factor determining the optimization of grids spacing and hence their economic –apart from technical- efficiency. Specific examples demonstrate how the proper design may enhance the overall wall-soil system to be able to sustain seismic excitations well beyond the design earthquake due to the inherent redundancies of such systems

## 1 INTRODUCTION

In early practice, the only criterion for achieving good performance was to avoid failure; designers had only to ensure that the available capacity of the geotechnical system is sufficiently higher than the expected demand (i.e. Load and Resistance Factor Methodologies). On the other hand, during the last 20 years, the examples of recorded earthquakes that overly exceed the design standards keep increasing. Recall the Wenchuan (2008) earthquake, the fatal M8.8 Chile (2010) earthquake and the triple earthquake events that consecutively shook Christchurch New Zealand with recorded accelerations in the order of 1g. Design against such unprecedented seismic actions would certainly impose enormous demands to the structure and would result in a tremendously resource-demanding task.

Even when the key design requirements against collapse are satisfied, the system may still be placed into a condition in which the deformation constraints are not met (Bolton, 2012). To address these issues, geotechnical design has been moving to a predominantly performance-based approach that correlates deformation considerations to the severity of the experienced loading. In earthquake-related problems, the definition of good performance of a geotechnical system is no more unique; for a low intensity earthquake the system should remain serviceable (i.e. the developed deformations should remain low), whilst for the design earthquake the maintenance of life-safety is the sole performance criterion.

Following this very definition of good performance, the paper investigates the seismic behavior of Mechanically Stabilized Earth (MSE) systems. Since the first MSE wall construction in California in the early 70's, MSE systems have become increasingly popular due to their low cost and ease of construction especially when it comes to 'deep cuts'. Moreover, MSE systems have repeatedly documented an excellent seismic performance during major past earthquakes – Loma Prieta 1991, Northridge 1994, Kobe 1995; Chi-Chi 1999; Tohoku 2011 (Chen et al. 2000; Eliahu and Watt 1991; Frankenberger et al. 1996; Huang 2000; Kobayashi et al. 1996; Nishimura et al. 1996; Sitar et al. 1995; Stewart et al. 1994; Tatsuoka et al. 1995; Fang et al. 2003; Kuwano et al. 2014). Tatsuoka et al (1995) examined several types of retaining walls (including masonry, leaning unreinforced concrete, gravity, cantilever and geogrid-reinforced soil walls) after the Kobe 1995 earthquake and concluded that geogrid-reinforced soil walls performed very well compared to conventional walls; many of the latter failed due to a combination of structural and foundation failure. Huang (2000), and Fang et al. (2003) recorded the failure of quay walls, masonry walls, gravity walls and modular-block retaining walls after the Chi-Chi 1999 earthquake. More recently, Kuwano et al. (2014) reported that over 98% of the approximately 1600 Reinforced Soil walls exhibited only light to non-existent damage after the 2011 Tohoku earthquake, although the imposed seismic motion was much higher that the design values.

Anastasopoulos et al (2010) conducted a series of 1-g small scale MSE wall experiments subjected to earthquake motions overly exceeding code specifications. As in the reported case studies, the MSE walls displayed a remarkable ability to sustain seismic disturbance without collapsing. Kourkoulis et al (2016) envisioned a direct comparison of two conventionally equivalent retaining structures - a typical pile-wall system and a MSE wall.

They confirmed that both systems respond equivalently to moderate shaking; yet the MSE system outperforms the conventional retaining structure when it comes to extreme shaking due to its considerably larger deformability.

This paper continues the earlier work of Kourkoulis et al (2016) exploring further the inherent safety margins of an MSE retaining structure offered by:

(a) The constructive role of soil dilatancy, especially in shallower soil layers where the wall deformation is increased

(b) The inherent redundancies of the system

The former is demonstrated through a set of pull-out experiments performed at National Technical University of Athens (NTUA) while the latter is illustrated by means of non-linear FE analyses as shown in the ensuing.

## 2 THE ROLE OF DILATANCY: EXPERIMENTAL DEMONSTRATION

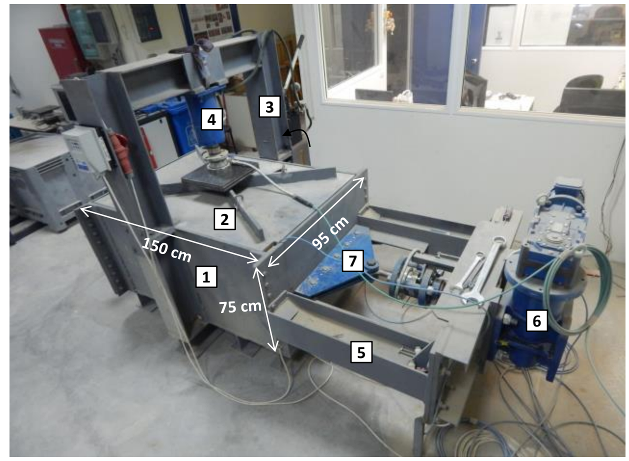
A set of pull-out tests have been performed at the pull-out box of the Soil Mechanics Laboratory at NTUA. Although not extensive at this stage, our test series aim to highlight the mechanics governing the behavior of steel grid reinforcements and identifying the main sources of conservatism in design. Namely, we investigate the effect of (i) soil dilation and (ii) transverse reinforcement on the pull-out resistance of steel-grid reinforcements as a function of overburden pressure.

### 2.1 The Experimental Set-Up

A view of the experimental device is pictured in Fig. 1. It consists of: (i) *a rigid steel box* (W150 cm, L95 cm , D75 cm) that essentially acts as the soil container within which the reinforcement is installed; (ii) *a vertical steel frame* carrying a hydraulic actuator to apply the overburden pressure, (iii) *a rigid steel plate(being placed on the top of the rigid box)* to distribute uniformly the loading from the vertical actuator over the entire soil surface, (iv) *a horizontal frame* carrying an electronically controlled mechanic horizontal actuator used to apply the pull-out force, (v) an appropriately designed *clamp system to* prohibit reinforcement sliding and allow uniform application of the pull-out force along the grid width and (vi) *a sleeve* on the front wall edge extending 15 cm into the soil container (to minimize the front wall effect).

The Pull-out Resistance is measured by means of : (i) *two load cells* (on the vertical and horizontal actuator), (ii) *a crack-meter* controlling the displacement of the horizontal actuator, (iii) LVDT's and linear extensometers measuring the extension of the grid and (iv) two *pressure cells* (installed on the soil layer immediately overlying the reinforcement) to measure the actual soil pressure acting on the reinforcement level. In all cases presented, the soil is the NTUA Sand, classified as a granular A1-b material based on AASHTO guidelines with a $D_{50}$ value at 0.5mm and coefficient of uniformity $D_{60}/D_{10}$ = 2.5. Its relative density equals $D_r$ = 86%, while the critical state friction angle is measured at $\varphi_{cv} \approx 33 - 35^o$.

Inextensible welded steel meshes (with Young's modulus of 200GPa and tensile stress of 500 MPa) were used in the tests. Results are presented below for a grid configuration consisting of 12mm bars at 20cm and 30cm spacing in the longitudinal and the transverse direction, respectively.




1. Rigid Steel Box [ L150xW95xD75 ]
2. Rigid Steel Plate
3. Vertical Steel Frame
4. Vertical Actuator
5. Horizontal Frame
6. Horizontal Actuator
7. Clamping System

Figure 1. View of the NTUA Pull-Out Device

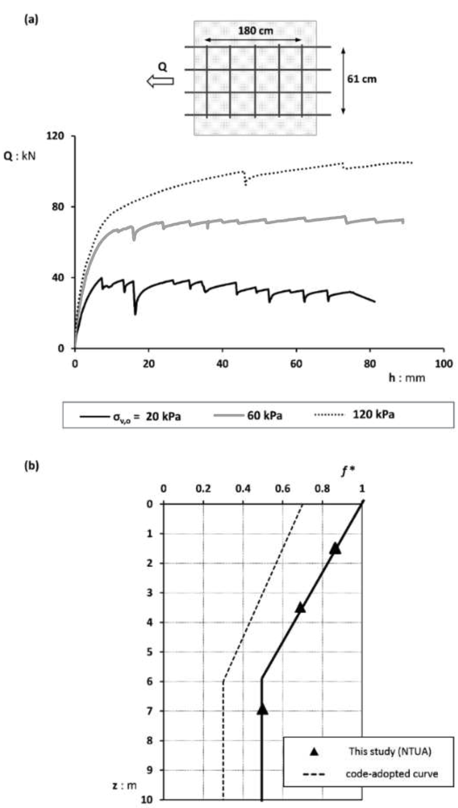



Figure 2. Dependence of Pull-out resistance on overburden stress expressed in: (a) absolute terms and (b) terms of the apparent friction coefficient

2.2 Variation of f* with depth

Figure 2a plots the pull-out force as a function of the applied displacement for three tests on a 4-bay grid of plan dimensions equal to 61x180 cm for 3 values of the overburden pressure, namely 20, 60 and 120 kPa. Evidently the ultimate resistance is attained at a displacement of between 5 and 10 mm and increases with increasing vertical load.

However, the evolution of the apparent friction coefficient f* (defined as $f^* = \tau_{max}/\sigma_v$, where $\tau_{max}$ is the maximum mobilizable shear stress between soil and a reinforcement layer and $\sigma_v$ is the vertical stress at the reinforcement level) demonstrates quite the opposite trend (Fig. 2b).

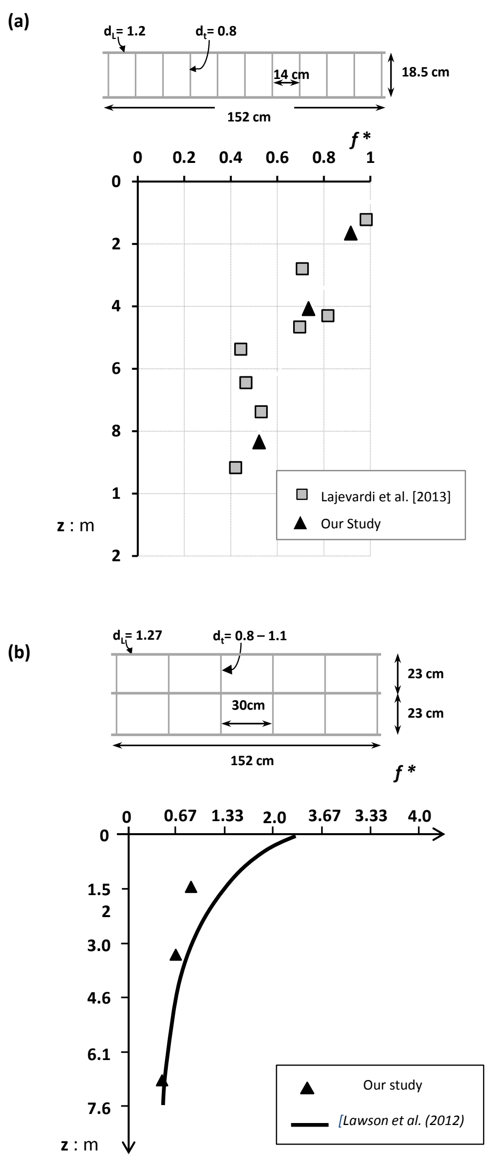



Figure 3. Experimental findings are compared to *Lawson et al. (2013)* and *Lajevardi et al (2013*) results.

Apparently, our experimental results indicate an over-strength of between 35 – 60% compared to code specifications - This observation is in agreement with previous research results by e.g *Lawson et al. (2013)* and *Lajevardi et al (2013)* presented in Fig. 3. The overstrength is attributable mainly to the grid configuration (presence of transverse bars). According to previous studies Suksiripattanapong et al. (2013), the pullout bearing mechanism is essentially controlled by $D_{50}$ and transverse bar spacing S. Considering that larger grain soils (e.g. gravel) are more frequently used in geotechnical projects, the experimental results presented herein rather constitute a lower-bound scenario.

2.3 Influence of Transverse Bars

The pullout interaction mechanisms between soil and geogrid reinforcements is complicated further by the presence of transverse bars. As explained by Teixeira et al (2007), this is because the pullout resistance includes two components: The interface shear resistance that takes place along the longitudinal ribs (and to a lesser extent along the transverse ribs) and the passive resistance that develops against the front of transverse ribs.

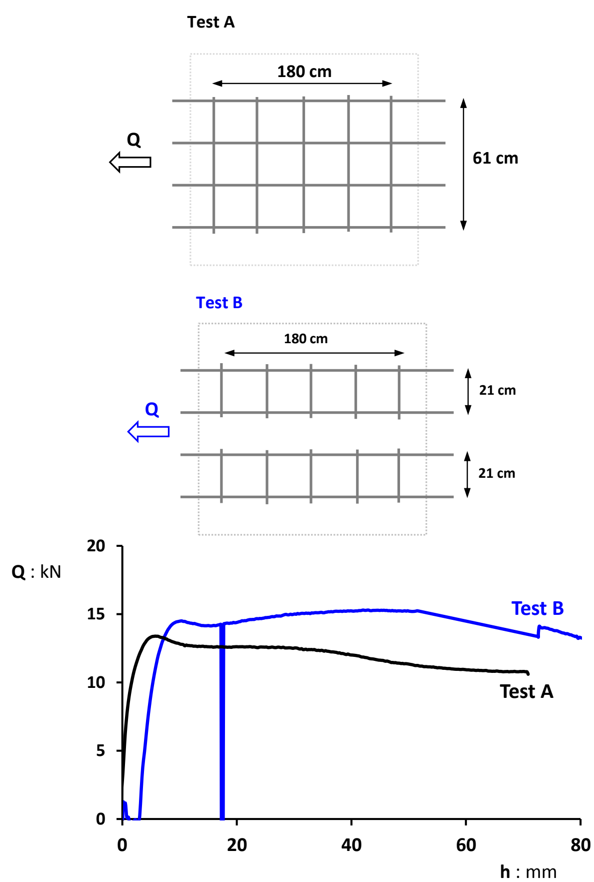



Figure 4, The Effect of transverse bar on the pull-out capacity of geogrids in highly dilatant soils.

In order to distinguish the contribution of these two mechanisms on the pull-out capacity of a steel geogrid, two hypothetical configurations, defined here as Test A and Test B, are analyzed. The two setups have the same number of longitudinal and transversal bars, the same bay geometry and the exact same number and locations of welding points. The only difference is that in Test B all transverse bars are truncated in their middle part thereby creating a system practically consisting of two independent grids.

Figure 4 compares the pull-out force vs displacement curves produced during the two tests under very low confining pressure ($\sigma_v$ = 8 kPa) – when dilation is prevalent. Remarkably, the truncated System (Test B) outperforms Test A despite the lack of the passive resistance on the missing part of its transverse bar. The mechanics explaining this unexpected response are quite interesting. During pullout, the soil tends to dilate over the transverse ribs. As dilation is partially inhibited, the normal stresses tend to increase in the vicinity of the transverse ribs and tend to decrease in between them. Thus, a dense transversal spacing could potentially limit the actual vertical stress developed along pull-out process, leading to an under-exploitation of the grid system. Thus, in the upper layers of a retained embankment the implementation of dense transverse grid configurations may not be optimal.

## 3 MSE WALL RELIABILITY AND REDUNDANCY

Having identified the key stressing mechanisms governing the pull-out performance of steel geogrids in sandy deposits, this section briefly investigates the seismic redundancies of a mechanically stabilized 10m-high backfill. To this end, two grid configurations are investigated. The first one (which is commonly encountered in actual embankments and is herein defined as System A) consists of 7m – long steel grids vertically spaced at 0.6m. In the second set-up (System B) we have preserved only half of the originally prescribed grids (i.e. grid reinforcements are now considered at vertical distances of 1.2m instead of 0.6m). This setup corresponds to a rather hypothetical scenario where half of the required reinforcements are unable to offer any resistance. This scenario is rather unrealistic, but it aims at highlighting the inherent redundancies of such retaining systems, since current seismic design codes do not appear to fully incorporate their flexibility.

A plane strain analysis is conducted in the FE code ABAQUS taking account of material and geometric nonlinearities (Fig.5). The facing panel is modeled by elastic continuum elements of 0.2m in thickness and 0.6m in height. Each facing is connected with its neighboring panels with a pinned connection. The latter allows relative rotation between panels, but restricts the horizontal and vertical degrees of freedom. The steel reinforcement is simulated by means of non-linear truss elements (that may only carry axial force). Reflecting the findings of the previous section, a non-uniform soil-reinforcement friction coefficient is assumed (refer to Fig.2b). Both configurations are excited by the severe Rinaldi time-history (recorded during the Northridge 1994 earthquake). Results are plotted in Fig. 6, revealing a quite astonishing response. The under-designed (and far more compliant) System B configuration sustains the extremely strong shaking by just marginally increasing the horizontal displacements of the more robust configuration.

As portrayed in the figures that follow, the two configurations perform quite differently. In System A, the reinforced soil behaves as an elastic rigid block transferring all straining in the un-reinforced area. On the other hand, System B sustains severe plastic yielding within the “internal” soil, evidenced also by the increased distress along its reinforcement bars. In this case, soil failure acts as an inherent seismic safety fuse which provides the system with redundancies. Therefore, although the nominal capacity of System B is lower, the seismic performance of the two systems is equivalent.

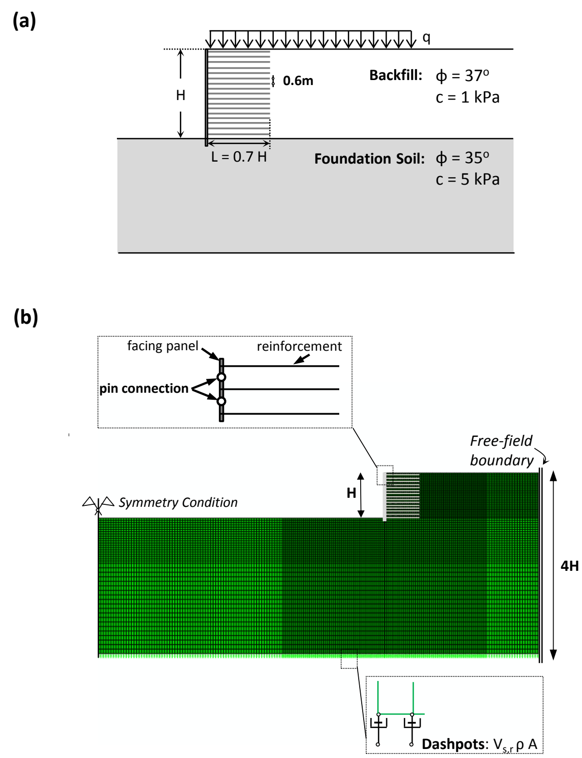



Figure 5. The MSE Retaining Structure: (a) geometry and material properties; (b) the FE configuration (the reinforcement vertical spacing is 0.6m and 1.2m for System A and B, respectively).

## 4 CONCLUSIONS

This paper has presented a preliminary investigation on the inherent (and sometimes concealed during initial design) redundancies of steel-grid reinforced earth retaining systems. Based on a series of pull-out tests performed at NTUA, we have shown that dilatant soils may be responsible for a very considerable increase of the apparent friction coefficient compared to that conventionally estimated. This increase, which may well exceed 100% in most cases, may be accounted for in design thus leading to configurations that are more economical. Dilatancy has also proven to be a decisive factor affecting the optimization of transverse ribs spacing. It has been shown that a dense transversal spacing could potentially limit the actual vertical stress developed along pull-out process, leading to an under-exploitation of the grid system.

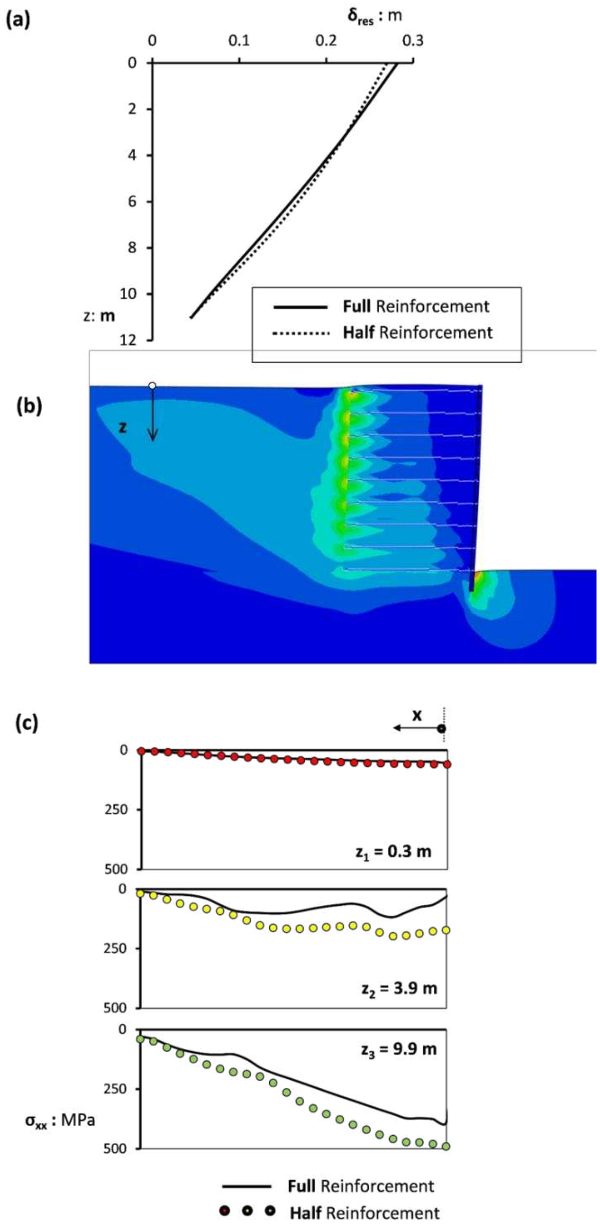



Figure 6. Seismic performance of the two MS Walls (System A and B) when excited by the Rinaldi record: (a) residual displacements along the wall height; (b) plastic strain distribution (of System B) at the end of the shaking ; (c) axial stresses along the rib length at three characteristic elevations ($z_1$, $z_2$ and $z_3$).

The second class of phenomena we examined, were relevant to the purely seismic response of Grid-Reinforced systems. Focusing on the inherent redundancies of such configurations, we have used numerical analyses to show that a severely under-designed and far more compliant System (containing only half of the conventionally required reinforcement grids) may sustain the extremely strong shaking by just marginally increasing the horizontal displacements of the more robust “conservatively designed” configuration. Although not extensive at this stage, our analyses illustrate the significant safety margins potentially offered by Mechanically Stabilized systems.

## 5 REFERENCES

Anastasopoulos I., Georgarakos T., Georgiannou, V., Drosos, V., Kourkoulis, R., (2010), Seismic performance of bar-mat reinforced-soil retaining wall: Shaking table testing versus numerical analysis with modified kinematic hardening constitutive model, Soil Dynamics and Earthquake Engineering, 30, pp. 1089-1105.

Bolton, 2012. Performance-based design in geotechnical engineering, 52nd Rankine Lecture, London, UK.

Chen, T.C., Chen, R.H., Lin, S.S., 2000. A nonlinear homogenized model applicable to reinforced soil analysis. Geotextiles and Geomembranes 18 (6), 349–366.

Ching-Chuan Huang (2000), Investigations of soil retaining structures damaged during the chi-chi (Taiwan) earthquake, *Journal of the Chinese Institute of Engineers*, 23: 417-428

Eliahu, U, and Watt S, (1991), Geogrid-reinforced Walls withstands earthquake, *Geotech. Fabrics. Rep*, 9(2), 8-13.

Fang Y.S., Y.C. Yang, and T.J. Chen, (2003), Retaining walls damaged in the Chi-Chi earthquake, *Canadian Geotechnical Journal*, 40: 1142–1153

Frankenberger, P.C., Bloomfield, R.A. & Anderson, P.L. (1997), Reinforced earth walls withstand Northridge Earthquake, *International Symposium on Earth Reinforcement*, Fukuoka, Kyushu,Japan, 12 – 14 November 1996.

Kobayashi, K., Tabata, H., Boyd, M., (1996), The performance of reinforced earth structures in the vicinity of Kobe during the Great Hanshin Earthquake, *Proceedings of International Symposium on Earth Reinforcement*

Kourkoulis R., F. Gelagoti, I. Georgiou, S. Nikolaou and G Gazetas, (2016), A Resilience Approach for Design of Earth Retaining Systems with Application on MSE Walls, *International Conference of the Deep Foundation Institut*e, New York.

Kuwano, J., Miyata, Y. & Koseki, J. (2014). Performance of reinforced soil walls during the 2011 Tohoku earthquake. *Geosynthetics International*, 21, No. 3, 179–196.

Lajevardi, S. H., Dias, D., and Racinais, J. (2013). Analysis of soil-welded steel mesh reinforcement interface interaction by pull-out tests, Journal of Geotextiles and Geomembranes; 40, pp. 48-57.

Lawson, W.D., Jayawickrama, P.W., Wood, T.A., Surles, J.G., March 3e6, 2013. Pullout resistance factors for steel reinforcements used in TxDOT MSE walls. Paper 417, Geo-Congress 2013. In: Stability and Performance of Slopes and Embankments III. American Society of Civil Engineers Geo-Institute. San Diego, CA.

Nishimura J, Hirai T, Iwasaki K, Saitoh Y, Morishima M, Shintani H, Yoshikawa S & Yamamoto H, (1996), Earthquake resistance of geogrid-reinforced soil walls based on a study conducted following the south Hyogo earthquake, *Proceedings International Symposium on Earth Reinforcement*

Sitar, N., 1995, *Geotechnical Reconnaissance of the Effects of the January 17, 1995, Hyoken-Nanbu Earthquake, Japan*, Report UCBIEERC-951 10 1, Univ. of California at Berkeley, Berkeley, CA.

Stewart, J.P., Bray, J.D., Seed, R.B. and Sitar, N., (1994), *Preliminary report on the principal geotechnical aspects of the January 17, 1994 Northridge earthquake*, Earthquake Engineering Research Center, University of California at Berkeley, Report UCB/EERC-94/08, 1994.

Suksiripattanapong C., Horpibulsuk S., Chinkulkijniwat A., Chai J.C. (2013) "Pullout resistance of bearing reinforcement embedded in coarse-grained soils", Geotextiles and Geomembranes, 36, 44- 54

Tatsuoka, F., Koseki, J. & Tateyama, M. (1995), Performance of Geogrid-Reinforced Soil Retaining Walls During the Great Hanshin-Awaji Earthquake, January 17, 1995, Proceedings, *1st International Symposium on Earthquake Geotechnical Engineering*.

Teixeira, S.H.C., Bueno, B.S. and Zornberg, J.G., (2007), "Pullout Resistance of Individual Longitudinal and Transverse Geogrid Ribs", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 1, January 1, 2007, pp. 37-50

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