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Behavior of anchorage narrow mechanically stabilized earth walls under seismic loading

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ABSTRACT: The rapid increasing and development of infra-structures in Egypt in very limited spaces involves construction of Narrow Mechanically Stabilized Earth (NMSE) walls having an aspect ratio (ratio of reinforcement length, L, to wall height, H) below 0.70. These walls are constructed Infront of existing rigid faces and the top reinforcement layer is fixed to the face by mechanical connections. These walls are subjected to dynamic loads when constructed in active seismically areas. The purpose of this paper is to present a typical -one- reduced scale (1/8 of the prototype model) shaking table test of (L/H) equal 0.30. The model is shaken using stepped sinusoidal base accelerations with incrementally increasing displacement amplitude and constant frequencies to generate an equivalent base acceleration ranging from 0.05 g to 0.80 g or until failure occurs. The frequency of the wave is 2.50 Hz. The results of the tests displays that the yield acceleration of the tested wall is about 0.48g. The results of NMSE walls with reinforcement anchorage resulted in yield acceleration higher than 26 % when correlated to the published NMSE walls without reinforcement anchorage. Finally, 1.25 average amplification factor can be used in the seismic design of NMSE walls of similar configurations.

Keywords: (Shaking Table, NMSE Walls, Reinforcement, Anchorage, Amplification)

1 INTRODUCTION

Nowadays, the fast progression of infrastructures required the overcoming the geometric restrains of MSE walls at locations having steep terrain. Accordingly, NMSE walls are spreading widely as a technique to expand the width of embankments and roadways on slopes that are already stable. The available database of MSE walls with L/H below 0.70 until now include results from full scale field test performed by Morrison et al. (2006) [6], centrifuge modeling parametric studies by Woodruff (2003) [10] and numerical modeling by Yang et al. (2007) [11]. These databases still don't provide comprehensive documentations about the behavior of these walls, specifically, when they are constructed in active seismically zones.

Very rare studies on MSE walls with L/H less than 0.70 were examined nevertheless without existing shoring face. These studies involved only the behavior of shortening the reinforcement length on the stability of traditional MSE walls. Watanabe et al. (2003) [9] and Guler and Selek (2014) [3] studied the reduction of reinforcement length below 0.70H on the global stability of MSE walls and they concluded that the walls with shorter reinforcement provided less ductile behavior than walls with longer reinforcement.

Rabei et al. (2018) [8] performed a series 1/8 scale 1-g shaking table tests on NMSE walls in-front of existing rigid face *without any connections between the reinforcement and the rigid back face*. 11 small scale shaking table tests examined the effect of input motion characteristics, facing rigidity, wall aspect ratio on the global stability and acceleration responses of these walls. This long research concluded that NMSE walls without connection with the existing shoring face didn't offer any displacement until reaching the yield acceleration and then excessive deformations occurs. Moreover, the base yield acceleration was very sensitive to the ground motion predominant frequency and the wall configurations. The base yield accelerations increased twice when the predominant frequency dropped from 5 to 1 Hz. Also, when the wall aspect ratio increased from 0.20 to 0.40, the base yield accelerations increased from 0.2g to 0.33g. the walls of rigid facing can withstand base yield accelerations twice the flexible facing.

NMSE walls tested by Rabei et al. (2018) [3] concluded that the input base accelerations were amplified throughout the height of the model. A value of 1.10 and 1.35 amplification factors for the rigid facing walls with predominant frequency below 2.5 Hz and larger than 2.5 Hz are assumed, respectively. While for flexible facing walls, an amplification factor of 1.15 was proposed. The future recommendations of Rabei et al. (2018) [3] proposed an anchorage between the top reinforcement layers of NMSE walls with the existing shoring face by a fixed mechanical connection to enhance the behavior of NMSE walls after the base yield accelerations.

This paper offers one Full scale shaking table tests performed on NMSE walls using a fixed mechanical connection between the top reinforcement layers with the rigid box to enhance the behavior of Narrow walls.

2 MODEL LOADING TEST

2.1 Shaking table

A rigid square steel box of dimensions 1.10 m wide/length × 1.0 m height is used in simulating the tested model. The box is covered from inner with 8 mm-thick of transparent Plexiglas to allow the monitoring of the model through shaking. This box is bolted to the Cairo University shaking table by 20 mm high strength bolts. The table steel platform measures 1.50 m by 1.50 m with one horizontal degree of freedom and 2.0 tons maximum pay load capacity at frequencies up to 50 Hz and peak base acceleration amplitude up to ±1g.

2.2 Wall configuration and model construction

Figure 1 highlights the model wall configuration with the full instrumentations used to capture the behavior of the tested wall. A typical cross section and instrumentation plan is described also, in Figure 1. The reinforcement length, L, to the wall height, H, is set to 0.3. The reinforcement vertical spacing, Sv, was taken 0.10 m in order to isolate the effect of reinforcement density on the dynamic response of the model walls [2]. Full height rigid facing panel of wood is used. The facing panel was constructed using 3 panels of counter wood of total thickness 24 mm as given in Figure 1.b and interlocking each to gather to perform a full height panel.

A steel bracing system to support the model during construction is used. The sand backfilling as well as the reinforcement layers in 100 mm lifts were installed. The backfilling was placed in a loosest state and compacted to 95 % relative compaction using 0.05 kN steel hammer and falling from 150 mm Height. The reinforcement is connected to the wall facing without any connections from its end with the steel box. Finally, the external bracing system was removed at the end of construction. This condition corresponds to the starting point (static loading condition) prior to shaking. The methodology of the model construction is considered to be a technique that falls

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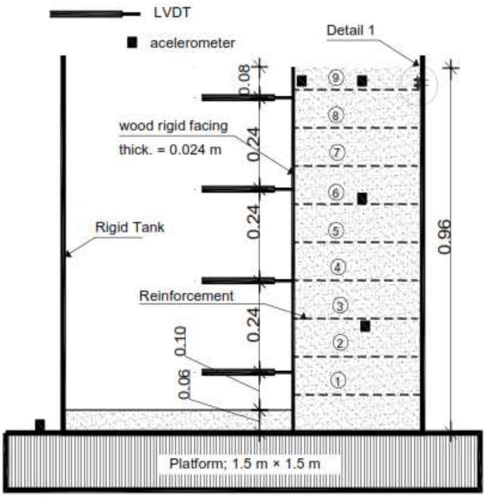
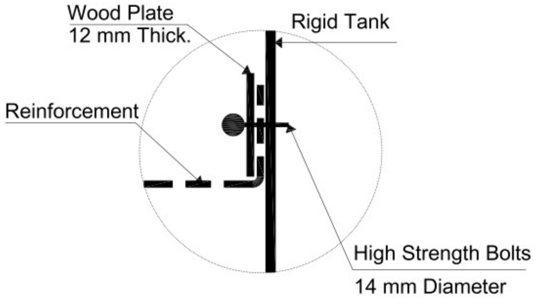
between the field case of the full height braced rigid panels and an incrementally unbraced modular block wall.



(a) Typical cross section

(b) Model View



Detail 1

Fig. 1. Instrumentation layout and details of reduced scale NMSE wall (all dimensions in m). (a) Typical cross section and (b) Model View

2.3 Material

2.3.1 *Soil*

The material used in the model obtained from Dahshour district, north of the Nile valley, Giza, Egypt. According to the unified soil classification system (USCS), the soil is a uniformly graded sand with about 1% fines and the soil characteristics are produced in Table 1. The backfilling is prepared at 82 % relative density and zero moisture content.

2.3.2 *Reinforcement*

Table 2 produces the characteristics of geosynthetic material used in the wall model. It is classified a commercially bi-axial knitted polyester (PET) geogrid and coated with polymer of green color. The reinforcement is selected because of the relatively low tensile strengths at 2 % strain to fit the small-scale behavior of the tested wall. According to the scaling law suggested by Iai (1989) [4], the relationship between prototype –scale reinforcement stiffness (J_P) and model scale stiffness (J_m) can be calculated as $J_P = J_m \lambda^2$, where $1/\lambda$ is the model scale and was taken 1/8 in this research. Hence, the stiffness of the reinforcement in the small-scale models (J_m = 105 kN/m at 2% strain) is equivalent relatively stiff to very stiff geosynthetic reinforcement products at prototype scale (J_p = 6720 kN/m at 2% strain). the product of PET geogrid is used in this research rather than any reinforcement because the axial load-extension of PET geogrid are essentially strain-rate dependent.

Table 1. Backfill material properties.

Raw material	silica sand
Specific Gravity, Gs	2.67
Max. Void Ratio, $e_{max.}$	0.77
Min. Void Ratio, $e_{min.}$	0.59
Coefficient of Curvature, C_c	2.74
Coefficient of Uniformity, C_u	0.91
Max. Dry Density	16.70 kN/m^3
Peak Friction Angle ϕ_{peak}	41^o

Table 2. Geogrid reinforcement properties (reduced-scale model).

Raw material	Polyester (PET)
Coating material	Polymer (color green)
Mass/unit area (gm/m^2)	130
Aperture size (mm)	
Machine and cross machine direction	3.5
Wide-width strip tensile strength (kN/m)	
At 2% strain (MD)	2
Ultimate (MD)	15

2.4 Instrumentation and base input motion

Wall facing deformations and the model acceleration response are the main objective of the tested model. Accordingly, lvdts are mounted at the facing elevation to measure the displacements during base shaking assuming that the datum for the recorded displacements is the shaking table platform. Each lvdt transducer is attached to a rigid vertical steel bracing system (see Fig. 1b) that is fixed with the rigid box by c-clamps. While the model wall acceleration response is measured using five accelerometers with a range from 1g to 2g and frequency response was ranged between 1 mv to 5 v. Three accelerometers were embedded in the middle of soil model in addition to one accelerometer embedded at the top of wall behind the facing. Moreover, one accelerometer attached to the table platform to measure the input base acceleration as shown in Figure 1a.

 https://doi.org/10.1051/e3sconf/202336802026

The horizontal base acceleration is a stepped-amplitude-sinusoidal function as shown in Figure 2 with a predominant frequency 2.5 Hz. The amplitude is increased in 0.05 g increments every 5 sec until excessive deformation occurred. According to Bathurst and Hatami (1998) [1], Matsu et al. (1998) [5] and Rabei et al. (2018) [8] this simple base excitation record is more aggressive than a typical earthquake record with the same predominant frequency and amplitude.

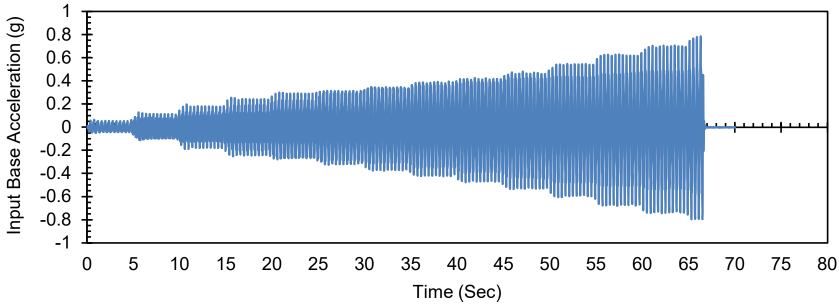


Fig. 2. Measured input base acceleration of frequency 2.50 HZ

3 MODEL TEST RESULTS

3.1 General

This section produces the model wall test results. Displacement behavior versus time and acceleration response through the wall height are displayed. Moreover, a model view for the surface failure and the anchorage at the end of shaking are, also, presented. Figure 3 shows the mentioned parameters as described.

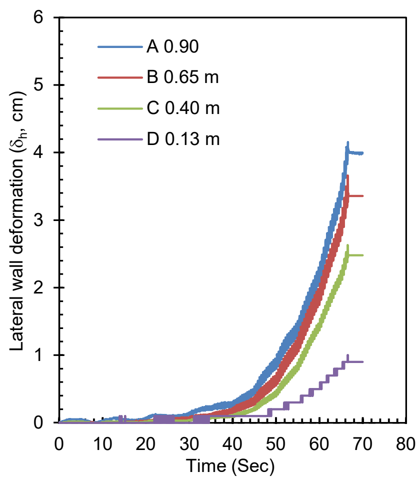
3.2 Facing displacement

The wall deformations for the full height panel rigid facing are measured using the four lvdts as displayed in Fig. 1. The progression of wall deformations against the increasing of input base accelerations are recorded in Fig. 3-c. the full height panel facing produces a rotational movement with a small toe movement. This confirms the same behavior as reported by El-Emam and Bathurst. (2004) [2] and Rabei et al. (2018) [8]. Fig. 4 shows the progression of top wall facing deformations against the input base accelerations. It is obvious from the results that the wall remains stable until 0.48g input base accelerations. Then, wall excessive deformations occurs when the peak base acceleration amplitude increased over these values.

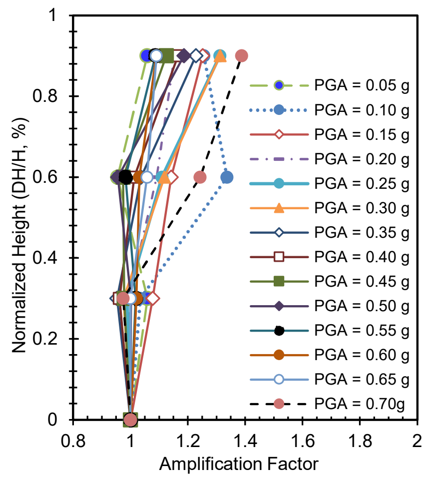
3.3 Model wall acceleration response

The input base acceleration and the model acceleration response were measured at the center of the wall at elevations 0.35, 0.6 and 0.9 m in addition to one accelerometer at elevations 0.90 m behind the facing. Fig. 5 shows the input base and wall acceleration response. The results show amplification factors up to 1.45 at the top of walls in outward directions and 1.90 in inward directions as shown in Fig. 6.

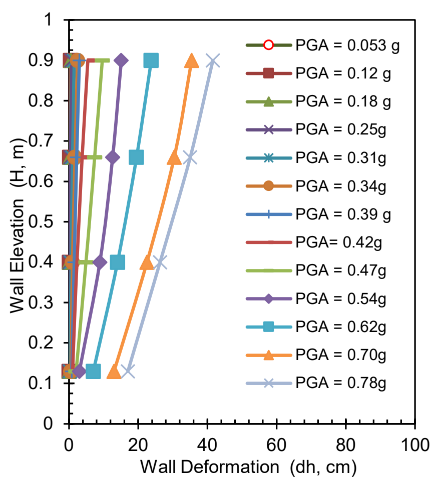
As shown in Fig. 5, the input base acceleration is correlated to the wall response accelerations producing an average amplification factors 1.43 and 1.16 inward and outward directions, respectively with average value 1.25. These results are similar to the behavior reported by El-Emam and Bathurst (2004) [2] and Rabei et al. (2018) [8] and confirm the decoupling phenomenon stated by Muir wood et al. (2002) [7]. This was attributed to the decoupling of backfill at strong ground motions that reduces its bearing capacity to transmit the shear stresses to higher elevations which would otherwise contribute to acceleration amplification up to the height of the backfill.



(a) typical displacement-time history



(b) Amplification factor profile



(c) wall rotation versus progression of input peak ground acceleration (PGA)

(d) Model view for the surface at the end of shaking

Fig.3. Model wall test results of L/H = 0.3 and frequency = 2.5 Hz. (a) typical displacement-time history, (b) Amplification factor profile, (c) wall rotation versus progression of input peak ground acceleration (PGA) and (d) Model view for the surface at the end of shaking

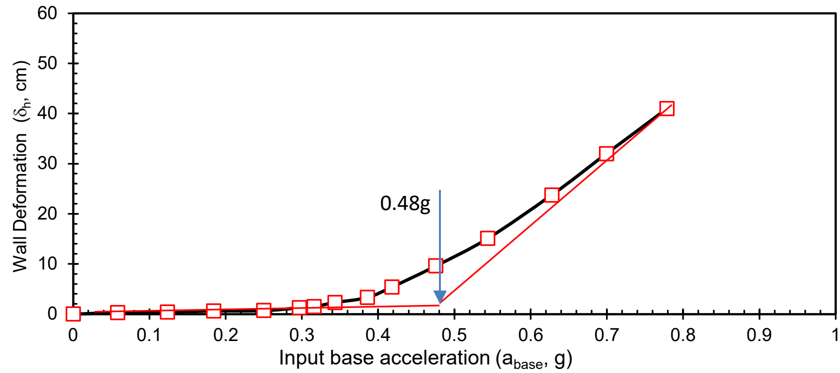


Fig 4. Top wall displacement versus base acceleration

4 WALL YIELD ACCELERATION

Fig.7 displays the wall deformation-input base acceleration relationship of NMSE walls with anchorage and NMSE walls without anchorage [8]. From this relation, the ultimate input base accelerations are obtained at the point where the slope of curve reach zero. Then the excessive deformations occur. At the point of the zero slope, the term of yield acceleration is defined and a value of 0.48g is measured. Rabei et al. (2018) [8] studied NMSE walls without any connections of reinforcement with the back rigid face and the tested wall produced a yield value of 0.38g. This means that the anchorage of top layer of reinforcement with the rigid back existing face increasing the point of yield acceleration about 26 %. Also, the anchorage reduced the displacement at failure one-half time the walls without anchorage.

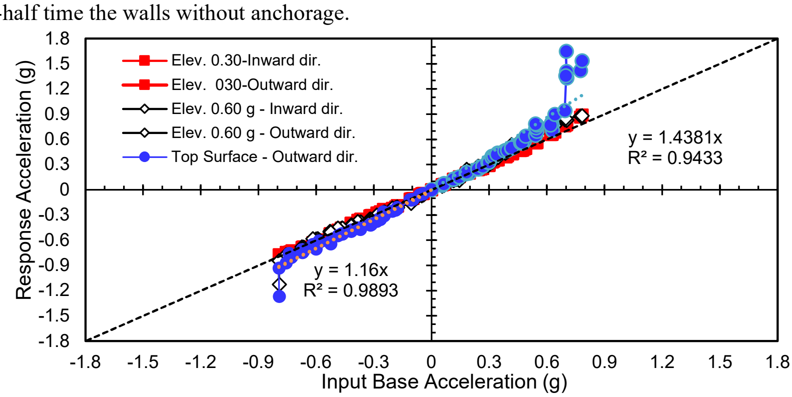
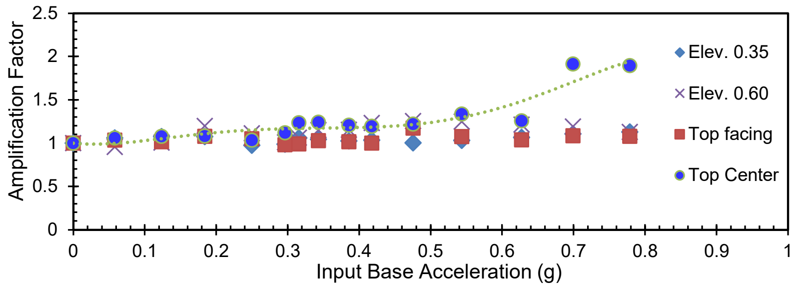
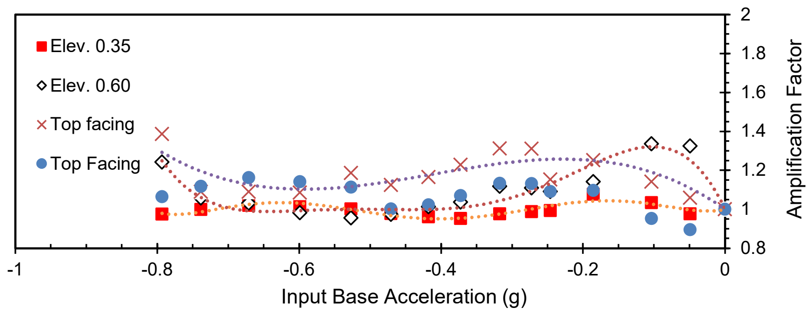


Fig. 5. Input base acceleration versus wall response acceleration



(a) Amplification factor in inward directions



(b) Amplification factor in outward directions

Fig. 6. Amplification factor in inward and outward directions

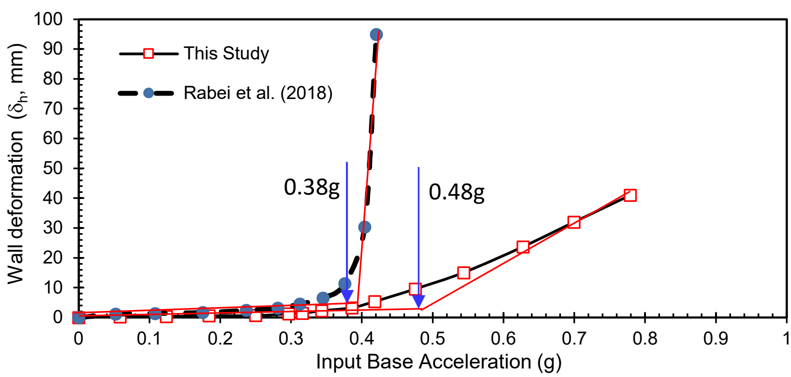


Fig. 7. Wall deformations versus input base accelerations for NMSE walls with and without anchorage

5 CONCLUSIONS

This paper presents the results of a unique shaking table test on NMSE wall with mechanical anchorage for the first top reinforcement layer with the existing rigid back face (the strong box in this study). The results indicate that a rotation mode with slightly toe movement is monitored. A yield acceleration value of 0.48g is measured. This value is considered the threshold value that the wall will fail if the wall subjected to peak ground acceleration exceeds this value. Comparing the tested wall with anchorage of top reinforcement layers

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with walls without anchorage, the displacement at failure reduced to one-half time. Correlating the input base peak acceleration to the average response acceleration within NMSE wall with anchorage, average design seismic coefficient 1.25 can be used in the seismic design of NMSE walls of similar configurations.

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