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Investigation of Seismic Response of Reinforced Soil Retaining Walls

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ABSTRACT

Dynamic response of a segmental (modular block) retaining wall system to recorded ground motions is investigated. The magnit and characteristics of wall response are compared to those obtained under harmonic input base acceleration. The calculated maxim lateral displacement and reinforcement load of the segmental retaining wall model subjected to a single frequency, harmonic ir acceleration were considerably larger than the corresponding values obtained using a number of earthquake accelerograms v comparable predominant frequencies. It is concluded that the random characteristic of actual ground acceleration may partly exp the relatively good performance of reinforced-soil retaining wall systems that were designed without seismic considerations or at l using simple pseudo-static limit equilibrium methods. Nevertheless, it was found that low-frequency ground motions with h intensity values can result in significant structural response magnitude of short-period retaining wall systems.

KEYWORDS

Retaining walls, Reinforced soil, Seismic response, Earthquake characteristics, FLAC

INTRODUCTION

Reinforced soil retaining walls are composite structures comprised of horizontal layers of geosynthetic or metallic reinforcement extending into a soil backfill and typically attached to a hard facing. The hard facing may comprise full-height reinforced concrete panels, articulated incremental concrete panels, gabions or modular block systems. Where geosynthetic polymeric reinforcement materials such as geogrids have been used, these systems have proven to be very cost effective particularly with respect to traditional gravity-type structures. Within the family of reinforced soil wall technologies, reinforced segmental (modular block) retaining walls constructed with a dry-stacked column of masonry concrete units to form the facing are the most economical. In addition, these walls offer ease of construction for the contractor and a wide range of aesthetic appearances for the architect.

Design and analysis methods, albeit conservative, are well established for reinforced soil walls under static loading conditions (FHWA 1996, NCMA 1996, PWRI 1992, AASHTO 1998). However, increasing numbers of reinforced soil retaining wall systems are being constructed in seismically active areas. Simultaneously, more significant earthquake ground motion events are being recorded around the world and national seismic hazard maps (e.g., NBCC 1995, Leyendecker 2000, Leyendecker et al. 2000) are be continuously modified to accommodate the most recent ma seismic records and to reflect updated seismic hazard lev Reinforced soil retaining walls have generally shown a g performance record under seismic loading when comparec conventional gravity retaining wall systems (Tatsuoka et 1995, Bathurst and Alfaro 1997). Nevertheless, seismic des methods are not well advanced for reinforced soil v structures. The most common approach for seismic design reinforced soil retaining walls is to use Mononabe-Ok theory and a selected peak ground acceleration to calculat modified lateral earth pressure coefficient (Bathurst a Alfaro 1997). Typically, the backfill dynamic incremental l (i.e., the load in addition to the static part) is calculated a empirically partitioned into soil reinforcement layer loads (AASHTO 1998, Bathurst 1998, PWRI 1992). The sh comings of pseudo-static methods are recognized in United States where these methods are limited to sites wh the peak ground acceleration is not expected to exceed 0.2 For greater accelerations, sliding block displacement meth have been proposed for reinforced soil walls (Cai and Bath 1996, Ling et al. 1997). However, displacement methods limited to internal and external sliding mechanisms and do address potential internal failure mechanisms such reinforcement over-stressing or pullout.

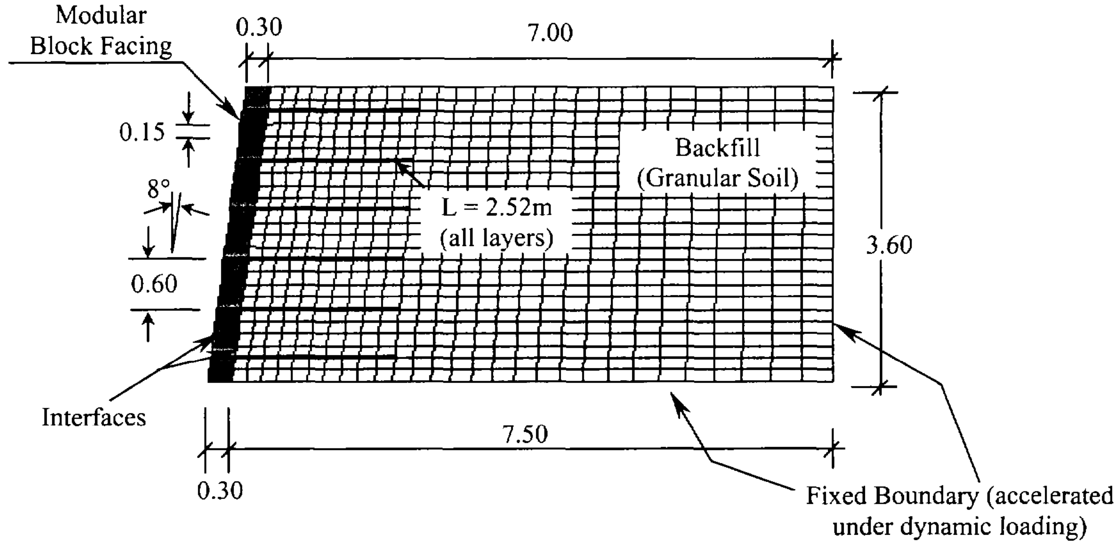

Fig. 1 Discretised model of segmental (modular block) retaining wall system at end of construction (dimensions are in metres).

Due to the shortcomings of the limit-equilibrium methods described above, numerical methods that allow the entire response of a structure to be simulated hold promise as a tool to develop new seismic design guidelines for reinforced soil retaining wall structures and to quantify levels of safety against collapse and serviceability under site-specific seismic loading.

As a first step in this direction, the writers investigated the influence of wall height, backfill width, reinforcement properties and toe restraint condition among other parameters on characteristic frequency and seismic response of idealized full-height propped-panel walls using a numerical approach (Bathurst and Hatami 1998, Hatami and Bathurst 2000). To simplify the dynamic loading, the wall models were subjected to a variable-amplitude, harmonic input ground motion. Numerical simulation results showed that retaining wall models developed significant lateral displacement and reinforcement load when subjected to a single-frequency base excitation in the vicinity of retaining wall fundamental frequency. The wall fundamental frequency was predicted using available closed-form solutions and interpretation of the response of the numerical wall model to a range of input frequencies.

This paper extends the earlier work by the writers by examining the response of a reinforced soil retaining walls to a number of recorded earthquake ground motions. The magnitude and characteristics of the retaining wall response to real accelerograms viz. single-frequency, harmonic input accelerations are also presented and discussed.

NUMERICAL MODEL

General Description and Geometry

The segmental (modular block) wall model depicted in Fig. 1 was used as the example structure in the current paper. The wall is 3.6m high and includes 24 concrete block courses which are stacked to produce an 8° batter angle from the vertical. The wall geometry represents a full-scale segmental retaining wall that was recently constructed at the Royal Military College and tested under surcharge loading (Bathurst et al. 2000). The wall consists of six geosynthetic reinforcement layers with a length of 2.52m from the front of the facing into the backfill. This reinforcement length provides the minimum reinforcement length to wall height ratio (i.e., L/H=0.7) recommended by current design codes for static stability (e.g., FHWA 1996, AASHTO 1998). The vertical spacing, S_v, between the reinforcement layers is constant and equal to 0.6m which is the maximum spacing permitted for the example wall structure according to AASHTO (1998) guidelines. The backfill width is extended to 7.25m (average value over the depth) behind the facing giving a width to height ratio, B/H=2 for the backfill model. A fixed boundary condition representing a rigid foundation is assumed at the bottom of the backfill.

Material Properties

The backfill soil is modelled as a purely frictional, elastic-plastic material with Mohr-Coulomb failure criterion. The backfill unit weight is assumed as $\gamma = 17$ kN/m^3. The bulk modulus and shear modulus values of the backfill material assumed during wall construction are calculated using the stress dependent, hyperbolic model proposed by Duncan et al. (1980). Thereafter, constant linear elastic-plastic properties were assumed in order to reduce computation time and to ensure numerical stability for all seismic simulation runs. The material properties assumed for the backfill soil are presented in Table 1. The reinforcement is modelled using linear elastic, perfectly plastic FLAC cable elements (see *Numerical Approach* below) with negligible compressive strength. The

Table 1. Backfill soil properties.

Stiffness Properties (Hyperbolic model)	Value
K_e (Elastic modulus number)	2000
K_b (Bulk modulus number)	2000
n (Elastic modulus exponent)	0.25
m (Bulk modulus exponent)	0.25
R_f (Failure ratio)	0.73
ν (Poisson's ratio)	0.15
Strength Properties	
ϕ (Peak friction angle)	46°
c (Cohesion)	0
ψ (Dilation angle)	6°

Notes: Soil stiffness parameters are all dimensionless.

reinforcement stiffness is assumed to be J=1000 kN/m which represents a typical stiff geogrid reinforcement.

Numerical Approach

The numerical simulations were carried out using the program FLAC (Itasca 1998). The retaining wall model was assumed in plane-strain condition and was constructed in layers. The backfill and wall facing were elevated in lifts of 0.15m and the reinforcement layers were placed in the model at designated elevations. The numerical model at each stage was solved to equilibrium with a prescribed tolerance before placing the next facing block, soil lift and reinforcement layer. After the wall model was constructed (Fig. 1), it was subjected to different horizontal input accelerations across the foundation. The excitation inputs were applied in terms of velocity histories with base line correction to ensure zero displacement at the base at the end of shaking. The base input velocities were simultaneously applied to the far-end boundary of the backfill model based on the assumption that the acceleration in the backfill depth was uniform at a sufficient distance from the wall facing.

Table 2. Fundamental frequency of model wall from numerical (FLAC) modelling and from closed-form solutions.

Method	f_1 (Hz)
Matsuo and Ohara (1960) (Neglecting vertical dynamic pressure in the backfill - σ_v =0)	4.65
Wu and Finn (1996)	4.71
Free vibration response (FLAC)	**5.2**
Scott (1973)	6.44
Matsuo and Ohara (1960) (Neglecting vertical vibration amplitude in the backfill - v=0)	6.92
Wood (1973)	7.5
Richardson (1978)	10.58

Note: The methods are described by Hatami and Bathurst (2000).

Table 3. Range and mean values of classified ground motion records (Naumoski et al. 1993).

Ensemble	VH	NH	NI	NL	NVL
A/V Range	2.63 to 3.52	1.60 to 2.43	0.82 to 1.21	0.62 to 0.79	0.36 to 0.59
$(A/V)_{mean}$	3.03	1.98	1.02	0.70	0.48

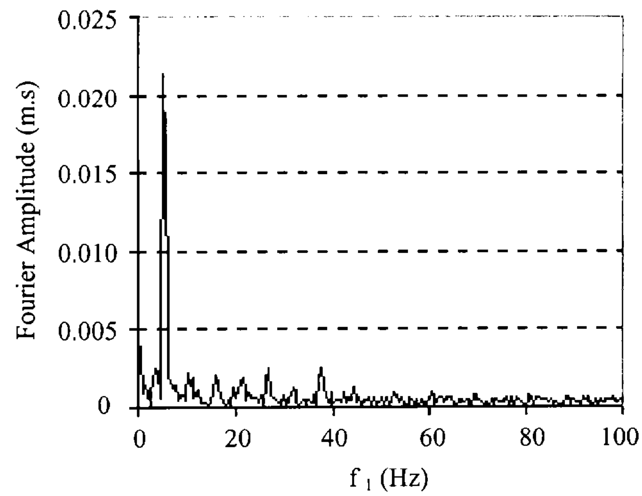

Fig. 2 Fundamental frequency of modular block retaining wall model from displacement response to sinusoidal impulse.

SEISMIC LOADING

Wall Fundamental Frequency

The fundamental frequency of the model segmental wall was evaluated before it was subjected to the actual recorded ground motions. One full sinusoidal impulse with the period T=0.1s was applied at the base and far-end boundary of the model. The fundamental frequency of the wall from free vibration response of facing lateral displacement was determined to be f_l=5.2 Hz (Fig. 2). From the closed-form solutions for fundamental frequency of rigid retaining walls shown in Table 2, the formula proposed by Wu and Finn (1996) gave a value closest to the numerical predicted value. It can also be noted that the observed wall fundamental frequency value falls between limiting values from solutions by Matsuo and Ohara in Table 2 (see also Hatami and Bathurst 2000).

Earthquake Records

A set of 6 recorded ground motions were selected as input accelerograms to the retaining wall model. These records were chosen from a database that is classified according to the accelerogram A/V ratio values (Naumoski et al. 1988, 1993) where A is the peak ground acceleration in g (i.e., acceleration of gravity) and V is the peak ground velocity of the recorded ground motion in m/s. Results of a number of studies indicate that the A/V ratio of a recorded ground motion correlates with its frequency content (Seed et al. 1976, McGuire 1978, Sawada et al. 1992). The database of records by Naumoski et al. (1993) contains a total of 75 recorded ground motions from around the world that are categorised into five different groups with 15 records in each group according to their A/V ratio

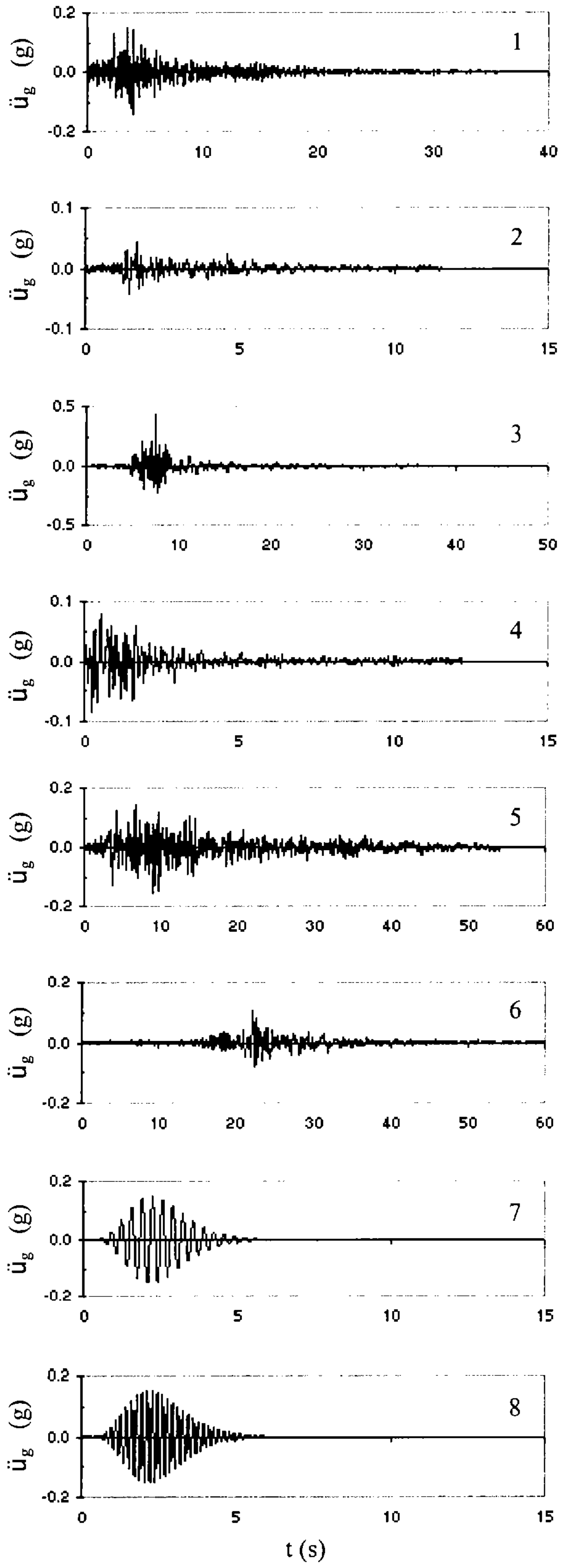

Fig. 3 Input acceleration histories applied to the modular block wall model (number in each figure refers to ground motion record in Table 4).

values (Table 3). The notations VH, NH, NI, NL and NVL in the table stand for very high, new high, new intermediate, new low and new very low categories in terms of the A/V ratio values of the records, respectively. The new groups are the refined versions of a previous classification of a smaller database comprising of only 3 different ensembles (Naumoski et al. 1988). The entire ensemble of records in the new set includes seismic events with magnitudes between M=5 and M=8 and epicentral distances between 3 and 500 km. Records with larger A/V ratio values are statistically associated with moderately strong to strong earthquakes at short epicentral distances. Records with lower A/V ratio values typically correspond to large earthquakes at large epicentral distances (Naumoski et al. 1993). Accordingly, records with higher A/V ratio values typically have higher predominant frequencies. The perfect positive correlation between the A/V ratio value and the predominant frequency of an acceleration record is evident for the case of a simple harmonic motion (e.g. records 7 and 8 in Table 4). Since an accelerogram can be generally considered as a summation of harmonic components, a correlation between A/V ratio and predominant frequency of a general acceleration record can be expected.

The ground motions chosen for this study are shown in Table 4 and plotted in Fig. 3. Ground motion records from the VH, NH and NI categories were chosen for seismic input to the model retaining wall. These categories were selected because they represent ground motion records with higher predominant frequencies that appear to be more aggressive to the short-period retaining wall model under study. Each of the records in Table 4 has a relatively narrow and well-defined peak in its spectral acceleration curves as given by Naumoski et al. (1993). The data in Table 4 includes three main characteristics of the ground motion records namely, duration, intensity and frequency content. In the *duration* section, the parameters Δt, t_T and t_d represent the time interval between recorded data points, the total recorded time of the accelerogram and the calculated duration of strong ground motion of the record, respectively. The duration of strong ground motion was calculated according to the method proposed by Trifunac and Brady (1975). One of the advantages of this method (e.g., over the bracketed definition proposed by Bolt 1969) is that the calculated duration is not affected by scaling of the records according to their peak acceleration amplitude. In the *intensity* section of the table, V_n is the peak ground velocity of the record after scaling to a reference peak ground acceleration A=0.15g. All the input ground motion records were scaled to a common peak ground acceleration magnitude to isolate the influence of other ground motion characteristics on wall response. Parameter I_A in Table 4 denotes the intensity of each ground motion record according to the following equation proposed by Arias (1969):

$$I_A(\xi) = \frac{\cos^{-1}(\xi)}{g\sqrt{1-\xi^2}} \int_0^{t_T} \ddot{u}_g^{\,2}(t)dt \qquad (1)$$

The parameter $I_A^{1/2}$ is a modified intensity measure equal to $(I_A)^{1/2}$. This intensity measure is introduced as an additional parameter because, in contrast to the Arias intensity, it is linearly proportional to the ground acceleration amplitude.

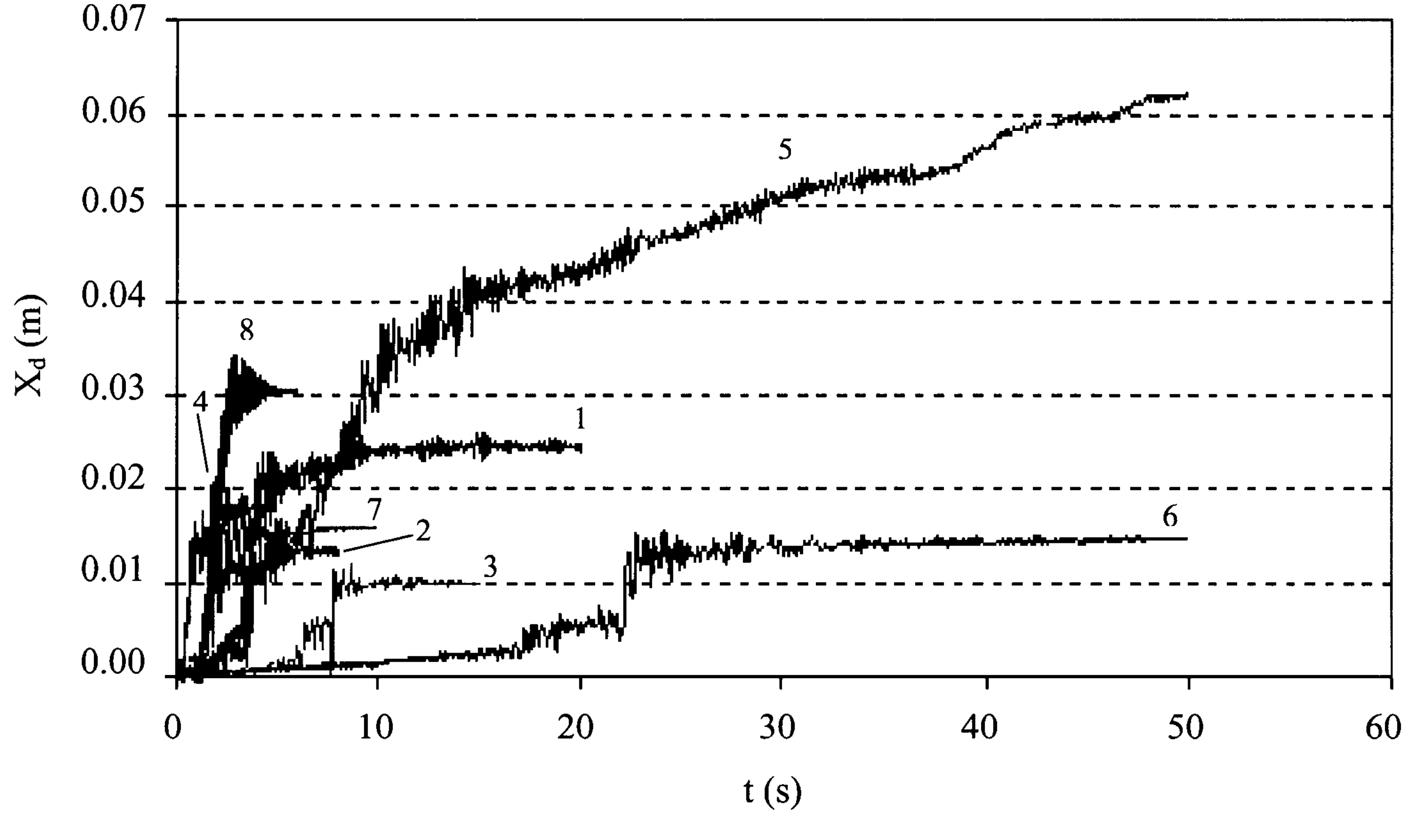

Fig. 4 History of facing lateral displacement at the top of wall subjected to input ground motions in Table 4 (all scaled to A=0.15g).

The intensity parameters I_A and $I_A^{1/2}$ are calculated using a viscous damping ratio value of ξ =10%. The parameter f_g in the table is the predominant frequency of each record which is determined from its Fourier transform. Two sinusoidal functions (records 7 and 8 in Table 4) were also used as input motions to compare the wall response to harmonic viz. recorded ground accelerations (Fig. 3). The sinusoidal input motions are defined by:

$$\ddot{u}_g(t) = \sqrt{\beta e^{-\alpha t} t^{\zeta}} \sin(2\pi f_g t) \qquad (2)$$

Where α = 5.5, β = 31 and ζ = 12 are constants, t is time and f_g is the frequency of base acceleration. The resulting peak acceleration amplitude, A, using the selected values for constant parameters is 0.15g.

RESPONSE OF RETAINING WALL TO GROUND MOTIONS

The response of the model modular block (segmental) retaining wall system to input ground motion is examined in terms of facing lateral displacement and reinforcement load.

Table 4. Ground motion records used in the current study.

Record			**No. of data points**	**Duration**			**Intensity** (1)					**Frequency**		**Foundation Condition**
No.	Naumoski et al. (1988)	Naumoski et al. (1993)		**Δt** sec	**t_T** sec	**t_d** sec	**$A^{(2)}$** g	**$V^{(2)}$** m/s	**$V_n^{(1)}$** m/s	**$I_A^{(1)}$** m/s	**$I_A^{1/2\ (1)}$** $(m/s)^{1/2}$	**f_g** Hz	**A/V** g.s/m	
1	H9	-	1851	0.02	37.0	12.60	0.146	0.085	0.087	0.190	0.436	5.0	1.72	Rock
2	H14	VH15	578	0.02	11.6	5.44	0.042	0.016	0.057	0.111	0.333	6.4	2.63	Rock
3	H2	NH2	2202	0.02	44.0	6.74	0.434	0.255	0.088	0.096	0.309	2.8	1.70	Rock
4	H7	NH7	612	0.02	12.2	3.52	0.084	0.044	0.079	0.138	0.372	5.2	1.91	Rock
5	I3	NI3	2719	0.02	54.4	30.54	0.156	0.157	0.151	0.474	0.689	1.5	0.99	Rock
6	I14	NI13	6001	0.01	60.0	14.80	0.105	0.116	0.166	0.281	0.530	1.4	0.91	Rock
7	S1		400	0.025	10.0	2.15	0.150	0.080	0.080	0.265	0.515	3.0	1.88	-
8	S2		480	0.0125	6.0	2.15	0.150	0.040	0.040	0.265	0.515	6.0	3.75	-

Notes: (1) Peak ground velocity V_n and Arias intensity I_A (including the modified form $I_A^{1/2}$) are reported for scaled records. (2) The reported A and V values correspond to the records before being scaled to A=0.15g.

Figure 4 shows the histories of lateral displacement at the top of the facing column when the wall was subjected to the scaled input base accelerations in Table 4. The parameter X_d in Fig. 4 denotes the wall lateral displacement due to dynamic loading of ground motion in excess of the amount of lateral displacement at the end of construction. The effects of ground motion characteristics on wall response are discussed separately in the following sections.

Wall Lateral Displacement

Effect of Predominant Frequency. The effect of ground motion fundamental frequency on wall response can be readily examined by comparing the displacement histories of records 7 and 8 in Fig. 4. The records have identical Arias intensity and strong motion duration (Table 4). The lower frequency record 7 has a larger peak ground velocity. However, the results of Fig. 4 clearly show that record 8 with a predominant frequency closer to the fundamental frequency of the wall (f_g=6 Hz vs. f_l=5.2 Hz) induces a significantly larger lateral displacement in the wall (0.035m vs. 0.019m).

Earthquake records 2 and 3 have comparative characteristics of predominant frequency, peak ground velocity (V_n), scaled intensity and strong ground motion duration that are closest to the corresponding values for harmonic records 8 and 7, respectively. However, maximum wall displacement due to record 2 (0.015m) is not much greater than the maximum displacement value resulting from earthquake record 3 (0.012m). Quantitatively, the difference in wall response to records 3 and 2 is proportionally smaller than the difference in wall response to records 7 and 8. It follows that in the case of actual recorded ground motions -as opposed to harmonic input accelerations- the predominant frequency is not the sole dominant parameter that determines the magnitude of wall structural response.

The effect of ground motion predominant frequency on wall response can be examined by comparing displacement results for records 3 and 4. The records have comparable scaled peak velocity values. Since the records are scaled to the same peak ground acceleration, the A/V ratios of the records are also close in magnitude. The magnitude of maximum wall response to record 4 (0.021m) is considerably greater than the response to record 3 (0.012m) even though the intensity (i.e., $I_A^{1/2}$) of record 4 is only about 20% larger than the intensity of record 3. The strong motion duration of record 4 is also smaller than that of record 3. However, this does not appear to be a dominating factor for the magnitude of maximum wall displacement because neither of the two displacement histories shows a gradual growth of displacement response with time.

Harmonic Motion vs. Recorded Earthquake Accelerogram. Comparison of wall displacement response to harmonic inputs 7 and 8 against records 1 and 4 shows that the single frequency harmonic records induced relatively large wall response although their frequencies are not as close to the wall fundamental frequency as the predominant frequencies of records 1 and 4. This observation is in agreement with the results of Bathurst and Hatami (1998) who compared the dynamic response of a 6m-high propped-panel model wall to harmonic and 1940 El Centro earthquake acceleration records. It may be concluded that single-frequency input accelerations are typically more aggressive to the structure than actual earthquake records with identical predominant frequency and peak ground acceleration. This conclusion is also supported by comparing the displacement response of the wall to records 8 and 2 which have practically equal predominant frequencies. The magnitude of $I_A^{1/2}$ for record 8 is about 55% greater than the $I_A^{1/2}$ value of record 2. However, the magnitude of maximum wall displacement response to harmonic input acceleration 8 (0.035m) is substantially larger than the maximum wall response to accelerogram 2 (0.015m).

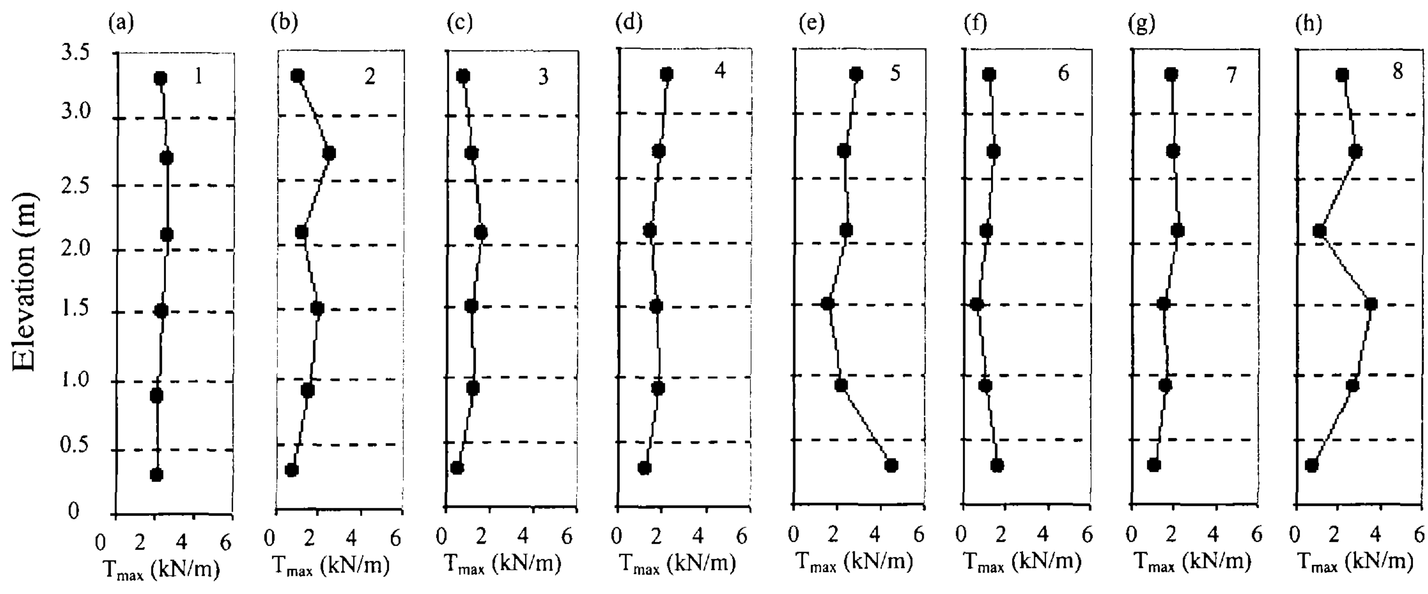

Fig. 5 Distribution of maximum reinforcement incremental load over the height of wall subjected to different input ground motions (number in each figure refers to ground motion record in Table 4).

Effect of Intensity and Strong Ground Motion Duration. The effect of ground motion intensity and strong motion duration can be examined by comparing the wall response to records 5 and 6. The two records have comparable scaled peak velocity and A/V ratio values. The predominant frequencies of the two records are almost equal and considerably lower than the fundamental frequency of the model wall. The $I_A^{1/2}$ value of record 5 is about 30% larger than that of record 6. However, the strong ground motion duration of record 5 is significantly higher than the duration of record 6. Figure 4 shows that the wall displacement response to record 5 is substantially larger than its response to record 6. The wall displacement response to record 5 is the only response history in Fig. 4 that shows a clear influence of the ground motion duration on the magnitude of wall response. Comparison of wall response to other ground motion records in Fig. 4 does not show a clear dependence of the wall response magnitude on either the ground motion duration (e.g., cf. records 1 and 4) or intensity (e.g., cf. records 2 and 6). The effects of ground motion intensity and duration on the magnitude of wall response are difficult to isolate for records with a predominant frequency close to the fundamental frequency of the wall. For example, comparison of wall displacement response to records 1 and 4 shows that the magnitudes of maximum wall displacement for walls subjected to acceleration records with predominant frequencies in the vicinity of the wall natural frequency are not significantly different (0.021m vs. 0.026m) in spite of their different intensity and strong motion duration values. It may be concluded that a short-period retaining wall structure subjected to a low-frequency ground acceleration can still develop a large response as a result of the combined effect of large intensity and duration of the ground motion.

Records classified as having intermediate A/V ratio values (i.e., records 5 and 6) typically have larger strong motion duration values than records in the high A/V ratio category (Table 4). Therefore, reinforced-soil retaining walls that may be subjected to ground motions from large distant earthquakes (e.g., in the range of a few hundred kilometres from a major fault) can be susceptible to failure or excessive deformation under seismic loading.

Reinforcement Load

Figure 5 shows plots of reinforcement incremental load (i.e., the axial force in the reinforcement due to dynamic loading only) in the retaining wall subjected to the scaled ground motions listed in Table 4. The relative performance of the wall model based on displacement response for the 8 base excitation cases applies equally to the relative performance of the wall based on maximum reinforcement load. In addition, inspection of maximum reinforcement loads in Fig. 5 shows that for ground motion records with $f_g \le f_1$ (i.e., all records except 8 and 2), the distribution of reinforcement incremental load over the wall height can be considered to be essentially uniform. The lowermost reinforcement layer may develop a significant load under a severe dynamic loading (Fig. 5e). A similar observation was made in a previous study on propped-panel type retaining wall models (Bathurst and Hatami 1998). For the cases with $f_g > f_1$ in Fig. 5 (i.e., input records 8 and 2), the distribution of maximum reinforcement incremental load shows comparatively low values at the toe and at an elevation of about 0.6H above the toe. This distribution pattern is consistent with the second vibration mode shape of a cantilever shear beam model. The reinforcement incremental load otherwise shows an overall parabolic distribution shape with the largest magnitude of reinforcement load at about mid-height of the wall.

CONCLUSIONS

Seismic response of a segmental (modular block) retaining model to recorded and harmonic ground motions is studied using a numerical modelling approach. The wall response is presented in terms of lateral displacement histories of the wall facing and maximum values of reinforcement incremental load. The displacement responses of the segmental retaining wall model subjected to single frequency, harmonic input accelerations were considerably larger than the response magnitude to earthquake records with comparable predominant frequencies. This result confirms a statement made by the writers in an earlier parametric study (Bathurst and Hatami 1998) that the use of simple harmonic functions to simulate the seismic behavior of reinforced soil walls may be useful to establish the relative performance of different retaining wall systems, although the magnitude of response is likely to be excessive. The predominant frequency of scaled earthquake ground motion records, in general, showed a dominant effect on the magnitude of wall response to seismic loading. In addition, it was found that low-frequency ground motions with high intensity and strong motion duration values can result in significant structural response magnitude of short-period retaining wall systems. For the combination of low intensity earthquake ground motion with predominant frequency below the fundamental frequency of the structure, the maximum incremental reinforcement loads were uniformly distributed over the wall height. This result is considered a useful observation for the future refinement of empirical-based seismic design methods for reinforced soil retaining walls.

The numerical model in the current study was selected to simulate a segmental retaining wall structure. Nonetheless, the conclusions from this preliminary study are believed to be applicable to other types of hard-faced concrete reinforced soil wall structures where the facing can add considerable stiffness and toe restraint to the composite gravity mass. Accordingly, it is concluded that the random characteristic of actual ground acceleration may explain the documented good performance of different reinforced-soil retaining wall systems during recent earthquakes (Tatsuoka et al. 1995, Bathurst and Alfaro 1997).

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