Dynamic Analysis of a Reinforced Earth Wall

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ABSTRACT The Port Mann/Highway 1 project in the greater Vancouver, BC is currently underway and includes construction of a new Port Mann Bridge over the Fraser River, widening Highway 1 for a distance of about 37 km and several mechanically stabilized earth (MSE) walls. The required seismic performance criteria for MSE walls were immediate functionality, repairable damage and no collapse after project design earthquakes with return periods of 475 yr, 975 yr and 2475 yr, respectively. A series of two dimensional numerical dynamic analyses using the program FLAC were performed to assess the internal stability of a 12m high Reinforced Earth retaining wall on firm ground foundation. The wall consisted of 16 layers of 9m long galvanized steel reinforcing strips at vertical spacing of 0.75m. The number of strips per metre length of the wall increased with depth. Numerical analyses indicated relative displacements within the reinforced earth structure in the range of less than 0.1m, 0.2m and 0.5m under 475 yr, 975 yr and 2475 yr design earthquakes, respectively. A number of strip layers yielded structurally, however the maximum calculated axial strains were less than 0.6%, 1.4% and 4.4% under 475 yr, 975 yr and 2475 yr design earthquakes, respectively and well below the allowable rupture strain.

Introduction

Reinforced Earth® structure consists of compacted granular soil, reinforced by steel strips which are connected to a segmental precast concrete facing. This composite structure forms a coherent mechanically stabilized earth (MSE) that is internally stable and can resist external loads.

The current Port Mann/Highway 1 project (PMH1) in Greater Vancouver, BC includes a new bridge over the Fraser River, 37 km of highway widening and MSE walls using a Reinforced Earth system. Throughout this paper the general term of MSE wall will be used.

Dynamic analyses were carried out to demonstrate that the proposed MSE walls were internally stable during seismic shaking and compliant with the seismic performance criteria of the project.

Numerical analyses were performed for improved soft ground and firm ground foundation conditions with three suites of earthquake time histories with return periods of 475, 975, and 2475 years. This paper presents the methodology and results of a series of analyses for a 12 m high MSE wall resting on firm ground foundation.

Seismicity

The PMH1 project is located in south-western British Columbia which is an area with active seismicity. The major source of this seismic activity is the oceanic Juan de Fuca plate subducting under and compressing the continental North American plate. This results in three potential earthquake sources: near-surface (0 to 30 km) crustal earthquakes, deep (40 to 50 km) intra-plate earthquakes within the subducting plate, and large inter-plate subduction earthquakes. The first two sources are accounted for in a probabilistic seismic model and are the predominant hazard for the site. The subduction earthquake is typically of larger magnitude but is at a significant distance (≈120 km) from the site and therefore, generally does not control the design.

Outcropping firm-ground response spectra for the 475-, 975- and 2475-year return period events, and for the deterministic subduction earthquake event are shown in Fig. 1. Sets of out-cropping firm-ground earthquake records in two orthogonal directions were fitted to the design response spectra by others and provided for use in the design (Table1). Fig. 2-Top shows one of the design outcropping firm ground motions.

Design performance criteria

PMH1 project requires the MSE walls to be designed according to the following seismic performance criteria:

- 475 yr: Limited traffic access immediately after earthquake, restorable within days.
- 975 yr: Limited access to emergency traffic within days after earthquake and repairable for full function.
- 2475 yr: No collapse or loss of life.

Dynamic numerical analyses were performed to get wall displacements and demands on the reinforcing strips for the purpose of assisting with assessment of compliance with the design criteria.

Fig. 1. Firm ground (Class 'C' soil) outcropping design response spectra.

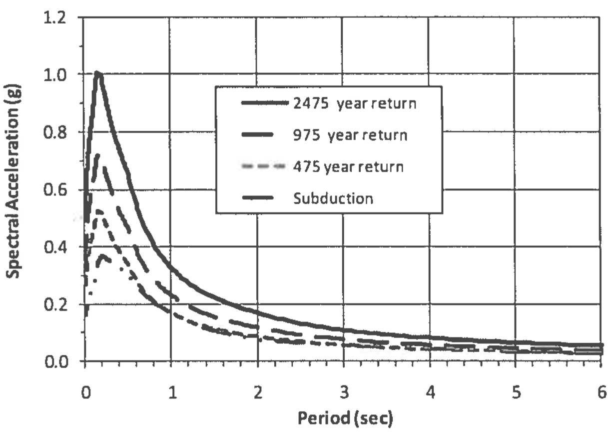
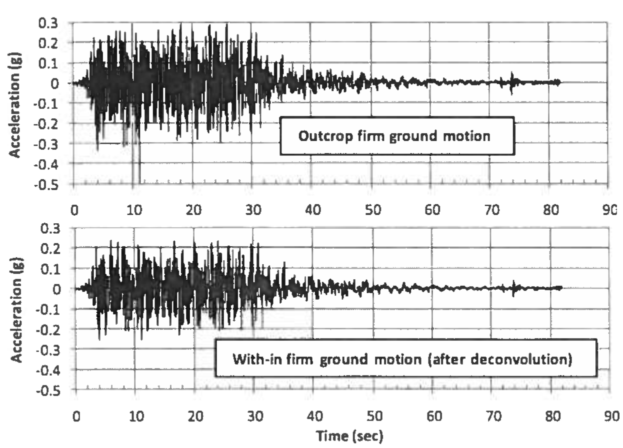


Table 1. Summary of PMH1 design ground motions

Earthquake Level	PGA (g)	Ground Motions Name, Year, Magnitude, Duration, Distance
475 yr	0.263	San Fernando,1971 M6.6, 7s, 36.0 km Loma Prieta, 1989, M7.0, 20s, 9.7 km Olympia, 1949, M7.1, ~21s, 26.0 km
975 yr	0.355	Landers, 1992, M7.3, 31s, 13.7 km Loma Prieta, 1989, M7.0, ~20s, 9.7 km Chi Chi, 1999, M7.6, 32s, 7.1 km
2475 yr	0.494	Landers, 1992, M7.3, 31s, 13.7 km Loma Prieta, 1989, M7.0, ~20s, 9.7 km Chi Chi, 1999, M7.6, 32s, 7.1 km
Subduction	0.16	Mexico City, 1985, M8.1, 32s, 107 km

Fig. 2. One of design firm ground motion time histories, CHICHI-NS-2475. (Top) Outcropping firm ground motion, (Bottom) With-in firm ground motion



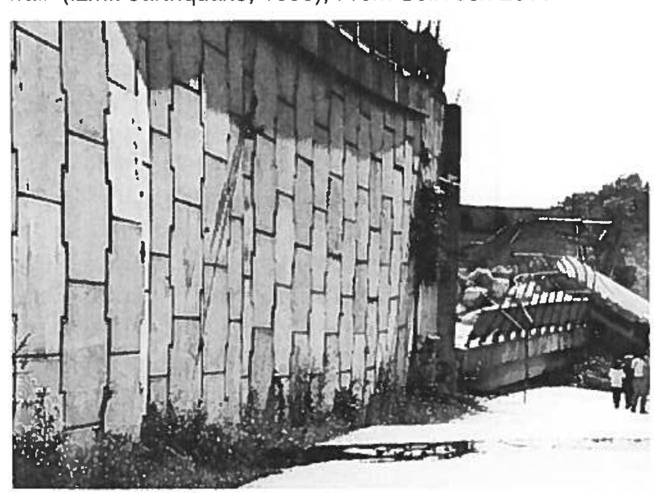
Performance of Reinforced Earth structures in the past earthquakes

Over the past forty years the MSE wall system under study has survived major earthquakes with no failures and little or no damage. These walls have undergone post earthquake inspections with the results duly recorded. Table 2 presents a summary of the survey of Reinforced Earth structures after the Northridge, 1994 and Kobe, 1995 earthquakes. It is interesting to note that all the walls remained functional after the earthquakes even though measured or estimated ground accelerations exceeded design values. Fig.3 shows an MSE wall with minor damage adjacent to a collapsed bridge after the 1999 Izmit, Turkey earthquake.

Table 2. Summary of survey results on the Reinforced Earth structures after Northridge and Kobe earthquakes (adapted from Sankey and Segrestin, 2001)

Earthquake	Northridge 1994	Kobe 1995
Total walls surveyed	23	120
Wall Height> 5m	65%	70%
Wall Height> 10m	25%	15%
Actual PGA Measured/estimated	0.07-0.91g	0.27g
Design PGA	75% of cases less than actual PGA 50% of cases no seismic design	0.15-0.2g
Conditions after earthquake	All functional Minor spalling	All functional Minor cracking Some movements

Fig. 3. Arifiye Reinforced, earth bridge approach MSE wall (Izmit earthquake, 1999), From SoilTech 2000



Dynamic numerical modelling

Two dimensional non-linear dynamic numerical analyses were carried out using the finite difference program FLAC version 6.0 (ITASCA, 2008).

Soil constitutive model

Mohr-Coulomb and UBCHYST constitutive models were used for the static and dynamic phases of analyses, respectively.

UBCHYST is developed at the University of British Columbia. It is a hyperbolic constitutive model with a Mohr Coulomb failure envelope and is used to simulate the non-linear hysteretic behaviour of soils during cyclic loading. The shear modulus is a function of stress ratio as presented in Equation [1] and Fig. 4.

$$[1] \qquad G_t = G_{\max} \cdot \left(1 - \frac{\eta_1}{\eta_f} \cdot R_f\right)^n$$

Where

G_t= tangent shear modulus

η = developed stress ratio = (τ_{xy} / σ'_v)

$\eta_1 = \eta - \eta_{max}$ = change in stress ratio η since last reversal

η_{max} = maximum η at last reversal

$\eta_{1f} = \eta_f - \eta_{max}$ = change in η to reach failure envelope in direction of loading

$\eta_f = (\sin(\varnothing_f) + \text{Cohesion} * \cos(\varnothing_f) / \sigma'_v)$

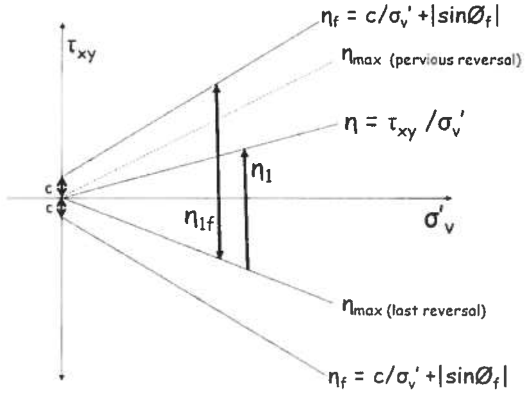
τ_{xy} = developed shear stress in horizontal plane

σ'_v = vertical effective stress

$\varnothing_f$ =peak friction angle

R_f and n = calibration parameters

Fig. 4. Failure envelope and parameters used in UBCHYST model



UBCHYST was calibrated to reasonably match Seed and Idriss (1970) G/G_{max} and Damping curves.

Model Geometry

The FLAC model was 12.75m high by 100m wide and consisted of 0.375m high by 0.41m wide elements in the proximity of wall. Elements became gradually wider towards the vertical boundaries of the model (Fig. 5).

The model was constructed in 8 lifts, each including two rows of 9m long reinforcing steel strips and one concrete facing segment. The density of strips varied with depth (Fig. 5). FLAC built-in "strip" and "beam" elements were used to model reinforcing strips and facing segments, respectively. One end of strips was connected to their respective facing beams. Both ends of facing beams were pinned. The bottom of the facing wall was connected to its respective grid point. The MSE fill and facing segments were separated using interface elements with a friction angle of 23 degrees.

Soil conditions and material parameters

The case presented here includes granular fill for MSE volume and backfill overlying firm ground. Firm ground is defined as very dense soil with shear wave velocities between 360 and 760 m/s. Table 3 presents the assumed soil parameters. Tables 4 and 5 present the assumed properties of the reinforcing strips and facing segments.

General procedure for numerical analysis

In FLAC, the dynamic analyses were carried out in the total stress mode in a chronological manner similar to the real conditions. The general procedure used for analyses included the following steps:

- Set up model grid and strip elements in 1.5m high lifts and bring to static equilibrium using Mohr-Coulomb models.
- Switch to UBCHYST constitutive model.
- Turn on dynamic configuration with large strain, and nominal 1% Rayleigh damping and bring to equilibrium by running with input motion of zero at bottom of the model and FLAC free field option at the side boundaries of the model.
- Set displacements to zero, apply the input time history at base of FLAC model, and solve past end of earthquake shaking. Input motions are the with-in firm ground motions obtained by deconvolution of the design outcropping motions. Program SHAKE2000 was used for deconvolution. Fig. 2-Bottom shows an example of input ground motion time history.

Fig. 5. Geometry of the FLAC model

No. of Strip Row from Bott. #	Distance from wall base (m)	No. Of Strips per m
16	11.625	1.33
15	10.875	1.33
14	10.125	1.33
13	9.375	1.33
12	8.625	1.33
11	7.875	1.33
10	7.125	1.33
9	6.375	1.33
8	5.625	1.33
7	4.875	1.33
6	4.125	1.33
5	3.375	1.67
4	2.625	1.67
3	1.875	1.67
2	1.125	2.00
1	0.375	2.00

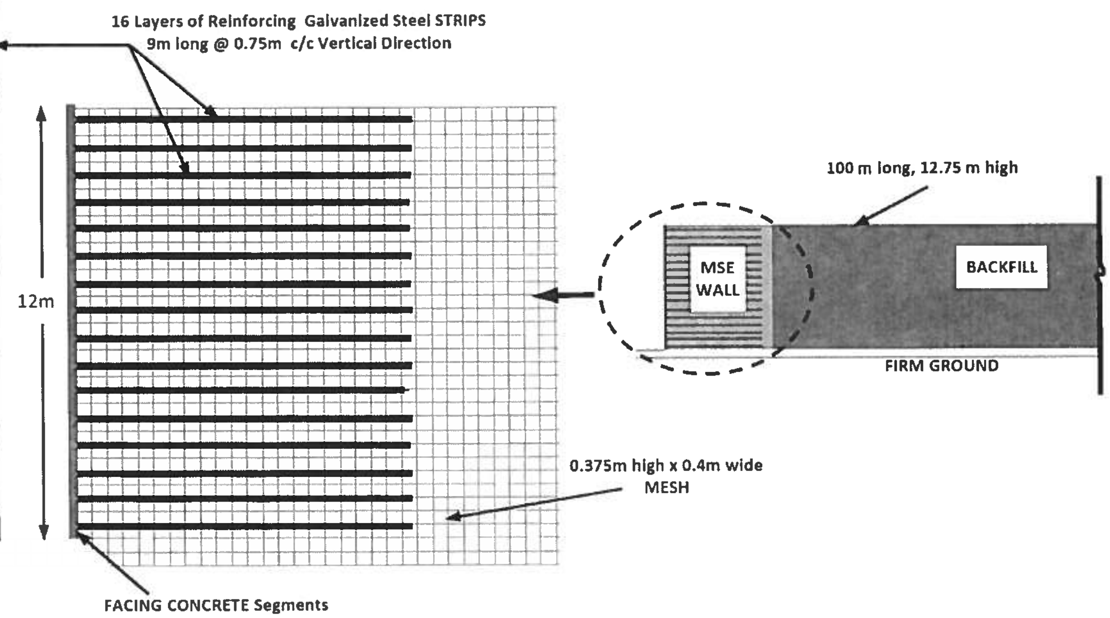


Table 3. Assumed soil properties

Soil Parameters	MSE Fill	Back fill	Firm ground
Unit weight (kN/m3)	20	21	22
Peak Friction angle (deg)	34	36	N/A
Dilation angle (deg)	4	0	N/A
Cohesion	0	0	N/A
Poisson's ratio (-)	0.3	0.3	0.3
Shear modulus in static phase, G (MPa)	22.6	22.6	180
Shear wave velocity, Vs	Note 1	Note 1	400
Constitutive model in dynamic phase	UBCHYST	UBCHYST	ELASTIC
UBCHYST parameters, Rf, n	0.8, 2.5	0.8, 2.5	N/A

Note 1: Shear wave velocity was estimated according to Chillarige et al. (1997) correlation as follows:

$$V_s = (A - B \cdot e) \cdot \left(\frac{\sigma_v'}{P_a} \right)^n \cdot (K_o)^{0.125}$$

Where A=295, B=143 and n=0.26 for the Fraser River Sand.
e is the void ratio assumed 0.68 equivalent to about 80% relative density.

Table 4. Assumed properties of reinforcing strips

Modulus of Elasticity (MPa)	2.1×10^5
Poisson's ratio (-)	0.3
Gross Cross section area (mm^2)	50 x 4 = 200
Corroded cross section area (mm^2)	50 x 2 = 100 (Note 1)
Yield strength (MPa)	440
Rupture axial strain (-)	0.2
Allowable axial strain (-)	0.15
Initial apparent friction coefficient (-)	2
Minimum apparent friction coefficient (-)	0.67
Transition confining pressure (kPa)	120

Note 1:
Gross section area was used for calculation of axial stiffness. Corroded cross section area (after maximum corrosion in 100 years) was used for calculation of axial yield strength of strips (44 kN/strip).

Table 5. Assumed properties of facing concrete segments

Gross area (m2/m)	0.14
Young's modulus, E (MPa)	2.5×10^4
Moment of inertia, I (m4/m)	2.30×10^{-4}
Density (kg/m3)	2500
Modified area (m2/m) (Note 1)	0.001

Note 1: Area reduced to account for reduction of overall axial stiffness due to inclusion of rubber pads between segments.

Numerical analysis results

Fig. 6 shows the typical pattern of post-earthquake deformations. Note that the deformations are exaggerated for clarity. The deformation pattern suggests that the lateral displacement of the MSE wall is mainly due to shear deformation of soils between the reinforcing strips. Maximum lateral and vertical displacements occurred at the top corner of the wall and behind the MSE volume, respectively. Fig. 7 shows displacement time histories at these two locations for the most severe design earthquake (CHICHI-NS-2475). Fig. 8 presents a summary of calculated displacements under all design ground motions. Maximum horizontal and vertical displacements of about 0.45m and 0.3m, respectively were calculated. Displacements significantly decreased for 975 yr and 475 yr earthquakes.

Fig. 9 shows the typical pattern of post-earthquake axial forces in the strips. Maximum axial forces occurred behind the facing and decreased gradually towards the back of the MSE wall as the loads were transferred to the ground through frictional resistance. The axial forces in the strips generally increased with depth and increased with time during earthquake shaking (Fig. 10).

Fig. 11 presents a summary of the distribution of axial forces in the strips before and after earthquake shaking. Post-earthquake curves represent the average for 6 ground motions from each return period. The long term yield strength profile of strips (after maximum corrosion, see Table 4) is also shown for comparison. All of the strips remained elastic in the static condition. The lower 6, 7 and 8 rows of the strips yielded in 475 yr, 975 yr and 2475 yr return period earthquakes, respectively.

Fig. 12 shows accumulated axial strain in the strips at the end of the CHICHI-NS-2475 earthquake. Maximum axial strains in the range of 0.6%, 1.4% and 4.4% were calculated for 475 yr, 975 yr and 2475 yr return period earthquakes, respectively.

Fig. 13-Top compares profiles of the coefficient of lateral earth pressure (K) before and after the CHICHI-NS-2475 earthquake behind the facing. The static K (before earthquake) was in the range of typical K_{Active} values. Earthquake shaking increased K to values greater than typical $K_{At\text{-}Rest}$ in the upper portion of the wall and to about $K_{At\text{-}Rest}$ in the lower portion of the wall. Fig. 13-Bottom compares profiles of K behind the MSE volume. The static K was in the range of typical K_{Active} in the upper half and gradually increased to $K_{At\text{-}Rest}$ in the lower half. Earthquake shaking decreased the average value of K due to permanent lateral displacement of the MSE volume. Fig. 14 shows the time histories of lateral earth

Fig. 6 . Typical post-earthquake deformation pattern (CHICHI-NS-2475)- deformations are 5 times exaggerated.

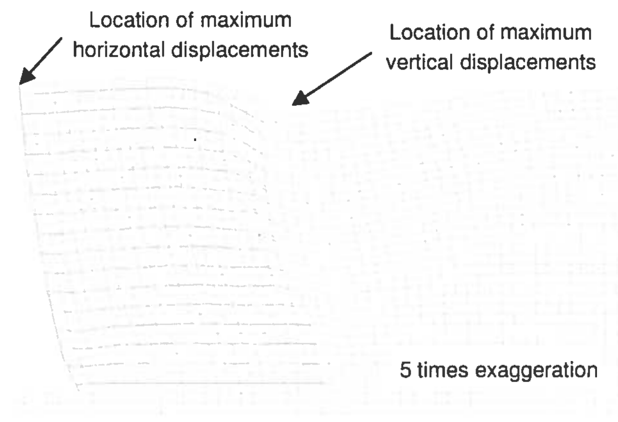


Fig. 7. Time histories of maximum horizontal and vertical displacements during CHICHI-NS-2475

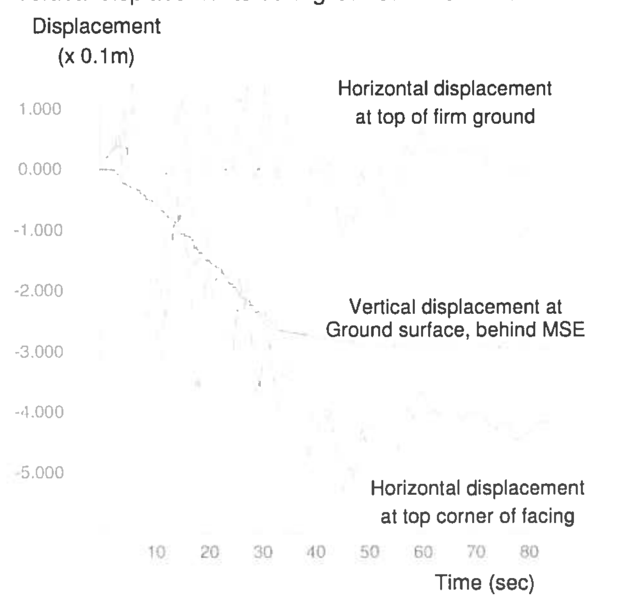


Fig. 8. Summary of calculated horizontal and vertical displacements under all design ground motions (does not include the subduction earthquake)

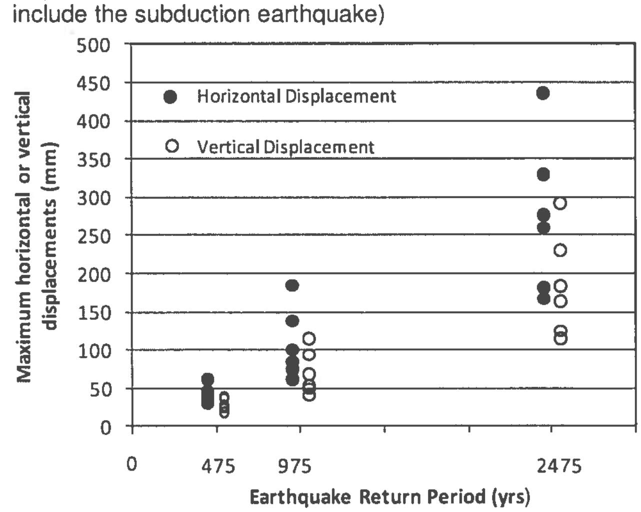


Fig. 9. Typical distribution of axial forces in strips after CHICHI-NS-2475

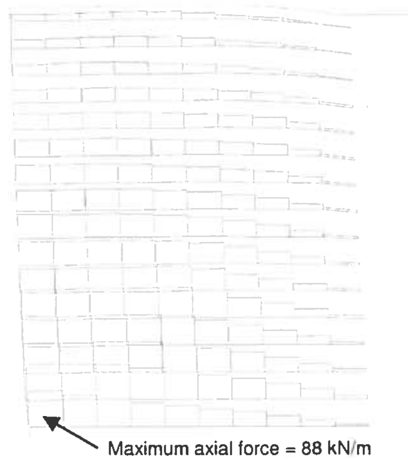


Fig. 10. Time history of axial forces in strips under CHICHI-NS-2475. Strip row numbering is from bottom to top. Negative sign signifies tensile axial force.

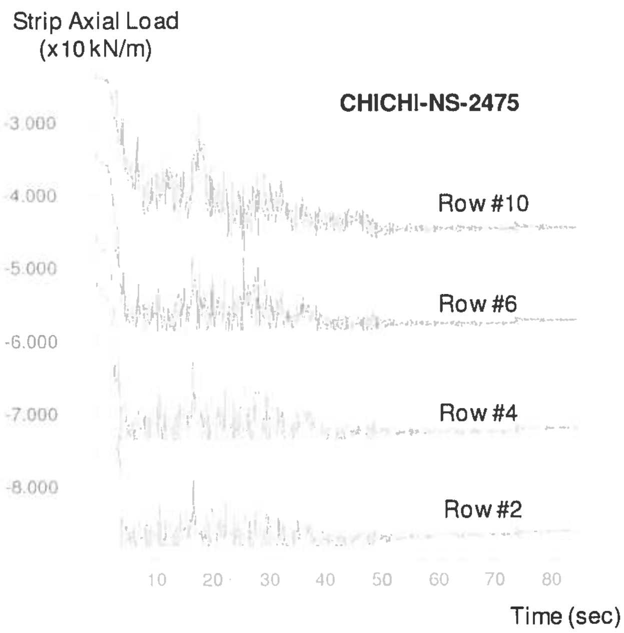


Fig. 11. Comparison of axial forces in strips before and after earthquakes. Each post-earthquake curve represents the average of 6 ground motions.

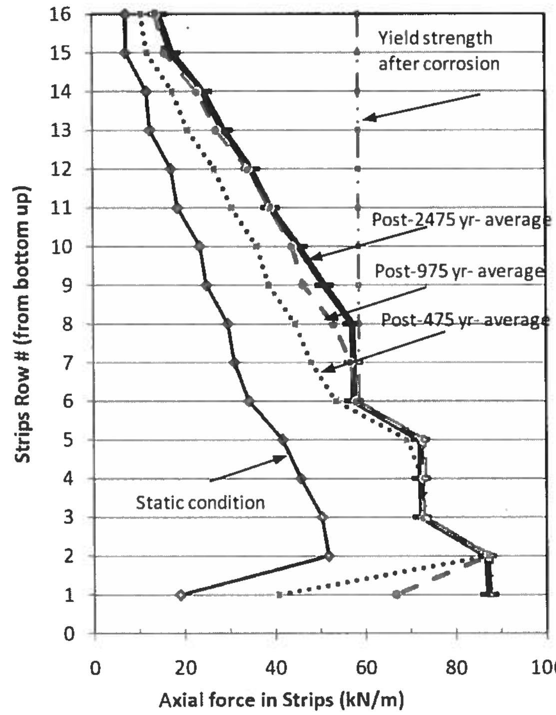
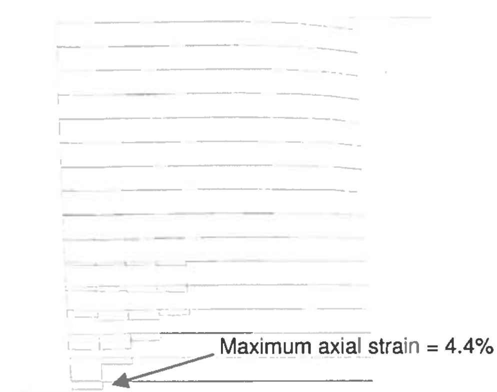


Fig. 12. Distribution of axial strains in strips under CHICHI-NS-2475



pressure at selected depths behind the facing and behind the MSE volume.

Fig. 15-Top shows typical stress-strain loops of an element in the middle of MSE wall at depth of 10.5m. Strains accumulated during the earthquake due to the presence of static shear stress bias. On the other hand, static shear bias in the far field was small and did not accumulate large shear strains (Fig. 15-Bottom).

Discussion

Numerical analysis provides much insight into behavioural patterns and modes of failure. However there is considerable uncertainty in the assumed parameters and analysis methodology, and seismic design in general. This should be understood and considered when using the results.

The calculated axial strains in the strips were well below the allowable rupture strains and are indicative of internal stability of the MSE wall.

A considerable scatter in the calculated displacement results was found among 975 yr and 2475 yr earthquake. This is partly due to the CHICHI ground motions being more severe than the other design motions. When considering the high return period and severity of earthquakes selected it is suggested that the use of the average calculated displacement rather than maximum displacement is appropriate for design. However, when considering the uncertainties in earthquake motion, soil

Fig. 13. Comparison of the coefficient of lateral earth pressure before and after CHICHI-NS-2475 earthquake (Top) Behind the facing and (Bottom): Behind the MSE volume

K values were calculated as follows:

K=ratio of effective lateral stress to static effective vertical stress

$K_{Active}=(1-\sin\phi)/(1+\sin\phi)$

$K_{At\text{-}Rest}=1-\sin\phi$

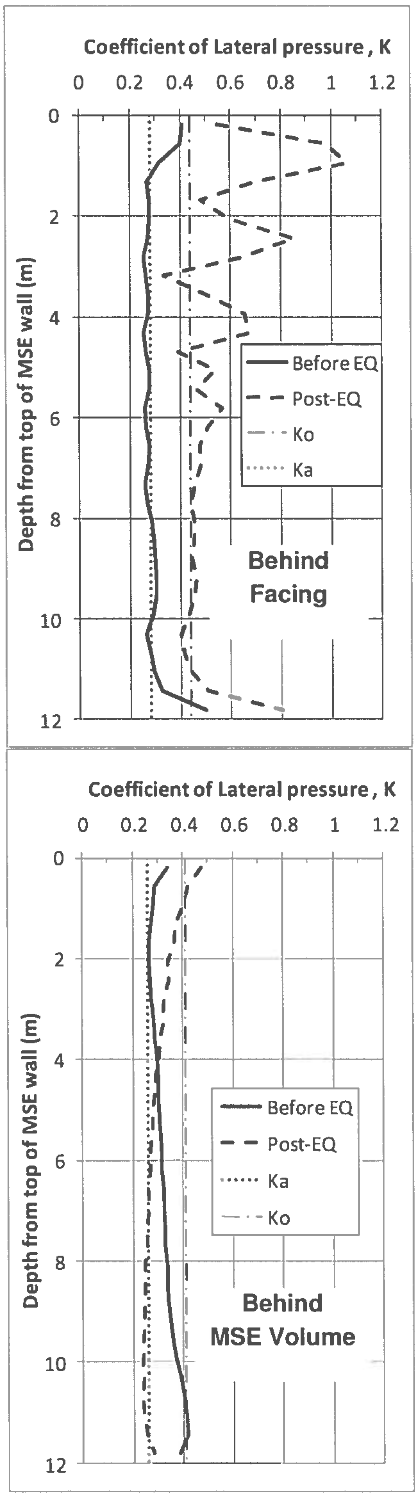


Fig. 14. Time histories of lateral earth pressure (Top) Behind the facing, (Bottom) Behind the MSE volume

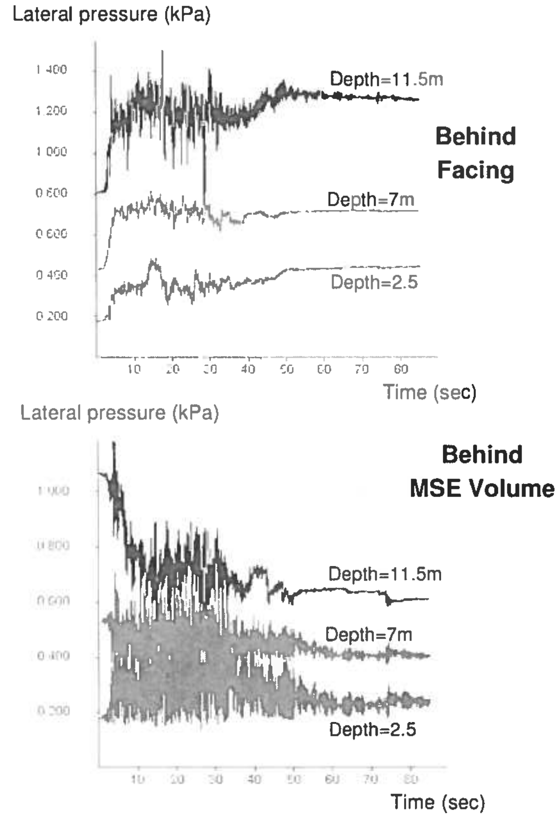
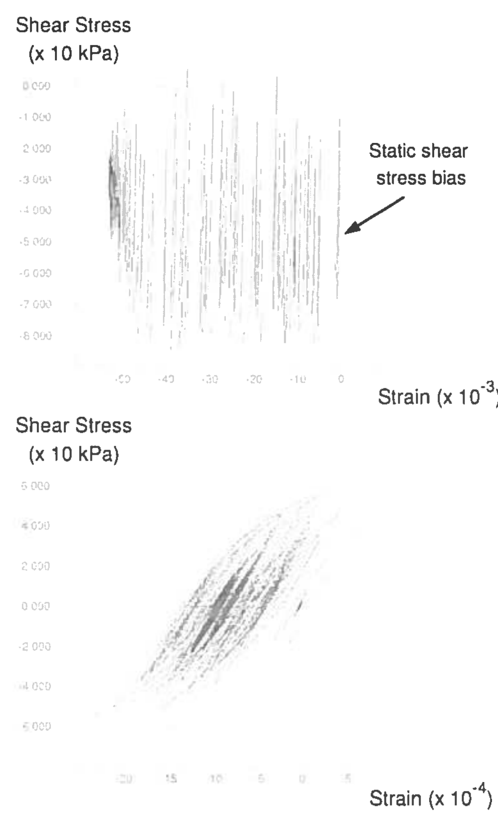


Fig. 15. Typical stress vs strain curves during CHICHI-NS-2475, (Top) In the middle of MSE volume at 10.5m depth, (Bottom) In backfill at long distance from the MSE wall at 10.5m depth



parameters, and the analyses methodology; the actual displacements could easily vary from 0.5 to 2 times the calculated values.

It should be noted that the calculated average displacements (especially for the 475 yr and 975 yr motions) are in the range of observed displacements of post-earthquake surveyed MSE walls. The relatively low calculated strains in the strips also agree with good past performance of MSE walls in earthquakes.

Initially the FLAC built- in hystyeretic damping was tried in lieu of the UBCHYST constitutive model, however unreasonable results were obtained when there was a static bias (unreasonably small displacements and in some cases permanent displacements in a direction opposite the static bias) and therefore its use was discontinued.

Summary and conclusions

A series of dynamic numerical analyses were carried out for a 12m high MSE wall with Reinforced Earth system founded on firm ground. The main conclusions are:

1. Numerical analyses indicated displacements in the range of less than 0.1m, 0.2m and 0.5m under 475 yr, 975 yr and 2475 yr design earthquakes, respectively. The average displacement of the six earthquake records at each return period were approximately half the above values.

2. Some strips in the lower rows yielded structurally. However the maximum calculated axial strains were less than 0.6%, 1.4% and 4.4% under 475 yr, 975 yr and 2475 yr design earthquakes, respectively and well below the allowable rupture strain of 15%.

Acknowledgment

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