NUMERICAL ANALYSIS OF UNDERGROUND STATION UNDER EARTHQUAKES – PSEUDO-STATIC METHOD VS. DYNAMIC TIME HISTORY ANALYSIS

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Abstract

Numerical approaches for analysing the seismic performance of underground structures vary from simplified pseudo-static method to sophisticated dynamic time history analysis. In the pseudo-static method, the horizontal ground deformation derived from one-dimensional, free-field site response analysis is applied at the side boundaries of a finite element/finite difference model of a cross section of the structure and soil system to study the forces and deformation induced in the structure. While this method is convenient to be carried out to study the effects of seismic ground motion on underground structures, it ignores the inertia of the structure which is justified when the inertia of the structure is small compared to that of the surrounding ground. In cases where the inertia of the structure is significant, for example, when there is mass concrete fill within the structure, the simplified pseudo-static method may not be sufficient to capture the response of the structure under seismic loading. This study compares the results of pseudo-static analysis and dynamic time history analysis on an underground station under different conditions including level of ground motion and presence of large inertial mass in the underground structure. For this purpose, an advanced non-linear finite element simulation approach is developed to undertake the time history analysis. The conventional design approaches commonly adopted in practice are assessed and the conditions under which the simplified pseudo-static method can be considered sufficient for design purposes are discussed.

Keywords: underground structure; pseudo-static method; dynamic time-history analysis; seismic performance

1. Introduction

The rapid urbanization and scarcity of valuable land enhance the use of underground spaces. The underground facilities in areas subject to earthquake activity must withstand both seismic and static loading (Fig. 1). The underground facilities historically have experienced a lower rate of damage than surface structures. However, the recent experiences revealed that the underground structures can suffer significant damage in large earthquakes. Nguyen et al. (2019) [1] stated several studies that documented the observed damage of tunnels in various earthquakes.

The seismic behaviour of underground structures differs from surface structures [2], mostly due to two distinct features: (1) underground structures are surrounded in ground and cannot vibrate freely, and (2) geometry in which the length is much longer than the cross-sectional dimension. For most underground structures the inertia of the surrounding soil is large compared to the inertia of the structure. Thus, the focus of underground seismic design normally is the deformation of the ground and its interaction with the structures [3].

The shallow transportation underground structures (i.e. subway stations, highway tunnels) are usually box-shaped cut-and-cover structures. The seismic characteristics of box-shaped underground structure is very different from circular tunnels considering that the box-shaped structure does not transmit loadings as efficiently as a circular lining. The rectangular box structure will undergo transverse racking deformations when subjected to shear distortions during an earthquake as shown in Fig. 1.

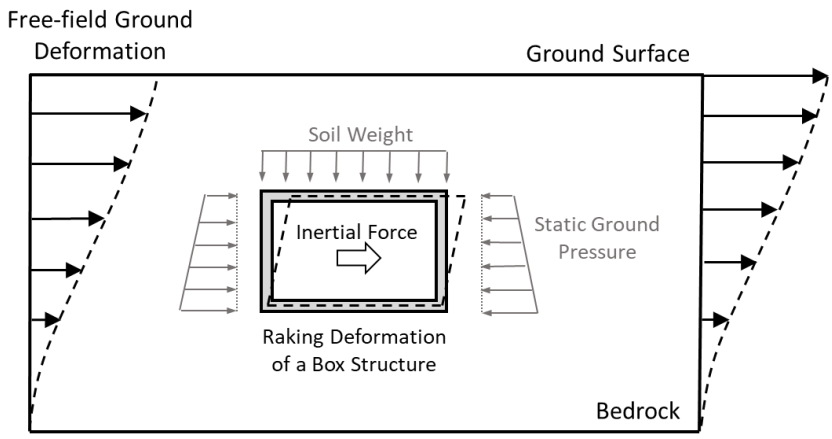

Fig. 1 – Schematic of underground box structure raking due to ground seismic deformation

The available approaches for the seismic analysis of the underground structures can vary in complexity and computational effort from simplified pseudo-static method to sophisticated dynamic time-history analysis. In this study, the response of an underground station under earthquake ground motion is analysed using both pseudo-static analysis and time-history analysis. The results are compared to highlight the appropriateness of the methods under different conditions including level of ground motion and presence of large inertial mass in the underground structure.

2. Case Study

2.1 Underground station specifications

A typical cross-section, illustrating the configuration of an underground station, is shown in Fig. 2. The station box is 400m long, and comprises of two track levels, with the upper track level being essentially a surface railway located above the lower track through the station for ease of connectivity. The primary structure is constructed by the top-down methodology and is formed from two lines of diaphragm walls and two slabs (i.e. the top and bottom slabs). The thickness of the diaphragm walls is 1.2m, while both the top and bottom slabs are 1.0m thick. Internally, there is a 1.2m thick, continuous wall of barrette piles that support the two primary slab levels and the underpinning portals. Additionally, the structure comprises of internal slabs and walls, of varying thickness, that define the line of the railway. The consequent, unused volumes created by these internal walls are filled with mass concrete to counter the tendency for the structure to be buoyant.

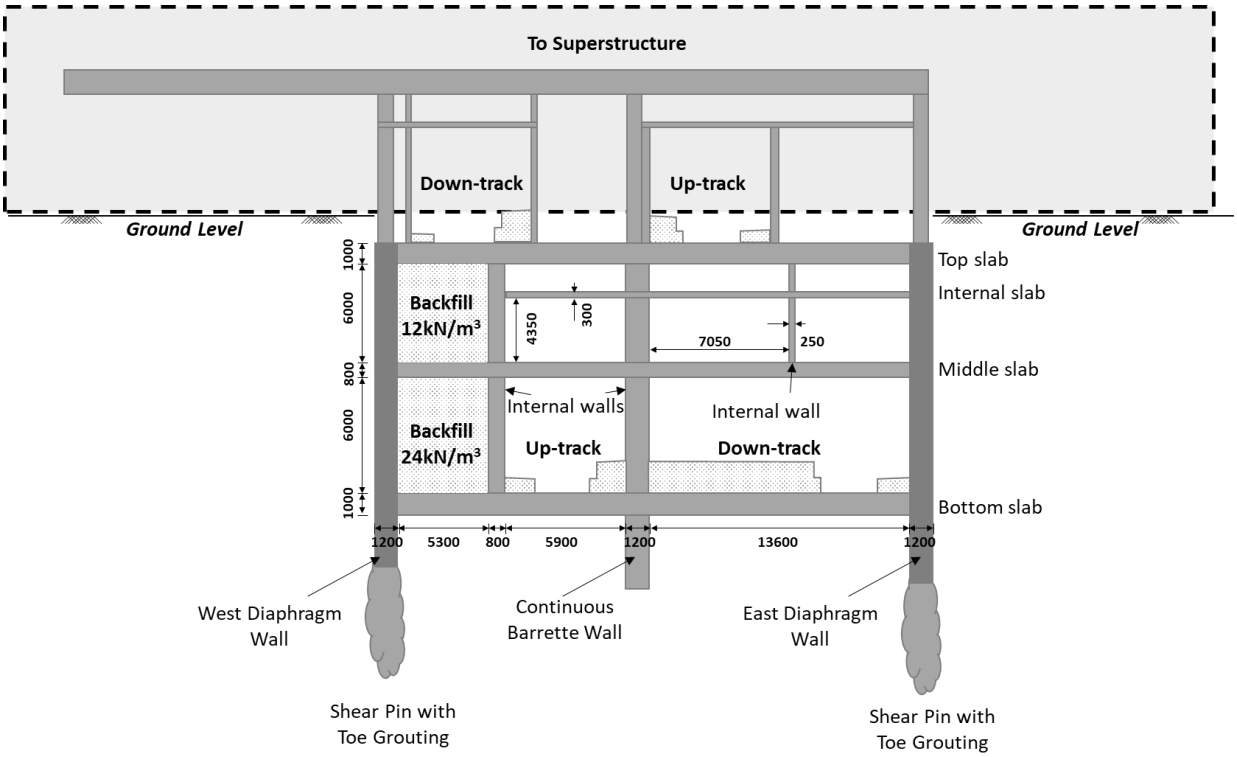

Fig. 2 – Cross-section of underground station

2.2 Ground conditions

The simplified ground profile is shown in Fig. 3(a). The ground generally consists of surficial sandy fill materials, which overlay silty marine deposit, which in turn overlay alluvium sand and then completely decomposed granite (CDG) above the bedrock. The bedrock is encountered at a depth of ~16.5m. The groundwater level is generally at 1m below the existing ground level.

The shear modulus degradation curves represent the reduction in soil shear modulus (G/G_0) as a function of soil shear strain are required for free-field site-response as well as time-history soil-structure interaction analyses. The degradation curves adopted for each soil type are shown in Fig. 3(b) which were defined based on literature and previous experience on similar geology.

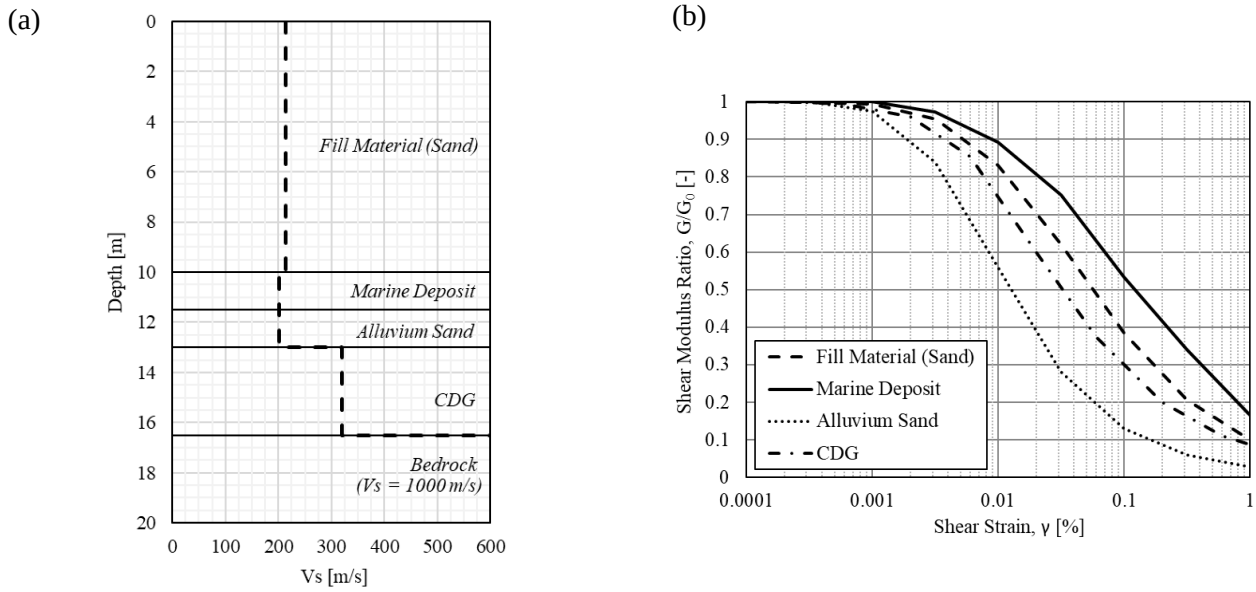

Fig. 3 – (a) Simplified ground profile; (b) Shear modulus degradation curves

3. Design Earthquake Ground Motions

The design earthquake characteristics were derived based on site-specific Probabilistic Seismic Hazard Assessment (PSHA) for the region considering all the available seismicity data, geological and tectonic information within 500km distance [4]. The uniform hazard response spectrum (UHRS) at bedrock for 1000-year return period is shown in Fig. 4.

The time history records were selected from database of measured real ground motion records that match the de-aggregated earthquake magnitude and distance combinations that contribute most significantly to the hazard at the site. The selected time history records were modified by the spectral matching technique to match the uniform hazard response spectrum (UHRS) for the site. Without modification, the response spectra of the selected time histories in most cases will not closely match the target design response spectrum. The bedrock time histories spectrally matched to the target UHRS are shown in Fig. 5.

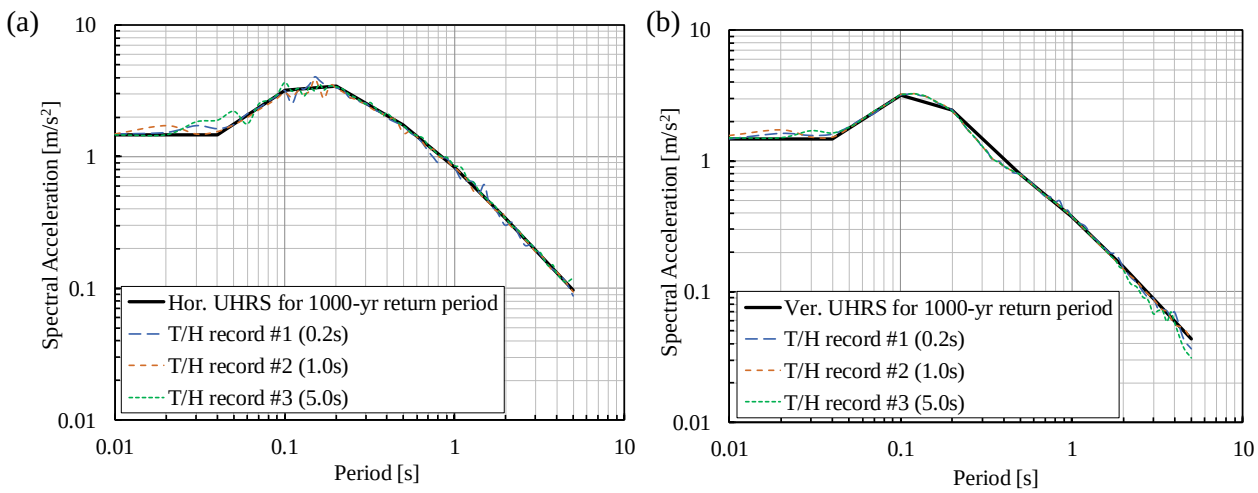

Fig. 4 – Uniform hazard response spectrum (UHRS) and spectrally matched earthquake time-history records: (a) Horizontal component; (b) Vertical component

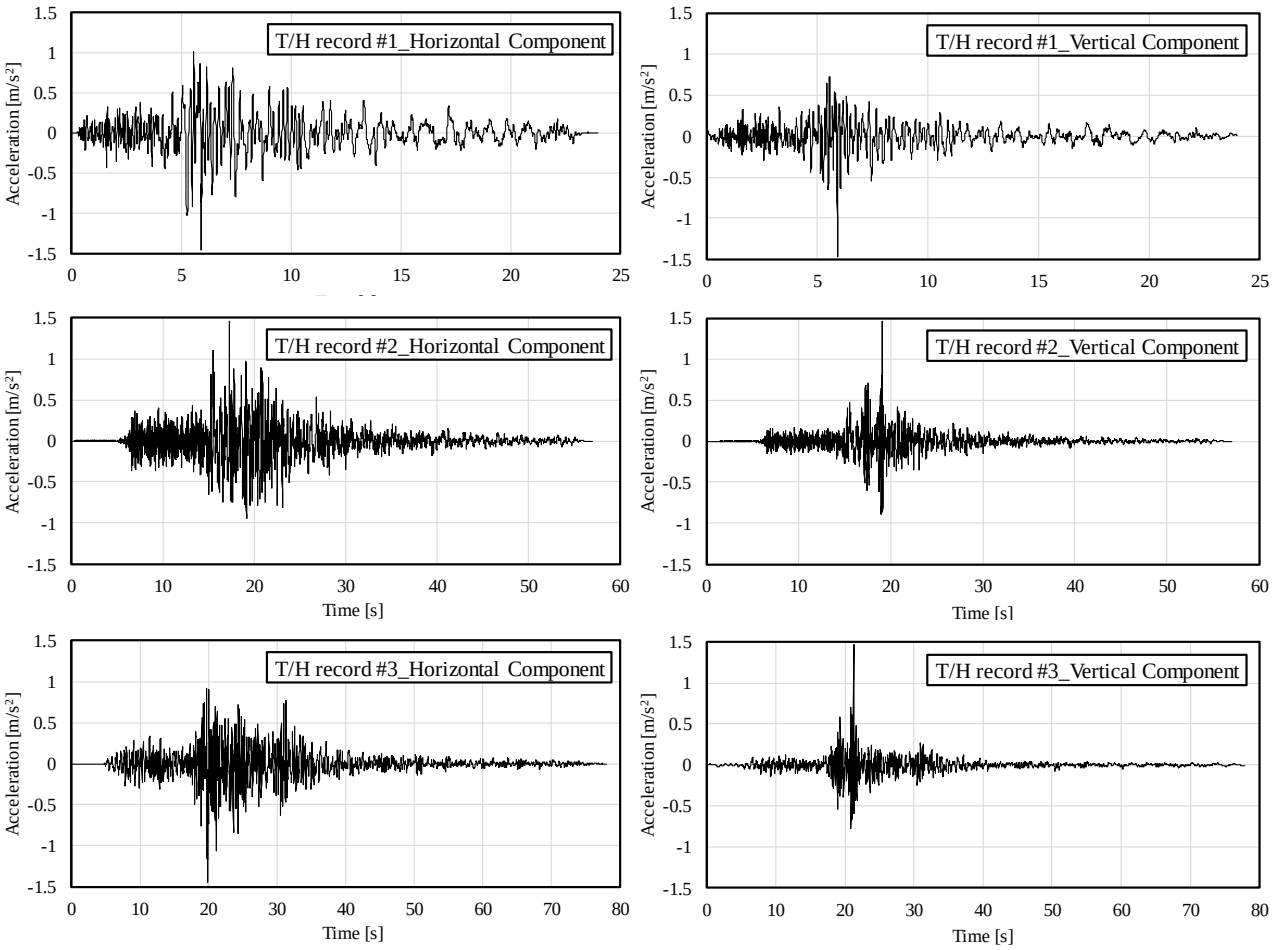

Fig. 5 – Spectrally matched earthquake time-history records at bedrock

4. Two-stage Method (Pseudo-static)

Pseudo-static method is considered as convenient method and is widely applied in research and design practice [5]. In this method, the free-field ground deformations due to a seismic event are estimated, and the underground structure is designed to accommodate these deformations. This approach is considered satisfactory particularly when low levels of shaking are anticipated, or the underground facility is in a stiff medium such as rock.

The procedure involved in the two-stage subgrade reaction method is as follows:

- Step 1) Carry out site-specific free-field site response analysis to estimate the maximum lateral ground movement profile (Section 4.1).
- Step 2) Carry out pseudo-static soil-structure interaction analysis by applying the estimated maximum ground movement to side boundaries (Section 4.2).

The detailed methodology and its application in design further explained by Free et al. (2001) [6].

4.1 Free-field site response analysis

Site-specific response analysis is carried out using *Oasys* SIREN [7] software to determine the maximum ground lateral deformation profile under the design earthquake ground motions. Oasys SIREN is a finite difference program that analyses the response of a 1-dimensional soil column subjected to an earthquake

motion at its base. In Oasys SIREN, the earthquake motions are modelled as vertically propagating shear-waves, and the soil column is specified as a series of horizontal layers, each layer being modelled as a non-linear material with hysteretic damping. The soil damping is calculated by SIREN from the defined degradation curves for each soil layer (Fig. 3(b)) using Masing's rule.

The estimated maximum lateral ground deformation profile under the design earthquake ground motions is shown in Fig. 6.

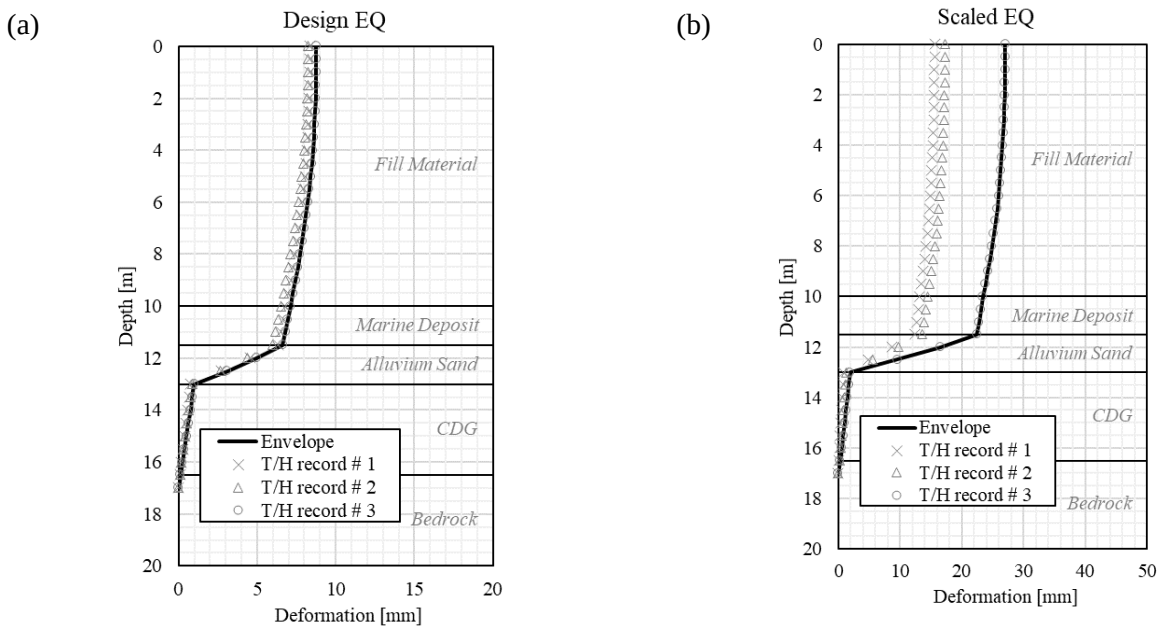

Fig. 6 – Maximum horizontal ground displacement from site response analysis; (a) design earthquakes; (b) scaled earthquakes (twice the design earthquake ground motion)

4.2 Pseudo-static finite element analysis

Plaxis 2D software is used for pseudo-static finite element analysis following the conventional procedure. The maximum free-field displacement profile as estimated in Section 4.1 is applied to Plaxis model boundaries together with imposing a pseudo-static horizontal acceleration to the whole model such that the desired maximum free-field lateral displacement profile is uniform across the soil mass. The Plaxis model setup is shown in Fig. 7.

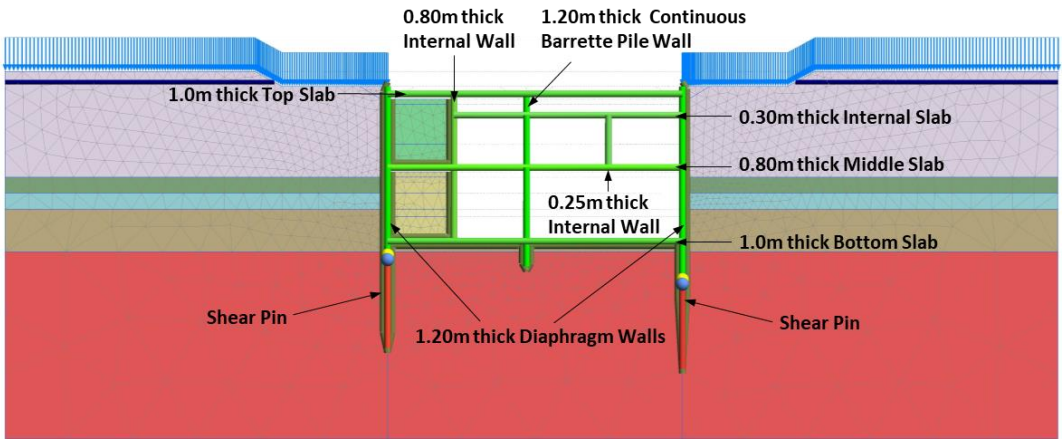

Fig. 7 – Plaxis model setup

5. Time-History Analysis

The general-purpose finite element program LS-DYNA was used for time-history analysis. This program is an explicit dynamic code that runs in the time domain and incorporates both structural and soil non-linearity. It is equipped with a large number of material models that can be used to model the soil and structure, and several contact algorithms that can be used to model the foundation-soil interface. Arup has been the co-developer of the software and many of LS-DYNA's capabilities for civil engineering were developed by Arup [8]. This software has been successfully used by Arup over many years for a wide range of foundation and soil structure interaction problems [9, 10].

The developed LS-DYNA model is shown in Fig. 8. The soil nonlinear behaviour under cyclic loadings was considered by defining the soil shear modulus degradation curves for each soil type (Fig. 3(b)). The soil model consisted of 8-noded solid elements using the non-linear soil material model MAT_HYSTERETIC_SOIL. This model captures the hysteresis of the soil under cyclic loading where energy dissipation under cyclic response is modelled explicitly (and automatically) as the area enclosed by the shear stress-shear strain hysteresis loops, rather than by an equivalent element viscous damping matrix. The structural elements including slabs, internal walls, D-walls and barrette pile walls were modelled as elastic elements.

Non-reflecting boundary conditions are defined at the base of the model. Non-reflecting boundaries are normally used on the exterior boundaries of an analysis model of an infinite domain, such as a half-space to prevent the reflection of the outward propagating waves back into the model and contaminating the results. Each soil layer side (left and right) boundary node set is constrained, not to allow relative displacement. Each constraint is also globally restrained from rotation.

The overall dimensions of the model were chosen as sufficiently large ensuring the free-field conditions close to the edge of the model. The ground motion time-history records were applied at the model base as a bi-axial earthquake excitation in horizontal and vertical directions.

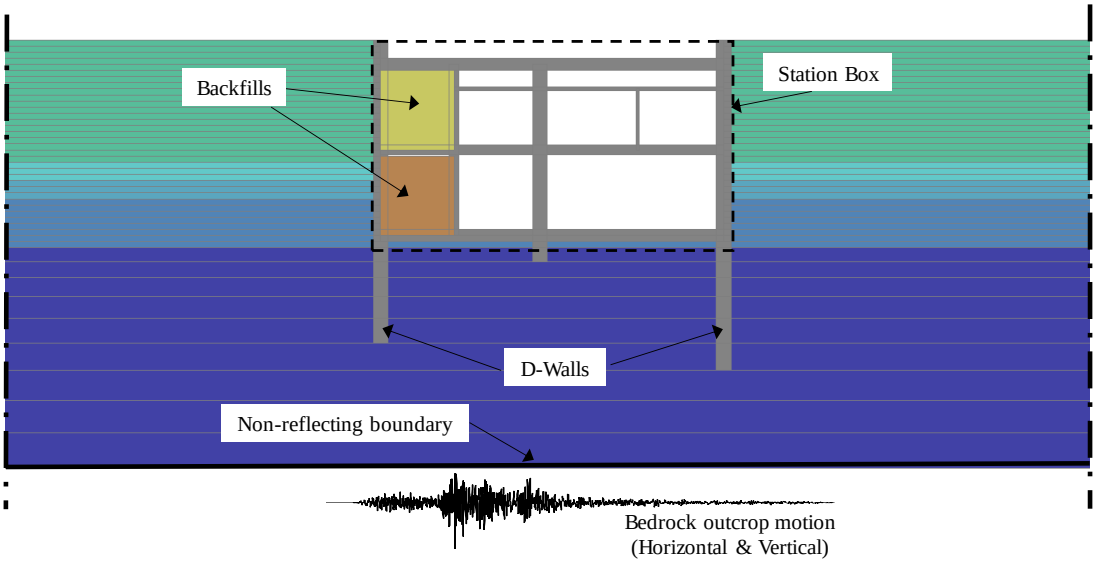

Fig. 8 – Numerical simulation in LS-DYNA

6. Results and Discussion

The results of the pseudo-static analysis and time-history analysis are compared in terms of the additional bending moment induced by ground motion in the top and bottom slabs, diaphragm walls, and the internal wall adjoining the confined soil backfills. The net bending moment presented excludes the bending moment

due to the self-weight of the structure, and the static earth and water pressures acting on the structure. The results of the following cases are presented:

1. Response of structure under the design earthquakes.
2. Response of structure under increased ground motion which is taken as two times the earthquake ground motion.
3. Comparison of the response of structure with and without mass concrete fill under both the design earthquakes and increased ground motion.

Results of the above cases are studied to assess the appropriateness of the pseudo-static analysis to model the seismic response of underground structure under different ground motions and with different masses.

6.1 Response of underground station under design earthquakes

The additional bending moments induced by the design earthquakes in the top and bottom slabs are shown in Fig. 9. The additional bending moments calculated in pseudo-static analysis and time-history analysis match reasonably well in general, except at the slab/wall joints of the bottom slab where pseudo-static analysis gives higher values. Consideration of vertical ground acceleration in the pseudo-static analysis has insignificant impact on bending moments in the slabs. Fig. 10 shows the additional bending moments in the diaphragm walls and the internal wall adjoining the confined soil backfills. The pseudo-static analysis gives higher bending moments at the slab/wall joints in general. The mismatch of bending moments at the joint between the bottom slab and the west diaphragm wall is due to the relatively short wall depth below the bottom slab in the Plaxis model. Nevertheless, the pseudo-static analysis is considered to be appropriate for the seismic design of underground structure under the considered design earthquakes.

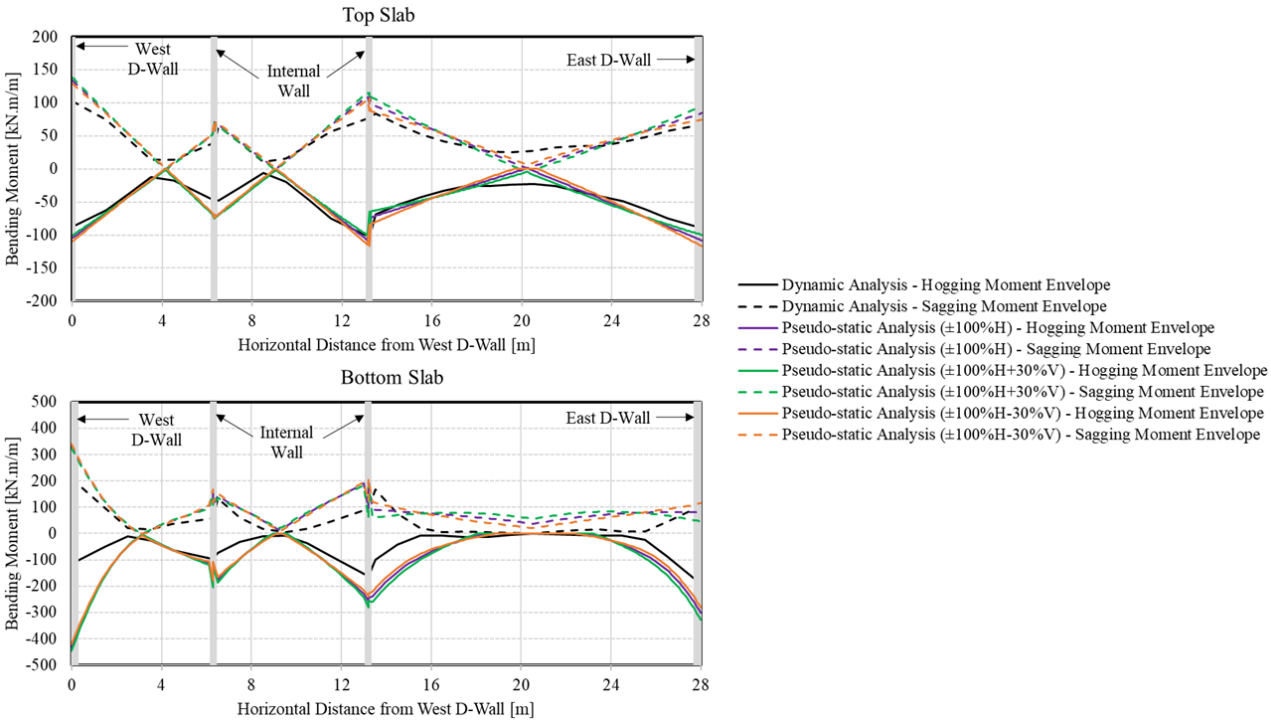

Fig. 9 – Bending moment of top and bottom slabs (Plaxis vs LS-DYNA)

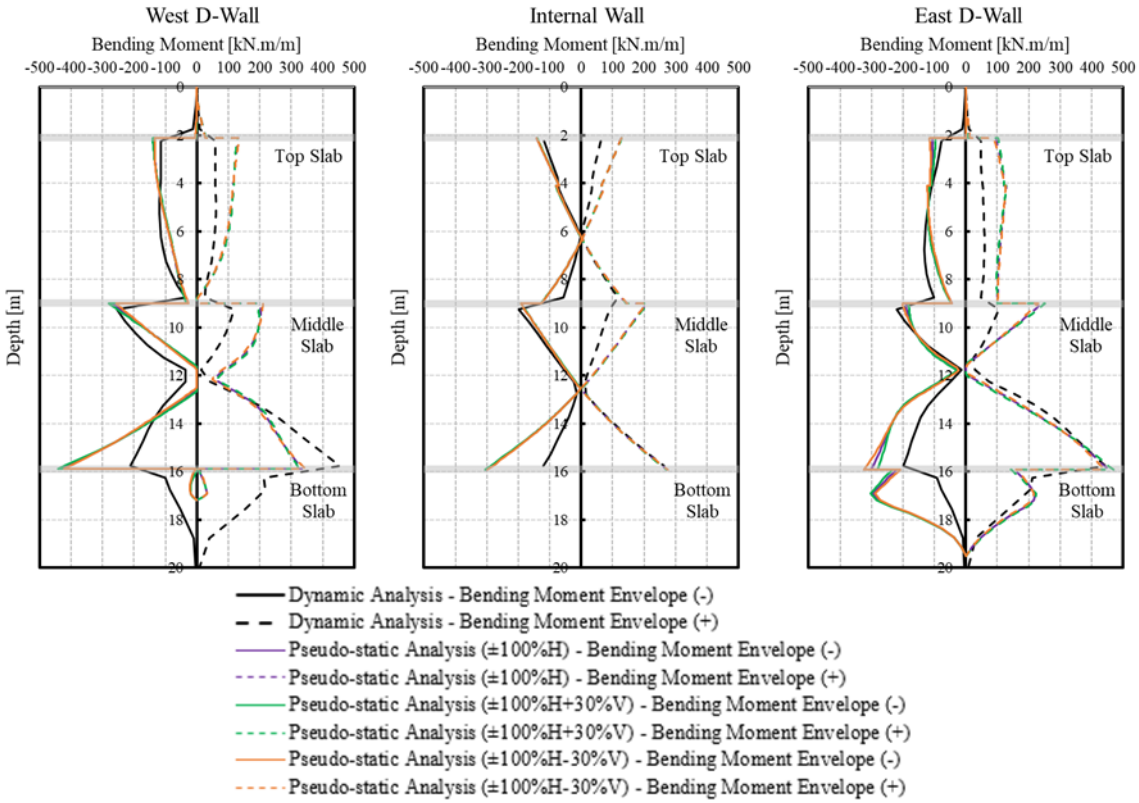

Fig. 10 – Bending moment of D-Walls and internal wall next to backfill (Plaxis vs LS-DYNA)

6.2 Response of underground station under increased ground motion

The additional bending moments induced by the increased ground motion (two times the design earthquake) in the top and bottom slabs are shown in Fig. 11. The pseudo-static analysis overestimates the bending moments in both the top and bottom slabs. The difference in bending moments between the pseudo-static analysis and the time-history analysis is particularly noticeable in the top slab under increased ground motion. The additional bending moments in the diaphragm walls and the internal wall, shown in Fig. 12, also show an overestimation of values at the slab/wall joints in general. The percentage difference is similar to that under the design earthquakes discussed in Section 6.1. In terms of the response of slabs, the results of pseudo-static analysis are less satisfactory under increased ground motion.

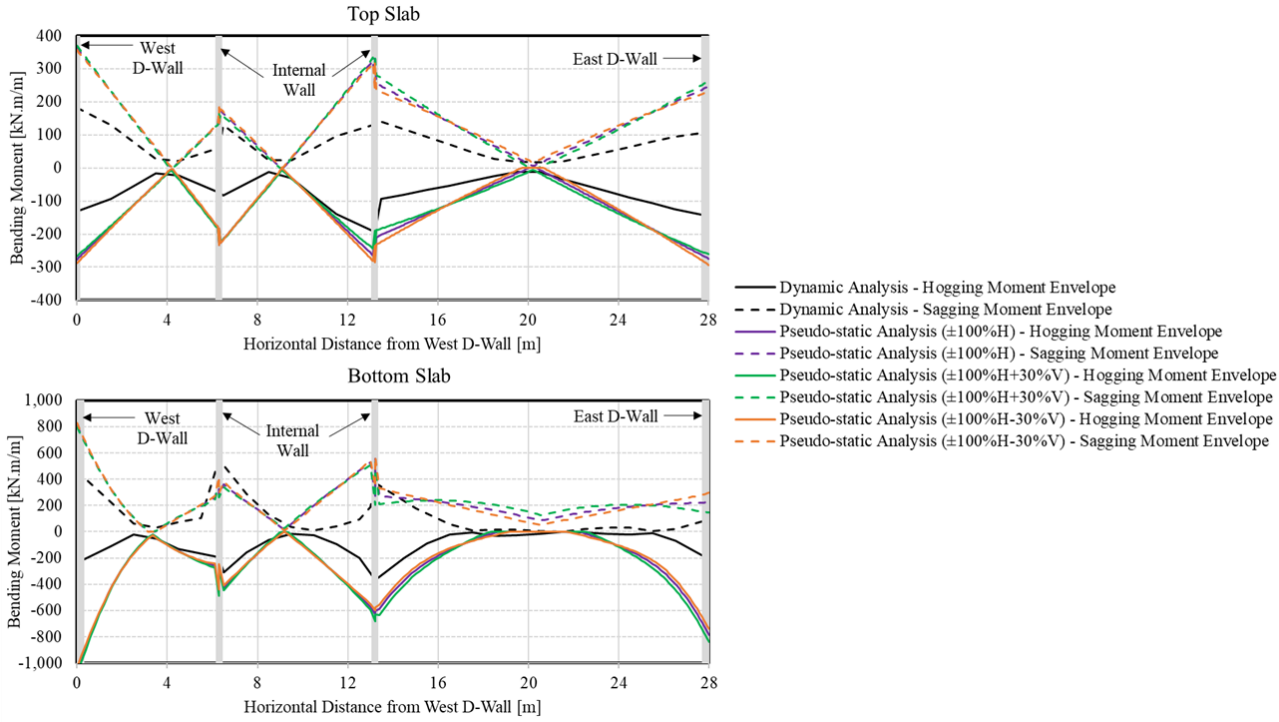

Fig. 11 – Bending moment of top and bottom slabs (Plaxis vs LS-DYNA) – Increased ground motion

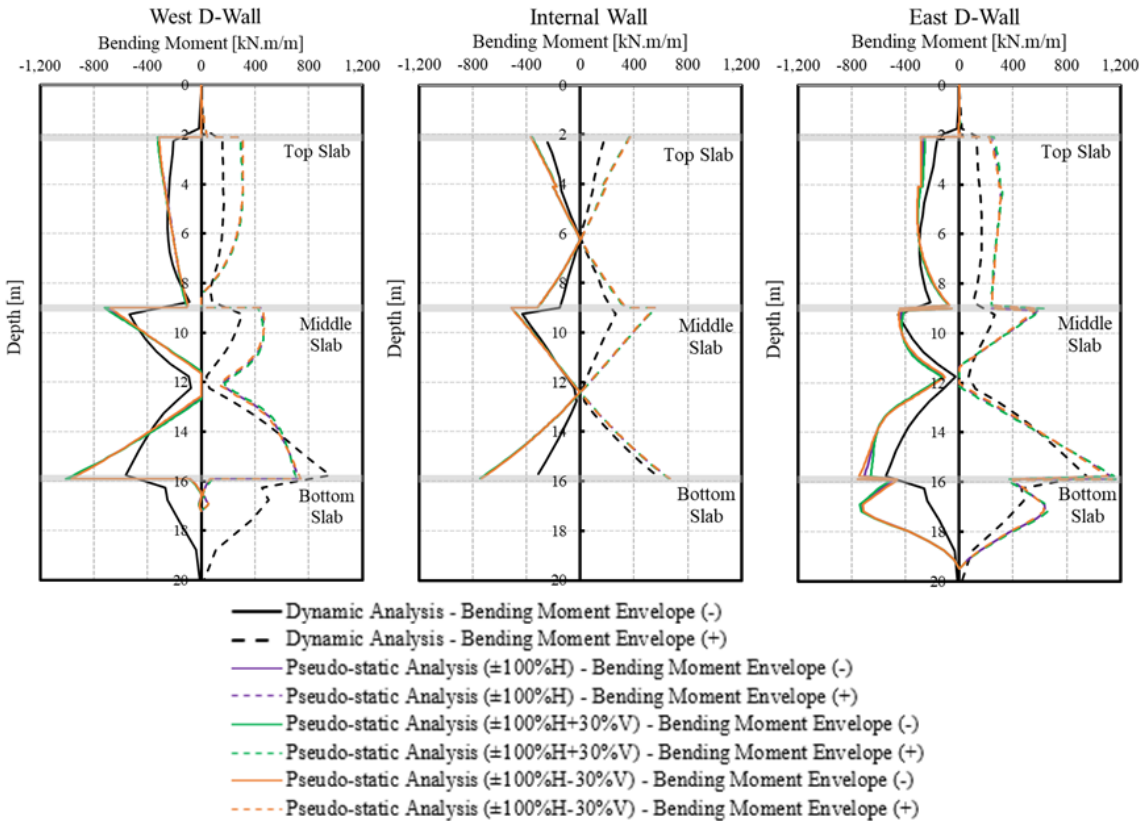

Fig. 12 – Bending moment of D-Walls and internal wall next to backfill (Plaxis vs LS-DYNA) – Increased ground motion

6.3 Influence of mass concrete fill

To study the influence of mass concrete fill which increases the inertial mass of the underground structure, the additional bending moments induced by seismic ground motion in the west diaphragm wall and the internal wall adjoining the mass concrete fill are compared in Fig. 13. Under the design ground motion, the bending moments in the walls are similar in cases with and without mass concrete fill in both pseudo-static and time-history analyses. The influence of inertial mass of structure is more apparent under increased ground motion, where the time-history analysis gives different bending moment profiles in cases with and without mass concrete fill in the west diaphragm wall. The results of the pseudo-static analysis are less sensitive to the inertial mass of the structure even under increased ground motion.

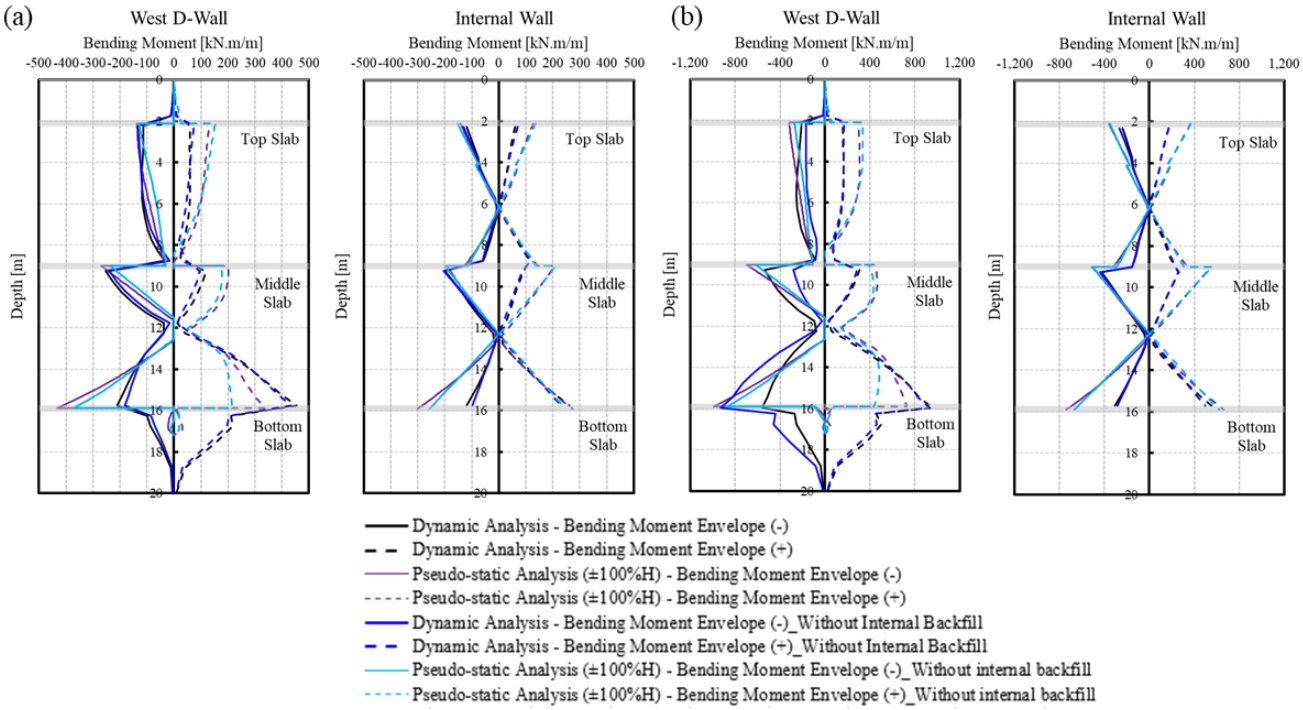

Fig. 13 – Bending moment of walls (with vs without mass concrete fill); (a) Design ground motion; (b) Increased ground motion

7. Conclusions

The response of an underground station under earthquake ground motion in terms of bending moments induced in slabs and walls are analysed using simplified pseudo-static method and dynamic time-history analysis. The results from the two methods of analysis are compared to assess the appropriateness of the methods under different conditions including level of ground motion and presence of large inertial mass in the underground structure. Under the considered design earthquake ground motion (PGA = 0.15g for a return period of 1000 years), the results of the pseudo-static analysis compare reasonably well with the results of time-history analysis, albeit higher estimation of bending moments at slab/wall joints. The pseudo-static analysis gives less satisfactory results under increased ground motion (twice the considered design earthquake ground motion), especially for the top slab where overestimation of bending moments at slab/wall joints is apparent. The effect of inertial mass of underground structure is not significant under the considered design earthquake ground motion. Under increased ground motion, time-history analysis can

capture the inertial response of the underground structure while pseudo-static analysis does not reflect the effect of inertial mass.

8. References

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