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Numerical study of dynamic structure-soil-tunnel interaction for a case of the Thessaloniki Metro

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ABSTRACT: The paper investigates the dynamic interaction between a metro station and a building with basement, prospecting the station, when subjected to seismic excitation in the transversal direction. A real case study of the under-construction metro in Thessaloniki, Greece is actually analyzed under plane strain conditions, employing ABAQUS, with the building being simulated in a simplified fashion as an equivalent single-degree-of-freedom oscillator. The results of the above analyses are compared with the predictions of additional analyses neglecting the building, so as to identify and quantify the effects of the dynamic interaction between the structures on the racking response of the station, as well as on seismic earth pressures and dynamic bending moments developed on the station diaphragm walls. The study indicates a general increase of the seismic response of the station due to the presence of the building. Additionally, a new racking ratio – flexibility ratio (*R-F*) relation is proposed to better describe the response of large metro stations, since the existing relations are found to be inadequate for this type of embedded structures.

1 INTRODUCTION

The seismic analysis and design of tunnels and embedded structures commonly disregards the dynamic interaction that may be mobilized between them and existing buildings and other above ground civil infrastructure. However, during ground seismic shaking, significant interaction phenomena may be mobilized between buildings, and an embedded structure, passing few meters below or close by the foundations of these buildings, which may alter the seismic response of both structures, compared to the one predicted for 'greenfield' conditions. These interaction phenomena are expected to be important in densely constructed urban environments.

Recent studies investigated the effects of cavities or embedded structures on the seismic loading at the ground surface (e.g. Abuhajar et al. 2015, Baziar et al. 2016), or on the seismic response of above ground structures (e.g. Wang et al. 2013). The effect of above ground structures on the seismic response of the embedded structures has also received attention the last years. However, the limited available studies have focused on the response of shallow tunnels (e.g. Pitilakis et al. 2014, Dashti et al. 2016, Abate and Massimino 2017, Tsinidis 2018).

The present study focuses on the dynamic interaction between a large metro station and a building with basement, prospecting the station, when subjected to seismic excitation in the transversal direction. More specifically, a metro station-soil-building configuration, which replicates a real case study of the under-construction metro in Thessaloniki, Greece, is numerically investigated. Particular emphasis is placed on the effect of the existing building on the seismic earth pressures and dynamic lining forces developed on the station walls. Additionally, a new racking ratio – flexibility ratio (R-F) relationship is proposed on the basis of pseudo-static analyses, so as to better describe the response of this type of structures, since the existing relationships are found to be inadequate for large metro stations (Tsinidis & Pitilakis, 2018).

2 CASE STUDY & NUMERICAL SIMULATION

2.1 *The Analipseos station of the Thessaloniki Metro*

The case study investigated herein refers to the Analipseos metro station in Thessaloniki, Greece that has been recently constructed with a top-down cut and cover method. The station, which is located in a densely-urbanized area, is 200 m long and around 22 m wide. It is composed of three underground levels. Levels -1 and -2 have a height of 6 m, while the height of floor -3, where the rail is located, is 8 m high. The roof slab has a thickness of 1.5 m and is covered by 2 m of backfilled soil. The bottom slab is 2 m thick, while the thickness of the diaphragm walls is 1.2 m. The latter elements continue for 20 m more under the foundation plate. The internal slabs have a thickness of 1.0 m. All structural elements were constructed by C30/37 reinforced concrete. The station is embedded in a soil deposit, which can be subdivided into three shallow layers of softer soil, overlying a stiffer subbase layer and the bedrock. Table 1 summarizes the geotechnical parameters of the main layers.

A six-storey building with basement that is located close to the station, i.e. 3 m from the station's diaphragm wall, was selected to investigate the dynamic interaction effects between the above ground buildings and the station. The building, built in 1985, has a total height of 19 m and consists of mixed type frame-shear walls supporting system. Its basement is completely made of shear walls and has a height of 4.0 m and a length of 9.0 m in the transverse direction of the station, which is analysed herein.

2.2 *Numerical simulation*

The station-soil-building configuration was simulated numerically, assuming plane strain conditions and employing the finite element code ABAQUS (ABAQUS, 2012). The numerical model layout is presented in Figure 1. The depth of the soil grid was set equal to 120 m, where the bedrock is found. The width of the soil grid was selected equal 300 m, based on the results of a sensitivity analysis that was conducted to investigate potential boundary effects on the computed response at the central area of the numerical model, i.e. where the station and the structures are located. The soil was meshed with quadratic plane strain elements, while the station slabs and diaphragm walls were modelled by means of beam elements. The extension

Table 1. Geotechnical parameters of the soil layers

Layer - Description	Depth	Density	Cohesion	Friction angle	Shear wave velocity
	z (m)	? (t/m3)	c' (kPa)	f' (?)	Vs (m/s)
1 - Artificial fills and debris	0 – 3	1.85	0	30	180
2 - Sandy-silty clays to clayey sands	3 – 36	2.01	10	30	240-350
3 - Stiff sandy-silty clays to clayey sands	36 – 40	2.07	40	27	450
4 - Hard sandy-silty clays to clayey sands	40 – 120	2.17	50	25	550-750
bedrock - Green Schists and Gneiss	120	2.30	-	-	1100

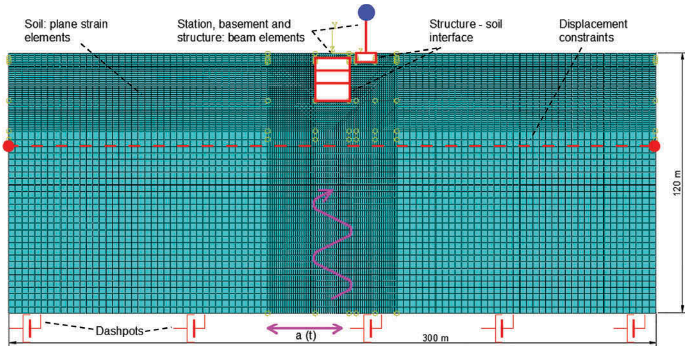

Figure 1. Numerical model of the examined station-soil-building configuration in ABAQUS

of the walls below the foundation slab was not considered in the analyses for simplification purposes. The soil elements size allowed for the efficient reproduction of all the waveforms of the whole frequency range under study (i.e. f = 0.2 - 10 Hz).

The building was simulated in a simplified fashion as an equivalent single-degree-of-freedom (SDOF) oscillator, founded on a 'shallow embedded box', representing the basement. The dynamic properties of the SDOF oscillator were defined through a modal analysis of the above ground storeys of the building, carried out in SAP 2000 (CSI, SAP2000 20) assuming a fixed-base condition at the foundation level. This analysis yielded a period T_{fix} = 0.32 s for the first translational mode in horizontal direction along the transverse section of the model, examined herein. The participating mass ratio that corresponded to the particular mode was also defined from the modal analysis. This mass was used to define the mass per unit length (in the out of the investigated plane direction), which was equal to 30 t/m and introduced as a mass element atop of the SDOF oscillator. For the above parameters (period and mass) the equivalent stiffness of the SDOF was defined and assigned in the SDOF of the 2D model, which was simulated by means of beam elements. The basement was modelled by means of beam elements as well, having a thickness of 0.4 m and 0.2 m for the slabs and the walls, respectively. A rigid connection between the SDOF oscillator base and the roof slab of the basement was established, by employing appropriate kinematic constraints, i.e. tie type in ABAQUS.

The interfaces between the station and the soil, as well as between the basement of building and the soil were modelled, implementing a small sliding hard contact model (ABAQUS, 2012). The model allows for the potential detachment and/or sliding between the interacting elements during ground seismic shaking. The shear behaviour of the interfaces was controlled by the classical Coulomb friction model, through the introduction of a friction coefficient, μ, set equal to 0.6.

A linear elastic model was implemented for the simulation of the station and the building basement response, assuming a Young's modulus E = 32 GPa, a Poisson's ratio v = 0.2 and a density ρ =2.5 t/m^3. The response of the SDOFs was simulated in a similar fashion. A viscous damping of 5 % was introduced in the form of Rayleigh damping.

A visco-elasto-plastic model was employed to simulate the non-linear response of the soil, during ground seismic shaking. The model combines a visco-elastic stress-strain relationship with a non-associated Mohr-Coulomb failure criterion, both available in ABAQUS. It was employed in the analyses, as follows; initially, a series of one-dimensional (1D) equivalent linear soil response analyses were conducted, to evaluate 'mean' effective soil equivalent properties for each soil layer (i.e. degraded shear modulus and viscous damping), corresponding to a medium soil strain range. The 1D soil response analyses were performed with code EERA (Bardet et al. 2000), by implementing adequate G-γ-D curves for each layer (Pitilakis et al. 2007). The mobilized stiffness and damping were then introduced in the two-dimensional equivalent linear numerical model of the soil-structures system. At that stage the yield strength of the soil was also defined, to account for the effect of soil nonlinearity for higher soil strain levels, using the strength properties defined in Table 1. A Poisson's ratio v = 0.3 was adopted for all the examined soil layers. The viscous damping, estimated by the 1D soil response

Table 2. Selected earthquake records

# – Earthquake	Country	Date	Station	Magnitude Mw	PGA (g)
1 – Umbria-Marche	Italy	5/4/1998	Cubbio-Piene (855-Y)	4.8	0.235
2 – Montenegro	Montenegro	15-04-1979	Hercegnovi Novi (MONT T)	6.9	0.256
3 – Campano Lucano	Italy	23-11-1980	Sturno T	6.9	0.323
4 – Kozani	Greece	13/5/1995	Kozani's Perfecture	6.5	0.142
5 – Thessaloniki	Greece	20-06-1978	ABC hotel	6.2	0.143

analyses, was modelled in the form of the frequency-dependent Rayleigh type. The main angular frequency of the soil has been chosen as first control frequency, while the second was taken five times higher, that mainly corresponds to the third mode of vibration. The above soil modelling approach is commonly used in practice due to its easy calibration and has been validated against experimental results from dynamic centrifuge tests carried out on tunnel models in soft soils. (Bilotta et al. 2014, Tsinidis et al. 2015, Tsinidis et al. 2016).

The analyses were carried out in steps. Initially the gravity loads were introduced, within a static step. The seismic loading was then introduced, within an implicit dynamic step. During the initial static step, the base of the numerical model was fixed in both horizontal and vertical directions. In the subsequent 'earthquake' dynamic step, the horizontal displacement of the base was released, and the seismic shaking motions were applied at the base of the model, through appropriate dashpots, as per Lysmer & Kuhlemeyer (1969). Kinematic tie constraints were set at the side boundaries of the model, allowing for common lateral displacement patterns, throughout the analysis procedure. This boundary condition imitates the desirable 'shear beam' response of free-field soil during ground seismic shaking.

The analyses were carried out, by applying five real records (Table 2) at the base of the numerical models. To examine the sensitivity of the results on the verse of the ground motion peak amplitude, each input motion was applied twice by reversing its orientation with respect to the domain.

Additional analyses were carried out by neglecting the building and its basement. The results of these analyses were used as benchmarks for the investigation of the effects of the building on the seismic response of the station.

3 RESULTS

3.1 *Racking response and efficiency of R-F relations*

In line with the recent findings for rectangular-shaped embedded structures (Cilingir & Madabhushi 2011, Tsinidis et al. 2015), the station exhibited a coupled racking-rocking deformation pattern during the ground seismic shaking. The racking response of the station was quantified on the basis of racking ratios R computed for the individual shaking motions, as $R = \delta_{str}/\delta_{ff}$, where δ_{str} is the maximum relative horizontal displacement of the roof slab to the foundation slab and δ_{ff} is the corresponding relative horizontal displacement in 'free-field' conditions. The definition of the racking ratio is of practical importance, since it is conventionally used by the simplified *R-F* method, proposed by Wang (1993) for the seismic analysis of tunnels and embedded structures of rectangular shape. The racking ratio, R, is usually correlated with the soil-tunnel relative flexibility ratio, F, which is expressed as follows (Wang, 1993):

$$F = (G_s \times a)/(S \times b) \tag{1}$$

where: G_s is the strain compatible soil shear modulus (e.g. the shear modulus corresponding to an average effective strain of the soil during shaking at the tunnel central axis), a and b are the width and the height of the tunnel section, respectively, and S is the force required to cause a unit racking deflection of the structure. Several analytical or empirical *R-F* relations may be found in the literature (e.g. Wang 1993, Penzien 2000).

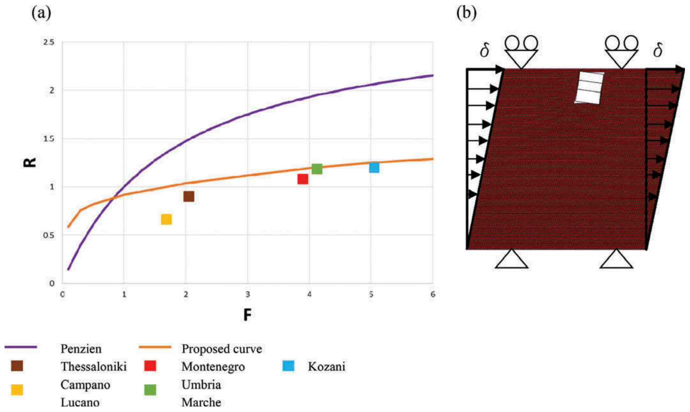

Figure 2. (a) Penzien's R-F relation (for full-slip conditions and soil's Poisson ratio v = 0.3) versus numerically predicted racking ratios, computed neglecting the building. (b) Deformed shape of the FE model of the station-soil system used to construct the new numerical *R-F* relationship.

Figure 2 compares the racking ratios computed herein by the numerical analysis for the diverse ground motions, with the analytical relation of Penzien (2000), commonly used in practice. The numerical results refer to the case where the building is neglected. The values of δ_{str} and δ_{ff} were computed at the time increment of maximum racking distortion of the station section during ground seismic shaking. It is worth mentioning that different values of flexibility ratio were computed for the station for each examined ground seismic motion, since this ratio depends on the strain-compatible shear modulus, which is affected by the ground motion characteristics.

Penzien's solution is found to overestimate significantly the racking ratio compared to the numerical results, with the differences being as high as 40 % for higher flexibility ratios, i.e. $F > 3$. Interestingly, racking ratios lower than unity are reported for $F < 2$. The above deviations and general observations should be partly attributed to the assumptions adopted by the Penzien's solution compared to the numerical analyses conducted herein. In fact, Penzien's solution is derived for pseudo-static simple shear loading of the soil-structure configuration, with the embedded structure assumed to be installed at a 'sufficient' depth in an elastic homogeneous soil deposit. Additionally, the simplified solution is developed for a full-slip interface condition with the potential separation between the lining and the surrounding ground in the normal direction being precluded. These conditions are quite different compared to those adopted in the numerical analyses. Additionally, the rocking response of the structure (see Figure 2b) is expected to bias to some extent the numerically predicted racking ratios (Tsinidis & Pitilakis, 2018).

A series of pseudo-static analyses were also conducted, implementing a station-soil model similar to one presented in Figure 2b. The model had dimensions 120 × 120 (m). The station was assumed to be embedded in elastic and homogeneous soil deposit, using a similar interaction model, as per the dynamic analyses. The base boundary of the model was fixed in both directions, while for the top boundary only the horizontal translation was allowed. An inverse triangular displacement pattern was introduced at both sides of the soil model. These boundary conditions resulted in an equivalent loading to a far field shear stress condition for the system, similar to Penzien's solution. Additional analyses were conducted, neglecting the station, so as to compute the racking response of the soil at 'free-field' conditions. The results of these analyses were used to construct a numerical *R-F* relation which is displayed in Figure 2a, compared to Penzien's solution and the dynamic analyses results. Evidently, this new numerical *R-F* relation is in a better agreement with the predictions of the dynamic analyses. In most cases it slightly over predicts the racking ratio compared to the full dynamic analyses (differences up to 25 % for lower flexibility ratios). The deviations between the numerical results should be attributed to the different loading conditions adopted in the pseudo-static analyses, which moreover do not consider non-linear and dissipative soil behaviour.

The above racking ratios were computed neglecting the presence of the nearby building with the basement. Figure 3 shows the effect of building on the numerically predicted racking

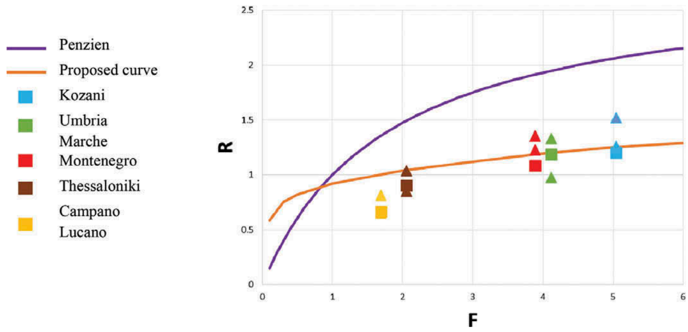

Figure 3. Comparisons of numerical racking ratios of the station computed by neglecting the building (), or accounting for the building () and using the seismic records oriented both ways.

ratios by comparing these with the results of the analyses that neglected the structure. The racking ratios computed with the presence of the building, refer to both orientations of the seismic input motion. The presence of the building seems to affect the racking response of the station, with higher racking ratios being computed for the majority of the cases when the building is considered in the analyses, particularly for higher flexibility ratios, i.e. $F > 3$. Interestingly, there are cases where the existence of the building results in a beneficial reduction of the racking ratio compared to that predicted for 'green-field' conditions. Another interesting observation is that the direction of seismic motion can affect the predicted racking motions of the station, with the differences between the two examined directions being as high as 20 %.

For lower flexibility ratios, i.e. $F < 3$, the numerical *R-F* curve, presented above, seems to lead to a safe design of the station even when the building is considered. On the contrary, for higher flexibility ratios, higher racking ratios are computed by the numerical analyses that consider the building, compared to those evaluated by the numerical *R-F* curve.

3.2 *Seismic earth pressures*

Significant earth pressures are expected to be developed on the station during seismic ground shaking. The presence of the building nearby is expected to affect the stress state around the station and therefore the earth pressures developed on its boundary. Figure 4a illustrates the dynamic increment of the earth pressures, computed at the time step of maximum racking distortion of the station, for the Kozani earthquake, considering the above ground building with the basement. Higher earth pressures increments are reported near the stiff corners of the station, as well on the upper side of right-hand side diaphragm wall, which is located nearby the basement of the building.

Figure 4b summarizes the differences between the seismic increment of the earth pressures computed on critical sections around the station when the building is considered, and those predicted at the same locations when the building is neglected. The increase of the earth pressures on right-hand side diaphragm wall of the station can be as high as 210 % near the building. Interestingly, there are some regions at the lower part of the right-hand side diaphragm wall where a reduction of the dynamic increment of the earth pressure is observed compared to the 'green-field' case. This shall potentially be attributed to the complex deformation pattern of the station during shaking (i.e. racking-rocking response, see Figure 2b) as well as on potential yielding phenomena of the surrounding ground, which alter the stress state around the station, hence leading to more complex distributions for the earth pressures (Tsinidis 2018).

3.3 *Seismic bending moment*

The building was found to alter the seismic bending moments developed on the station during ground shaking, compared to those predicted for a 'greenfield' environment. This observation is in line with previous studies (Pitilakis et al. 2014, Tsinidis 2018).

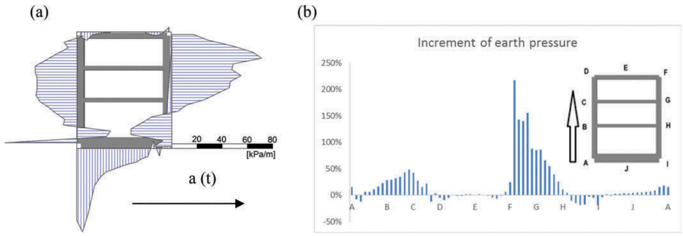

Figure 4. (a) Dynamic increment of the seismic earth pressures computed on critical sections around the station for the Kozani earthquake (PGA of the seismic input towards the building) at the time step of maximum racking distortion; (b) differences between the dynamic increments of the earth pressures computed when considering or neglecting the building.

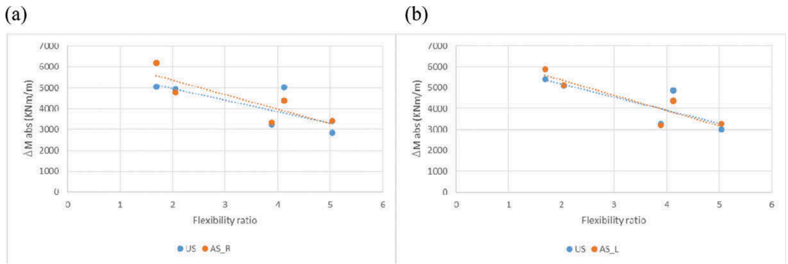

Figure 5. Effect of building and shaking motion direction on the absolute maximum dynamic increment of the seismic bending moment developed along the right hand-side diaphragm wall for various shaking motions; US: dynamic increment neglecting the building. (a) AS_R: dynamic increment accounting for the building with the PGA of the input seismic records toward the building. (b) AS_L: dynamic increment accounting for the building and using reversed (PGA towards the opposite side from the building) seismic records.

Figure 5a and b compare the maximum absolute envelope dynamic increments of the bending moment computed along the right-hand side diaphragm wall of the station during various ground motions when neglecting (US) or considering the building with the basement (AS_R and AS_L). These results are fitted by trendlines. Evidently, the computed bending moments decrease with increasing flexibility ratio, *F*, due to the higher flexibility of the station compared to the surrounding ground for these cases. Moreover, the existence of the building near the wall results in a dynamic interaction phenomenon. This is more evident in Figure 5a where the PGA of the input motion is in the direction of the building and the differences in terms of dynamic moments reaches 20 % in some cases. In Figure 5b the input motions were oriented along the opposite direction and interaction was again noticed, even if of a minor degree. The effect of the direction of the seismic input motion on the computed response of the station is also something that deserves attention.

4 CONCLUSIONS

This study examined the dynamic interaction between a multistory metro station and a building with basement, prospecting the station, when subjected to seismic excitation in the transversal direction.

Although limited to a single case study from the Thessaloniki Metro, the numerical analyses revealed that Penzien's solution is inadequate for the evaluation of the racking ratio of these types of shallow embedded structures. A numerical *R-F* relation was proposed to be used instead for large metro stations, developed on the basis of a series of pseudo-static analyses of the examined station-soil configuration. It was found to be in good agreement with the predictions of the full dynamic analyses of the examined station, when neglecting the building.

The presence of the building with the basement nearby the station affected the racking distortion of the station, as well as the seismic increments of earth pressures and bending moments along the walls. The latter effects were more evident on right-hand side diaphragm which is located aside the building. Interestingly, the orientation of the seismic input motion was found to affect the dynamic interaction phenomena.

Further research is deemed necessary to better understand and rigorously quantify the dynamic interaction effects between extended underground structures and buildings, particularly in densely urbanized areas.

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