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Soil-structure interaction analysis in a study of an offshore extension

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ABSTRACT: This paper studies the seismic non-linear soil-structure interaction effects on an offshore extension under construction in Monaco, which is a seismic area. The extension consists in backfillings enclosed by precast concrete caissons. Piled foundations are embedded in the backfillings, supporting several buildings. A non-classical design method is required to account for different phenomena such as the dynamic soil-piles-caissons-building interactions, non-linear soil (backfill) behavior and the tridimensional behavior of the extension. Due to computational limitations the calculation method is split in two parts: first bidimensional transient non-linear dynamic analyses, then a tridimensional pseudo-static analysis. The bidmensional analyses give the local soil accelerations, which are then introduced into the tridimensional pseudo-static global model. This modeling optimization allows for an efficient solution of the tridimensional dynamic non-linear soil-structure interaction problem. The soil pressure applied to the caissons obtained by the bidimensional and tridimensional analyses are finally compared.

1 INTRODUCTION

In the context of the Principality of Monaco's six-hectare offshore extension project, led by a Monegasque consortium, a seismic study has been carried out by Tractebel Engie as part of the project design team. The project area is about 400m long and 180m wide. The project includes the construction of 60 000 m^2 of residential buildings, public facilities and a car park.

The construction technique to be used for the maritime infrastructure is a fill area enclosed by a band of 18 trapezoidal reinforced concrete caissons. Each caisson is 26 meters high and weighs 10 000 tons. Figure 1 shows a 3D view of the project.

This construction is not a customary one: several buildings with different numbers of floors are founded on the piled raft foundations which are embedded in the backfills, not in-situ soil, and the substratum layer. These works are enclosed by the caissons which are very close to the buildings in some places. Furthermore, the project is located in a seismic area. That is why a non-classical design method is required to take account of various phenomena such as dynamic soil-piles-caissons-building interactions in the case of non-linear soil (backfill) behavior. In this context, both tridimensionnal (3D) and bidimensionnal (2D) calculations were performed:

- The 2D calculations allow transient analyses to be performed;
- The 3D calculation is pseudo-static and accounts for the 3D behavior of the site.

Among other results, these calculations give the soil pressure behind the caissons, which are discussed in this paper.

This paper will thus first detail the computational methods and hypotheses. Then it will describe the results regarding soil pressure behind the caissons for each of the calculations. These results allow to compare the 3D calculation with the 2D calculations and allow also to draw some conclusions about the proposed calculation methodology.

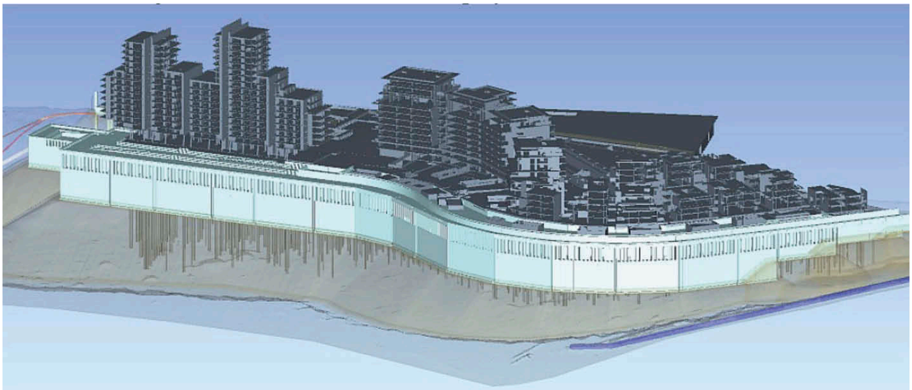

Figure 1. 3D view of the offshore extension project in Monaco.

2 COMPUTATIONAL METHODS

2.1 *2D calculations*

The 2D calculations use the Domain Reduction Method (Bielak & al. 2003), which allows significant gains in computational requirements. This method consists in performing two calculations: the first on the free-field model, the second on the near-field model. First, the free-field motion is computed through a soil column. This column is composed of the bedrock material and is large enough to cover the second model area. When performing the near-field calculation, the free-field motion given by the first calculation is used as an input on the near-field model borders. This motion is evaluated at the coordinates of the near-field model border nodes, and is introduced as equivalent effective forces. Viscous dampers on the near field model border allow to account for radiation damping and to avoid spurious reflections.

The accelerograms used as inputs for the free-field model are produced as follows:

1. Selection of recorded accelerograms from the PEER NGA strong motion database, based on the earthquake characteristics for the Monaco site (see Table 1). The retained signals are given in Table 2.
2. Accelerogram adjustment to fit the target spectrum at the top of the rock column (with RSPMatch code). The target spectrum is taken from Monegasque legislation, for a rocky

Table 1. Earthquake characteristics for the Monaco site.

Parameter	Magnitude	Distance	Soil type	PGA
Range	[M-0.3; M+0,3]	[d-10km; d+10km]		[0.5PGA; 2.0PGA]
Reference	M=6.0	d= 10km	Rock	PGA= 0.19g
Values	[5.7; 6.3]	[5km; 25km]		[0.095g; 0.38g]

Table 2. Earthquake characteristics of the selected accelerograms

No.	Earthquake	Year	Station	Magnitude [Mw]	Distance [km]	Soil type	PGA [m/s^2]
1	Whittier Narrows	1987	Mt Wilson	6	21.2	Rock	1.167
2	Friuli	1976	Tarcento	6	12	Rock	1.336
3	Valnerina	1979	Cascia	5.8	5	Rock	2.012
4	Lazio Abruzzo	1984	Atina	5.9	5	Rock	1.08
5	Umbria Marche	1997	Assissi-Stallone	6	21	Rock	1.83

outcrop. This spectrum as well as the spectra of the modified accelerograms are shown in Figure 2.

3. Baseline correction, to achieve zero displacement by the end of the seismic event.
4. Deconvolution through the rock column to obtain a signal at the base of the models.

In order to obtain relevant accelerations over the whole site for the 3D model, several 2D calculations were performed, along the planes shown on Figure 3.

When combining the results from the different accelerograms for the same section, recommendations from French nuclear legislation are applied. In particular, as an example, the retained maximum earth pressure is computed as the mean of maximum earth pressures for all five accelerograms, increased by 95% of the standard deviation of these maximum earth pressures.

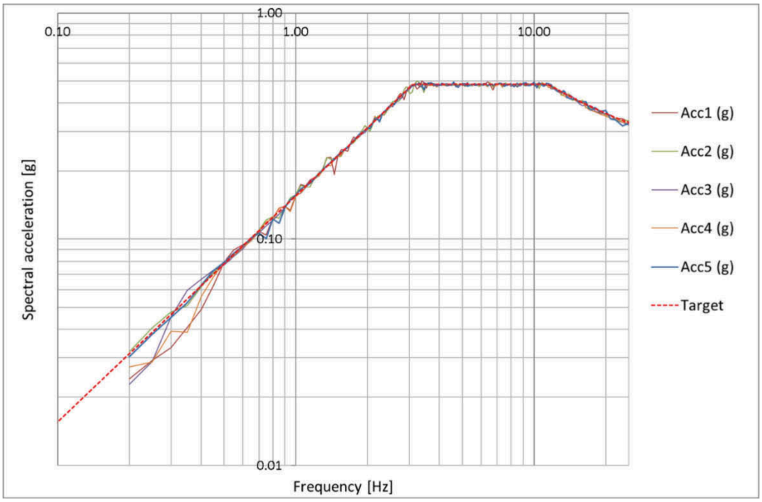

Figure 2. Target spectrum and spectra for the accelerograms produced.

Figure 3. Localization of the planes for bidimensional calculations.

These calculations are detailed more extensively in a previous paper (Taherzadeh & al. 2018).

2.2 *3D calculation*

A transient analysis of the tridimensional model is very costly to calculate. Hence a pseudo-static calculation is conducted, based on the accelerations obtained in the bidimensional calculations.

Several forces are applied within this model:

- the efforts coming from the superstructures are obtained from separate calculations and applied at the top of the piles;
- a soil acceleration field is applied to the whole model to account for the soil kinematic motion.

The acceleration applied to account for the soil kinematic motion is obtained by checking the accelerations computed in the 2D analyses for several nodes, for every computed section. A representative value is then chosen. Figure 4 illustrates this choice: for the two nodes of which the accelerograms are shown on this figure the representative value for the acceleration would be 0.15g. It is important to notice that this choice is partially arbitrary; furthermore in order to apply these accelerations in a pseudo-static model one must smooth the spatial heterogeneities observed in the 2D analyses. The retained accelerations for the different soils and structures are shown on Figure 5.

The acceleration field applied in the model is a Newmark combination, as shown in Figure 6: the retained acceleration values (see above) are applied fully in the Z direction and are applied at a 30% level in the X direction.

2.3 *Numerical modelling*

The calculations were performed using ZSoil Finite Elements code (ZACE 2013).

The 2D model is composed of two sub-models. The free-field model consists in a single rock column and allows the free-field transient analysis to be performed. Since it aims at computing the wave propagation in the substratum and must cover the second model area, this column is composed of very large meshes, respecting the criterion of the mesh size for one-dimensional

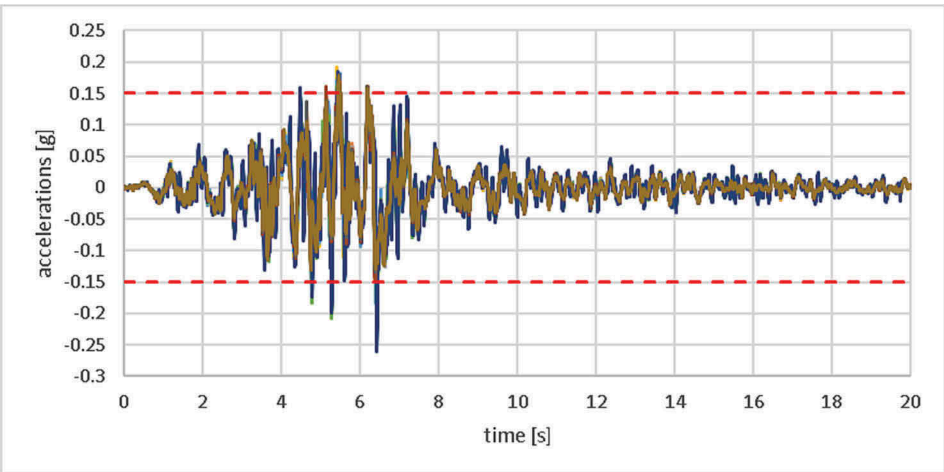

Figure 4. Examples of acceleration obtained in 2D calculations (full lines) and retained acceleration level in the 3D model for this soil (dashed lines).

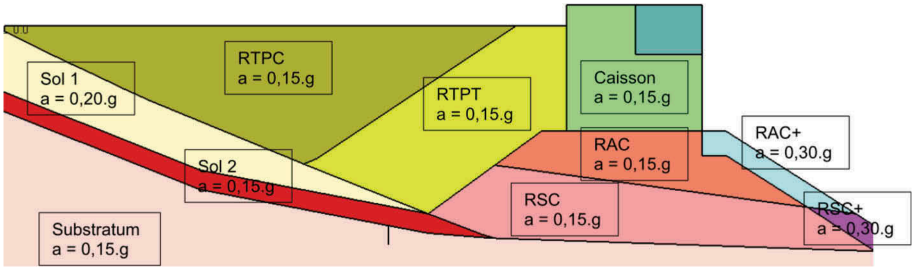

Figure 5. Retained acceleration values for the pseudo-static calculation.

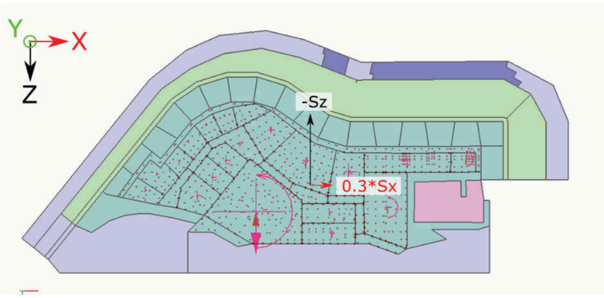

Figure 6. Kinematic load case for the 3D model and illustration of the efforts from the superstructures (inertial loading).

wave propagation in the vertical direction (at least ten nodes per minimum wavelength). The near-field model (shown on Figure 7 for the section 2 (localized in the Figure 3)) is composed of the extension with its backfills, the caissons, piles and superstructures, surrounded by the elastic substratum. This latter allows for coupling with the free-field model.

The 3D model is pseudo static. The loadings are applied one after the other. This allows the convergence of the calculation while having all the efforts applied simultaneously at the end of the calculation.

The soil and the caissons are modeled using four-node brick (resp. eight-node brick) elements with each node having two (resp. three) translational and one (resp. three) rotational degrees of freedom in the 2D model (resp. 3D model). The soil behavior is simulated by the hardening soil model (Obrzud & Truty 2014) taking into account the stress-dependent stiffness, different for loading and unloading/reloading, in the 2D calculations. The elasto-plastic

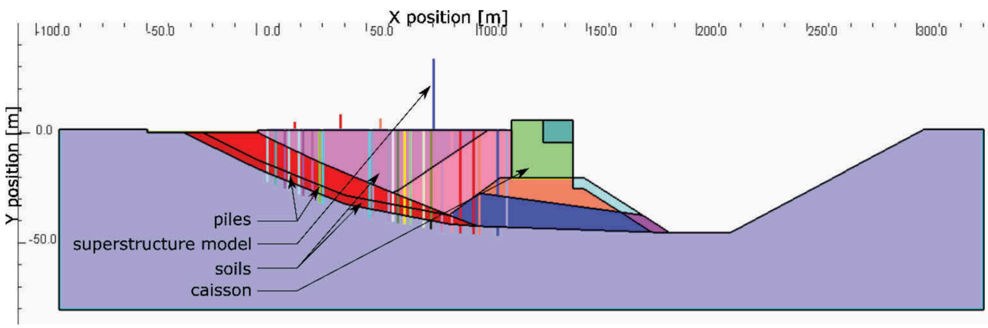

Figure 7. Section 2 of the 2D calculation.

Table 3. Geotechnical characteristics of the model.

	Backfill			In-situ soils			Substratum
	RSC	RAC	RTPC	RTPT	Soil 1	Soil 2	
Parameters							
γ_d (kN/m^3)	18	18	18	16	14,4	14,5	25
γ_{sat} (kN/m^3)	20.4	20.4	20	20.4	19	19	25
e_0	0.5	0.5	0.5	0.7	0,88	0,88	0
c' (kPa)	0	0	0	0	0	0	-
φ'(°)	48	48	33	48	33	38	-
R_f	0,9	0,9	0,9	0,9	0,9	0,9	-
E_m (MPa)	17.5	17.5	10	17.5	2.5	5.0	-
$G_{0,ref}$ (MPa)	270	270	110	270	20	50	-
ν_{ur}	0.2	0.2	0.2	0.2	0.2	0.2	0.25
$E_{0,ref}$ (MPa)	630	630	270	630	45	135	-
$E_{ur,ref}$ (MPa)	210	210	90	210	15	45	11000
m	0,5	0,5	0,5	0,5	0,5	0,5	-
$E_{50,ref}/E_{oed,ref}$ (MPa)	70	70	30	70	5	15	-
k_h (m/s)	0.1	0.1	0.001	0.1	1.0E-06	1.0E-05	1.0E-04
k_v (m/s)	0.1	0.1	0.001	0.1	2.0E-07	4.0E-06	1.0E-04
$V_{s,elas}$ (m/s)	280	280	220	280	115	180	1330

where:

- R_f – failure ratio ($=q_f/q_a$)
- $E_{ur,ref}$ – unloading/reloading stiffness at the reference stress
- $E_{50,\ ref}$ – secant modulus corresponding at 50% of q_f at the reference stress
- $V_{s,elas}$ – shear wave velocity

behavior provides the hysteretic damping of the soil. In the 3D model the soil behavior is accounted for with a Mohr-Coulomb model. The soil properties are presented in Table 3.

The piles are modeled by beam elements. The Young's modulus of the piles is set at 35 GPa. Their diameters vary between 1 m and 1.8 m and their length is up to 47 m. They are embedded in the substratum. Their behavior is assumed to be linear.

It is assumed that the soil and piles are not perfectly bonded. Thus, sliding between the soil and pile is considered. The latter is provided by Mohr-Coulomb criteria. The same behavior governs the contacts between the caissons in the 3D model, as well as between the soil and the caisson (except for the base of the caissons in the 3D model).

The superstructure is modeled on the basis of the structural calculations, which give the first natural frequency and the forces at the top of the foundations. Hence these structures are modelled by a mass-spring system calibrated by the first mode in the 2D model, and by the forces they induce on top of the piles for the relevant seismic combination in the 3D structural model. Some elements (beams in the 2D model, shells in the 3D model) allow the top of the piles to be linked together under a single superstructure.

3 RESULTS COMPARISON

The results of both the 3D and the 2D calculations in terms of earth pressure are shown in Table 4. The results shown for the 3D calculation in this table are obtained in the area relevant for each section it is compared with.

The greater values in the 3D model for the static efforts are likely due to ignoring the sliding effect at the base of caissons in this model, which is not the case in the 2D one. Hence, the 3D calculation leads to limit the relative displacement between the caisson and the backfill soil behind, consequently producing the lateral earth pressure closer to the at rest one.

Table 4. Earth pressure behind the caissons following the computational method.

Section	Earth pressure behind the caisson	2D model	3D model	Difference*
1	Static force (before earthquake) [kN/ml]	471	562	+19%
	Minimum force (during earthquake) [kN/ml]	265	420	
	Maximum force (during earthquake) [kN/ml]	2746		
2	Static force (before earthquake) [kN/ml]	526	531	+1%
	Minimum force (during earthquake) [kN/ml]	362	440	
	Maximum force (during earthquake) [kN/ml]	1443		
3	Static force (before earthquake) [kN/ml]	470	641	+36%
	Minimum force (during earthquake) [kN/ml]	264	644	
	Maximum force (during earthquake) [kN/ml]	1520		

* 3D relatively to 2D

In the 2D model, the transient analysis allows to account for the same or opposite phase motions of the caisson and the backfill soil behind. These differences of phase in the motion, and the differences of natural frequencies, are due to the massive rigid caissons having dynamic characteristics quite different of the backfill soil ones. Accounting for these different motions leads to:

- low values of the earth pressure in case of motions in opposite directions of the caissons and the backfill soil behind, when the two bodies are moving apart;
- high values of the earth pressure (passive state of the soil) in case of motions in opposite directions, when the two bodies are moving towards each other;
- intermediate values which include the case of the caissons and the backfill soil moving in the same direction.

The results obtained in the 3D model are within the range of the values obtained in the 2D model, and closer to the low values. Two phenomena may be involved in this result:

- the caissons and backfill soils are submitted to the same acceleration, but the soil is partly retained by the piled foundations, so it is less likely to move than the caisson.
- the soil under the caisson, on the sea side, is slightly more accelerated, leading to a lesser confinement of the soil right under the caissons. This leads to a rotation of the caissons, as shown in Figure 8.

These two phenomena lead to a more active state of the soil, resulting in a lower earth pressure in the dynamic case than in the static case. These statements are less relevant for the caissons in the section 3 area, given the acceleration direction and a geometry which is more likely to involve significant 3D effects.

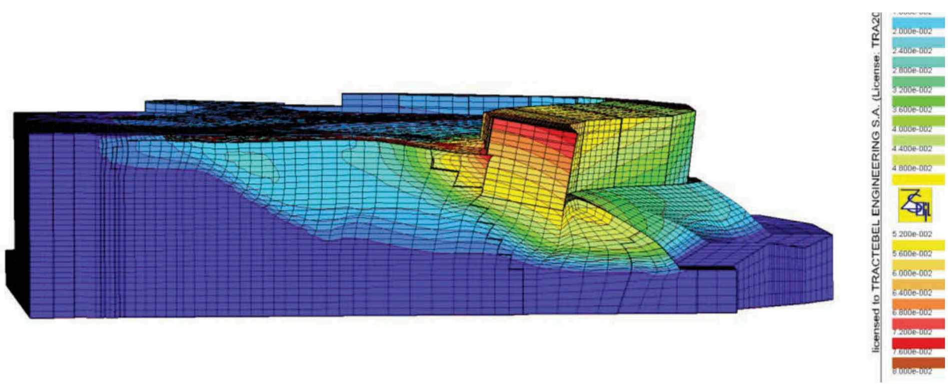

Figure 8. Absolute displacements in the 3D model under dynamic loads - section 1 area.

4 CONCLUSION

This paper studies the dynamic behavior of an offshore extension area under construction in the seismic area in Monaco. This construction of several buildings with various numbers of floors founded on piled raft foundations embedded by the backfills and all surrounded by a series of pre-cast caissons is not a customary one. This is why a non-classical method of calculation is used for the seismic analysis. In this context, two methods of computation are performed and combined: the first is a set of 2D dynamic transient analysis crossing several sections of the project and the second is a 3D pseudo-static analysis to take account of geometrical effects. The latter is excited by the acceleration field obtained by the 2D dynamic analysis. All these models take account of the presence of the piles and the superstructures for both kinematic and inertial effects.

The 2D dynamic results give evidence that the massive rigid caisson movement can be in the same phase and the opposite phase against the backfill soil behind, leading to production of a wide range of earth pressures varying from the active soil state to the passive soil state. The 3D pseudo-static model gives the earth pressure close to the low values of the 2D dynamic analysis due to the fact the both backfill soil and the caissons move in the same direction. It is important however to stress that a single 3D calculation does not allow to account for the seismic load in every direction and requires some approximations. Hence the proposed methodology should be completed with more pseudo-static calculations to cover more extensively the different seismic cases.

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