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COUPLED SOIL-STRUCTURE INTERACTION ANALYSIS AT THE PORT OF NADOR WEST MED

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Abstract: *This paper discusses the coupled soil-structure interaction analysis, results, and conclusions from the design of the Port of Nador being developed by the Moroccan Agency NWM. Located on the north coast of Morocco, the construction of the new industrial mega-port comprises over 5km of breakwaters and 3km of quayside to form two container terminals, petro-chemical berths, and solid fuel terminals. The design of the port has provided an opportunity to study post-earthquake performance of port structures including the onset and effects of liquefaction using advanced constitutive models. A performance-based design approach was adopted for the composite concrete caisson, nearshore and deep-water rubble mound breakwaters.*

The seismic design strategy included a two-staged approach to accelerate the design process. Initially an uncoupled pseudo-static analysis was performed on the structural model. The second stage included a coupled dynamic soil structure interaction analysis. Post-earthquake displacements and accelerations were compared to the conventional pseudo-static and response spectrum analyses and used to verify the performance-based design of the composite caisson-rubble mound structures.

The application of dynamic coupled soil-structure interaction analysis in place of conventional pseudo-static methods and the performance-based design approach resulted in an optimised and compliant solution with significant cost savings of the order of tens of millions of dollars in terms of seabed ground treatment, rock quantities and concrete reinforcement quantities.

Introduction

The new industrial mega-port Nador West Med is currently under construction on Morocco's Mediterranean coast (Figure 1). Nador West Med will comprise two container terminals, petro-chemical terminals and solid fuel terminals formed by 2.1km of caisson quays, 500m of diaphragm walls protected by over 5km of breakwater constructed in water depths up to 35m. There will be a total of 23 million cubic metres of land side enabling earthworks, 17 million cubic metres of dredging and ground improvement and 253 caissons weighing up to 9000 tonnes each. Ramboll was appointed to provide detailed design services for the design & build contracting joint venture STFA SGTM Jan de Nuul (SSN) on the marine civils and dredging works.

The project presented a number of technical challenges that required an innovative approach to design, not least due to the volume of material required, regional seismicity, and the soft soil site characteristics where strength mobilisation effects dominate. It was therefore important to examine the post-earthquake performance of the caissons and rubble mound on which they are founded.

Conventionally, pseudo-static or linear dynamic analysis methods are applied to structures with a fixed base, where the behaviour of the foundation system and soil-structure interaction is well defined. These conventional methods can lead to overly conservative designs, or structures with poor seismic performance, particularly when the seismic behaviour is dominated simultaneously by the structural and geotechnical performance of the composite system. The main breakwater at Nador West Med is an example of such composite system, which is formed by concrete caissons founded on deep-water rubble mounds overlying backfilled dredged pockets or

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improved natural soils. Typical failure mechanisms for breakwater structures include failure of the rubble mound slopes, global stability failure, total and differential displacements.

Figure 1. The Port of Nador project site during the initial phase of the project, looking north with the Mediterranean Sea in the background.

This paper presents an overview of the seismic design approach of the main Caisson Breakwaters and includes the assessment of liquefaction and the ground-improvement strategy undertaken. The commercially-available finite element package Plaxis (2017a) is used to analyse the soil-structure-interaction, model soil nonlinearity and onset of liquefaction. Behaviour of the caisson-rubble mound structure is assessed during the earthquake events and post-earthquake performance is validated using pre-defined performance criteria.

Site characteristics

Nador West Med is located east of the Riff region, a mountainous region in the northern part of Morocco, and to the south of the Strait of Gibraltar, and south-east of the Iberian Peninsula. Regional seismicity is governed by a series of active faults resulting from the convergence between the African and Eurasian plates, which extend for more than 1500 km east-west from Portugal to Tunisia and 1000 km north-south from France to south of Morocco. Three major faults 10km to 30km in length are located within 50km of the site: the Nekor Fault to the north, which has a strike-slip focal mechanism, and the Ajdir and Trougout normal faults to the east from Van der Woerd et al. (2013).

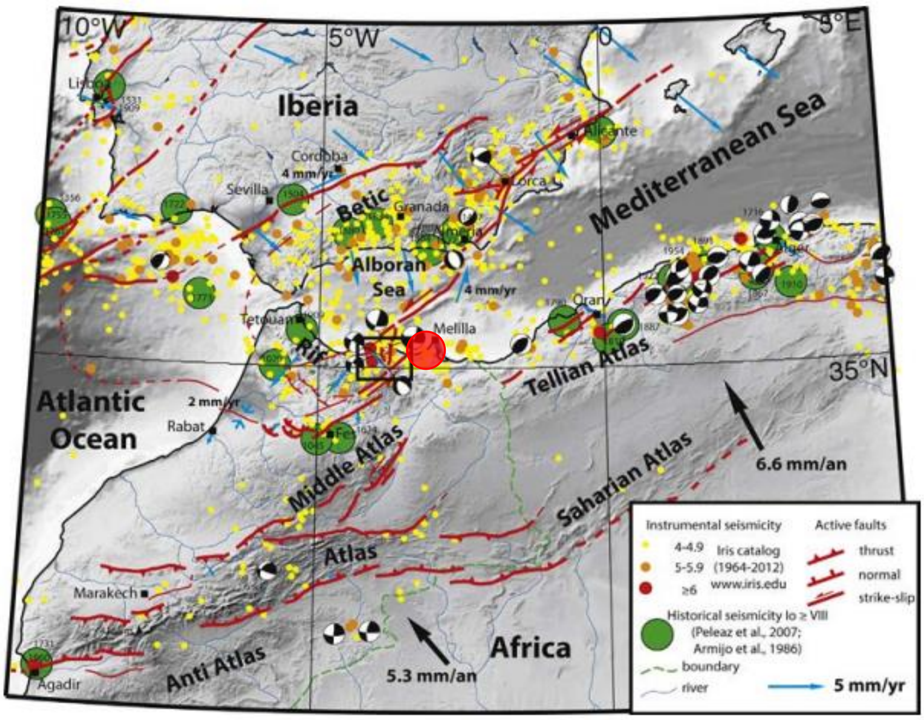

Figure 2. Seismotectonic map of western Mediterranean showing active structures of Africa–Iberia collision zone from Van der Woerd et al. (2013). Nador West Med site location in red.

A probabilistic seismic hazard study was undertaken for the project. In accordance with local codes three levels of seismicity were calculated and used to derive synthetic time-history records:

- Level N1 **-** PGA of 0.13g for return period of 475 years - equivalent to a 10% probability of exceedance in 50 years,
- Level N2 - PGA of 0.19g for return period of 975 years - equivalent to a 5% probability of exceedance in 50 years, and
- Level N3 - PGA of 0.26g for return period of 1975 years - equivalent to a 2.5% probability of exceedance in 50 years.

These design accelerations result in significant seismicity for higher Importance Class structures (i.e., port structures and petroleum berths), particularly where there is high risk of liquefaction.

The resultant 18 time-histories (comprising horizontal, vertical, North-South and East-West components) have a minimum duration of 18 seconds. Pre-processing of the signals included baseline correction to prevent drift.

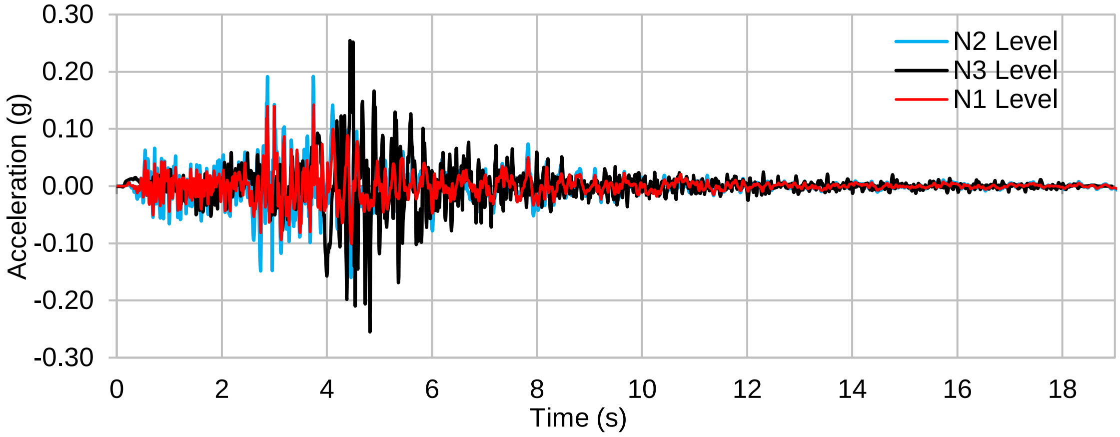

Figure 3. Employed time-histories for the breakwater caisson dynamic analysis.

Stratigraphy and ground conditions

Ground conditions at Nador West Med comprised marine deposits of silts and sands overlying Marl and Volcanic Tuff. A significant thickness of low plasticity silt was encountered under the area of the breakwaters. The site has a Site Class S2 and a $V_{s,30}$ of 176m/s. Shear wave velocities of the soil deposits range from 250 to 600m/s, as shown in Figure 4. The topography of the seabed dips rapidly to the north as the container terminals and breakwater extend offshore. The generalised stratigraphy is given in Table 1 and the soil profile is given in Table 2.

Stratum	Depth from (m)	Depth to (m)	Description
Silt	0	4.0	Soft organic silty clay/clayey silt (low plasticity)
Loose Shelly Sand	4.0	19.7	Fine medium grained sand with occasional medium to coarse grained layers
Dense Sand	19.7	20.8	Dense sand with occasional gravel and flint layers
Green Marl	20.8	22.0	Greenish in colour, and comprises predominantly silt and clay particles, occasionally interbedded with layers of sand and Volcanic Tuff.
Grey Marl	22.0	64.0	Greyish in colour, comprising predominantly silt and clay particles.

Table 1. Generalised stratigraphy.

	Rubble Mound	Vibro-compacted Sand	Loose Shelly Sand	Dense Sand	Green Marl	Grey Marl
γ (kN/m^3)	18	20	18	19	20.5	20.5
$φ_{pk}$ (°)	42	40	31	35	22.5	35
G_0 (MPa)	194	162	132	202	423	1,000
V_s (m/s)	325	310	269	326	400	600
where, γ: unit weight φ: angle of internal friction				G_0 : Small strain shear modulus V_s : Shear wave velocity		

Table 2. Key geotechnical parameters adopted for the analysis of the caisson breakwater.

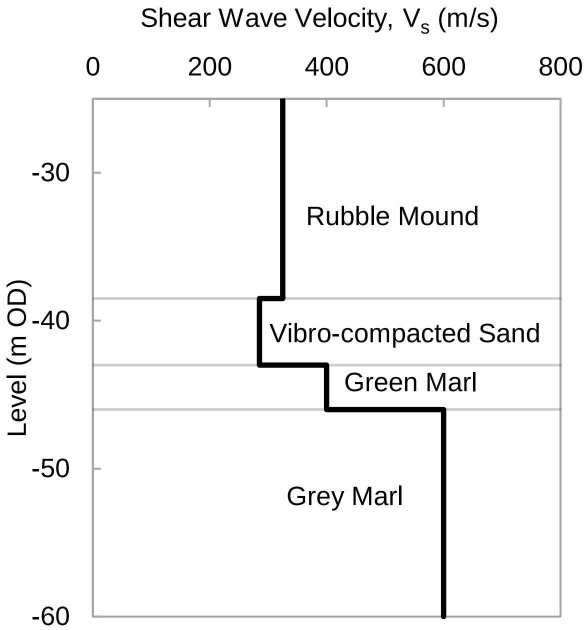

Figure 4. Shear wave velocity profile considered for the analysis.

Dynamic soil behaviour is modelled using the hardening soil model with small strain stiffness overlay. The model introduces small strain shear modulus G_0 and a reference shear strain $γ_{0.7}$ at which the secant shear modulus is reduced to 70% of G_0 to capture the nonlinear degradation of soil stiffness. Strain dependent characteristics of the soils are adopted from shear modulus degradation and damping curves by Idriss (1990) for the sands, Menq et al. (2003) for the rubble mound breakwater structures and EPRI (1993) for the marls and tuff. Nonlinearity of the EPRI curves reduces with depth and have been selected based on description and depth of the soil layer. The employed degradation and damping curves are found in Figure 5. These curves inform the quantities of G_0 and $γ_{0.7}$ that are implemented in Plaxis (2017a) which enabled the modelling of hysteretic damping behaviour of the soil.

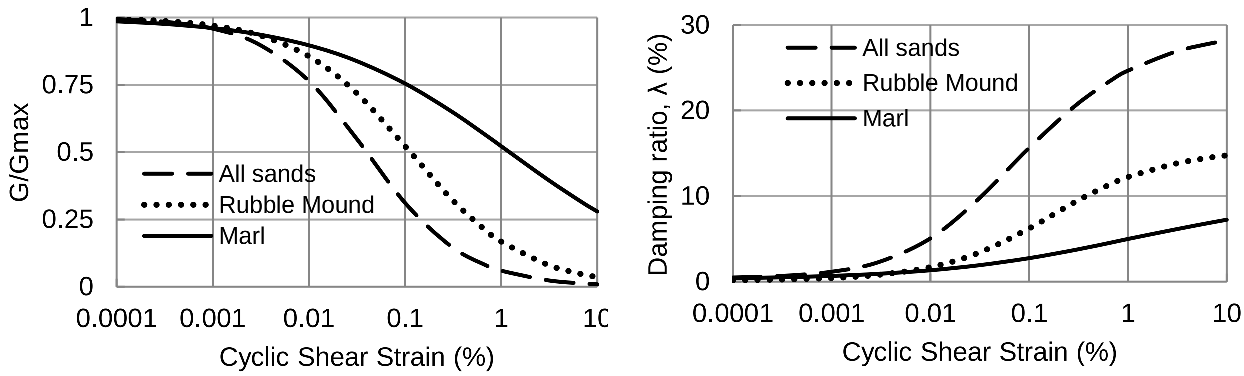

Figure 5. Normalised shear stiffness and Damping ratio curves employed.

Performance-based seismic design

Performance based seismic design approaches are becoming more commonly adopted for projects and integrated with modern design codes. This often leads to more effective and economic design solutions, creating benefits in terms of cost, reduction of material quantities and carbon footprint. In this example the governing performance criteria are derived in terms of the acceptable extent of structural and operational damage and are based on guidance given by the PIANC WG34 Seismic Design Guidelines for Port Structures (PIANC, 2001). Three levels of damage are considered:

- Degree I: No Damage (no structural damage, no loss of serviceability)
- Degree II: Repairable Damage (short-term loss of serviceability)
- Degree III: Near Collapse (excessive damage, long term loss of serviceability)

The performance criteria and degree of damage are assigned to specific port features based on their designated grade. At Nador West Med the petroleum berths are classified as *Grade S*, meaning that under a lower level of seismic loading (in broad terms a serviceability limit state) and under a higher level of seismic loading (representing in broad terms an ultimate limit state) there must be no damage (Degree I). The rest of the port features such as the main Caisson Breakwaters and Container Terminals are classified as *Grade A*, meaning that under a lower level of seismic loading there must be no damage, however some repairable damage may be accepted under a higher level of seismic loading.

With three levels of seismic loading and criteria for a lower and higher seismic event in terms of damage from PIANC (2001), the assignment of damage criteria becomes a key consideration in design which has a high economic impact on a large-scale infrastructure project such as the Nador West Med. On a project specific basis therefore, for the *Grade A* and N3 level seismic loading the performance criteria are considered as of Degree III (near collapse). For the N2 seismic loading the performance criteria are considered as of Degree II (repairable damage). The '*near collapse*' damage criteria are representative of an extended repairable damage where in any case there is no collapse. Due to the structural form and load transfer mechanism of the caissons it is largely expected that even for the N3 seismic loading, any potential local damage will be repairable. The Grade S petroleum berths require more stringent criteria and the N3 level of seismic loading is assigned to a repairable damage (Degree II), leaving the N1 and N2 levels of seismic loading to be assigned to a 'no damage' seismic performance. It has to be noted that specifically for the project, only vertical residual post-earthquake displacement has been allowed at N3 (under the repairable damage performance) for the petroleum berths, which generally and from a high level perspective will not cause any damage or affect directly the operability of the port. Due to the arrangement of the caissons (petroleum berth caissons extend transversely from the main breakwater caissons) any potential horizontal displacement and/or tilting of the caissons will not adversely affect the operation of the port if pounding effects take place between caissons. However, after an N3 event any services that connect the petroleum berths with the main breakwater will need to be reinstated. Therefore, a specific displacement limit for residual vertical displacement was introduced for the project.

The performance-based design criteria and their quantification for the Nador West Med caissons are listed in Table 3. The post-earthquake performance of the structures is based on the differential and permanent deformations of the soil that are considered acceptable at the site.

Port Features	**Component**	**Criteria**	**Degree I** No damage	**Degree II** Repairable damage
Breakwater formed by caissons on rubble mound	Gravity Wall	Residual horizontal displ. d/H	***N1 level*** < 1.5% or 30cm	***N2 level*** < 1.5-5%
		Residual vertical displ.	15 cm	25 cm
		Residual tilting towards sea	< 3 degrees	< 3-5 degrees
Petroleum berths formed by caissons on rubble mound	Gravity Wall	Residual horizontal displ. d/H	***N1 and N2 level*** < 1.5% or 30cm	***N3 level*** Not applicable for the project
		Residual vertical displ.	15 cm	25cm
		Residual tilting towards sea	< 3 degrees	Not applicable for the project
Container terminal quay formed by	Gravity Wall	Residual horizontal displ. d/H	***N1 level*** < 1.5% or 30cm	***N2 level*** < 1.5-5%
		Residual vertical displ.	10 cm	20 cm

retaining caissons on rubble mound		Residual tilting towards sea	< 3 degrees	< 3-5 degrees
	Apron	Differential settlement behind caisson	< 0.03-0.1m	Not quantified criteria
		Differential settlement between apron and non-apron areas	< 0.3-0.7m	Not quantified criteria

Table 3. Damage criteria and seismic loading level for key port structures.

Seismic design approach

The seismic design captured the soil-structure interaction (SSI) and satisfied the structural and geotechnical limit state design checks. Structurally, the critical elements are the slip-formed precast reinforced concrete caissons. Geotechnically, the critical design limit states are shallow and the global failure of the rubble mound, and bearing and overturning of the caissons. Examples are given herein for the Caisson Breakwater arrangement which are founded on a deep-water rubble mound constructed on a dredged pocket, which is backfilled with vibro-compacted sand.

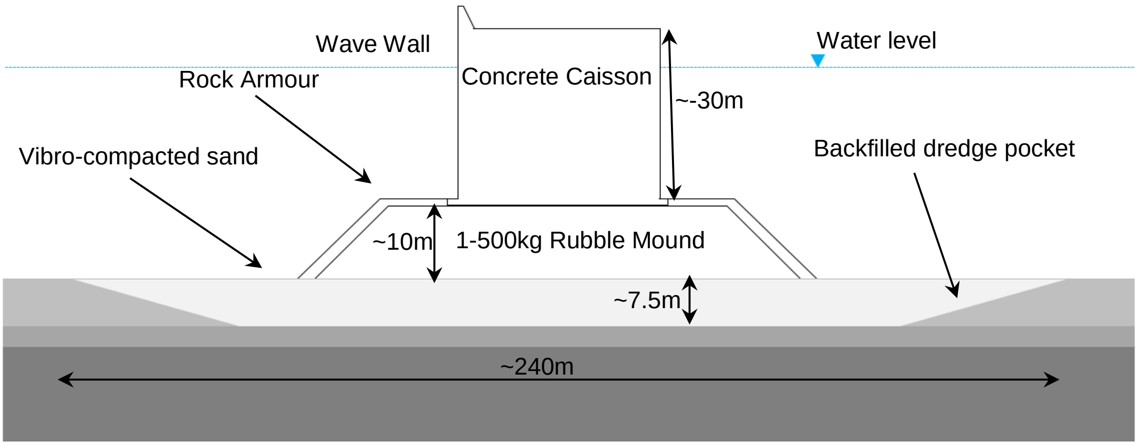

Figure 6. Particular Caisson Breakwater section.

The seismic design approach addressed structural and geotechnical considerations simultaneously, combining parameters for use in multiple analysis programs. Figure 7 outlines the process adopted for the Caisson Breakwater which eliminated multiple design iterations.

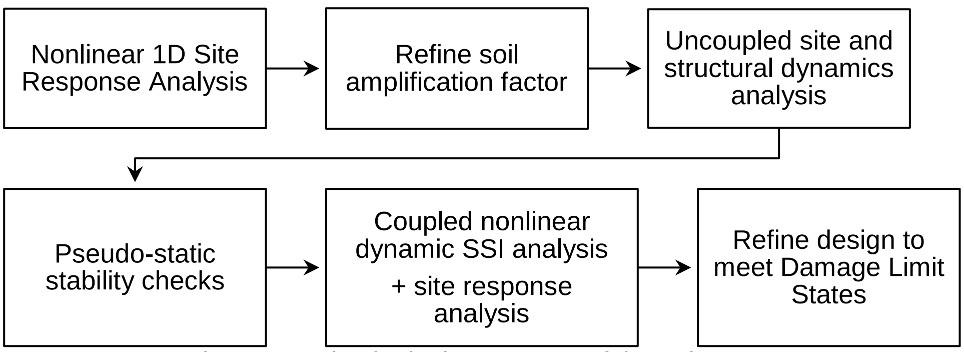

Figure 7. Seismic design process of the caissons.

The design process was initiated with a one-dimensional non-linear site response analysis. This was an important preliminary step as it provided a benchmark for an optimised soil amplification factors S_a for the site-specific seismic design spectra which were used in pseudo-static seismic design. A simplified uncoupled dynamic analysis of the site and structure was undertaken in Sofistik (2014) for the structural design and preliminary geotechnical review of the caisson concrete base bearing pressures. The SSI was simulated by modelling the elastic bedding below the caissons (i.e. the rubble mound formation) as springs based on a mobilised modulus of subgrade reaction. The structural model and the acceleration results after calibration with the coupled analysis are shown in Figure 8.

Pseudo-static analyses were then carried out in Plaxis (2017a) to assess the stability of caissons prior to dynamic analyses. Following this, the dynamic coupled SSI were carried out. The three levels of seismic time-histories were applied at the base of the model which is extended within the Marl layer. Advanced constitutive soil models were applied to capture nonlinearity. At the same time, a numerical simulation of the propagation of vertically incident seismic waves in horizontal soil layers was performed to capture attenuation or amplification of the waves through the soils. The initial response spectrum analysis results were used to validate results. Resulting post-earthquake displacements formed the basis of the reinforced concrete, ground improvement, and deep-water rubble mound design.

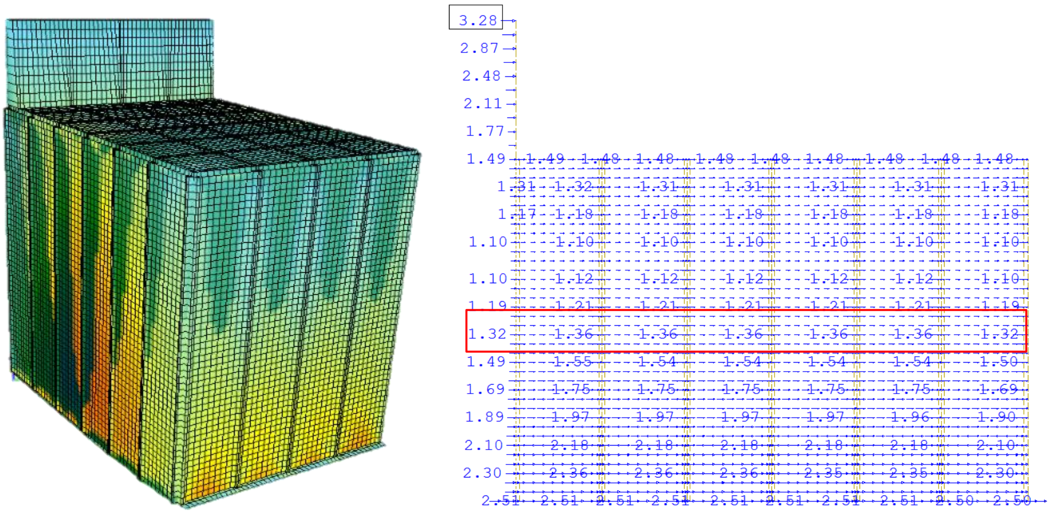

Figure 8. Sofistik (2014) isometric views of the caisson breakwater model and calibrated accelerations with coupled analysis (in red).

Uncoupled SSI - analysis details

A response spectrum analysis as per the provisions of BS EN 1998-1 and BS EN 1998-2 was deemed to be appropriate for structural design. It has to be noted that caissons are special structures and there are limited specific structural design rules for the precast concrete design elements. As a result, during the detailed design phase a set of structural criteria were developed mainly referring to the design criteria of reinforced concrete bridges. The structure was subject to the free surface excitations at the caisson concrete base level. The key features of this model were as follows:

- Only the caisson structure is included in the model
- Pairs of modulus of subgrade reaction (K_s) values were extracted at the foundation level of the caissons from an initial geotechnical finite element model and applied as springs in the structural model.

Coupled SSI - analysis details

Pseudo-static and response spectrum analyses make simplifying assumptions by separating the coupled interaction problem into an isolated free-field soil and structure problem. However, in the nearfield, the vibration of structures and supporting earthworks may modify the free-field ground accelerations and hence the resulting input motion to the structure at foundation level. By separating the coupled interaction problem into an isolated free-field soil and structure problem the interaction is ignored. This problem is overcome through the coupled soil-structure-interaction analysis which models the dynamic SSI. The geometry of the coupled system is presented in Figure 9. The key features of this model were as follows:

- The soil layers and caisson structure are both included in the model
- The caisson is connected to the soil mesh using horizontal and vertical springs
- The model was subjected to the seismic input motions at the base of the model which extends within the Marl layer and were attached to absorbent boundaries. It was ensured in the numerical model that only the upward propagating waves are modelled.

- Compatibility between the input motions and numerically generated free-field output motions were verified through studies in Deepsoil (2016).
- The site is subjected to a Site Response Analysis with lateral boundaries of the model as free-field absorbent to allow development of near-field effects
- The Hardening Soil Small Strain material models were used to capture stress-dependent stiffness development and small strain stiffness features and hysteretic damping
- The UBC3D-PLM in Plaxis (2017) soil model is implemented to capture the onset of liquefaction in the model
- An additional soil element on the face of the caisson accounted for the added mass due to hydrodynamic loading
- The presence of water and associated water pressures along the surface of the caisson and rubble mound were considered in the model.

The most onerous analysis showed a residual horizontal movement in the order of 20cm, within the limits of the associated degree of damage. A typical horizontal displacement time-history is shown in Figure 9. Peak displacement demands calculated by the uncoupled nonlinear dynamic analysis were found to be similar. The uncoupled approach validated the coupled SSI analysis and it was concluded a valid option in preliminary design stages.

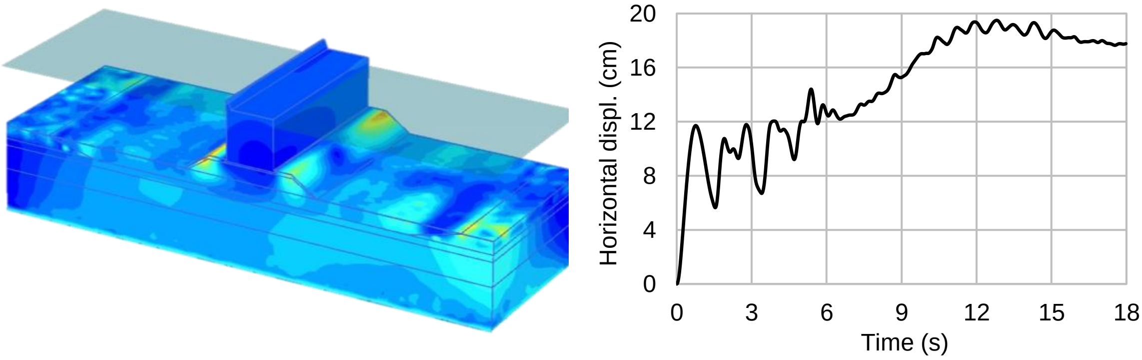

Figure 9. Geometry of the coupled system model in Plaxis and horizontal displacements under N2 seismic loading

Optimized design for liquefaction risk

Nador West Med is in the Bay of Bettoya which is characterised by significant depths of superficial deposits that have a high potential for liquefaction, in particularly the shallow marine deposits.

A significant risk of liquefaction over large parts of the site had to be managed to ensure stability of the new port structures. Several previous designs proposed the complete removal of the soils under the structures to remove the risk of liquefaction. Given the scale of the project, the cost of this would exceed $30,000,000 (cost figure for 2017) and would be challenging in water depths of up to 35 metres. Therefore, structures were designed to be stable under the three levels of seismic events and during liquefaction. The ground improvement strategy was designed specific to each structure to meet the performance-based design criteria. Additionally, acceleration and soil amplification factors were refined through a one-dimensional site response analyses to aid the assessment of liquefaction risk. This strategic approach resulted in reduction of ground improvement requirements from an average of 13m depth to a maximum of 6.5m depth, and no ground improvement requirements for large parts of the site, saving the Contractor approximately 2.2million m^3 in vibro-compacted fill.

An initial study of mobilised shear strength parameters from site response analyses proved overly-conservative prediction of liquefaction risk as the mobilised strengths were based on the assumption that entire soil layers liquefy. This was not practical to take into consideration the variability in the underlying deposits. Therefore, a user-defined constitutive model for sands was created and implemented in Plaxis (2017a) to assess the potential for liquefaction. This gave confidence that the breakwaters would remain stable in the event of liquefaction in a portion of the underlying ground.

Modelling the onset of liquefaction

The user-defined constitutive model UBC3D-PLM is implemented in Plaxis (2017a) to assess the evolution of pore pressures in sandy soils and fill material. Nonlinear effects i.e., cyclic-softening of stiffness, are also captured by the UBC3D-PLM. Liquefaction potential is assessed by means of the excess pore pressure ratio (r_u). The onset of liquefaction and cyclic-softening of the replacement fill and in-situ sands were assessed. The key parameter adopted for the UBC3D-PLM material include a normalised *SPT N* value i.e., $(N_1)_{60}$, which separates the effects of soil relative density and effective confining stress. Liquefaction potential is expressed by means of the excess pore pressure ratio (r_u):

$$r_u = 1 - \frac{\sigma'_v}{\sigma'_{vo}} \tag{1}$$

where σ'_v is the effective stress at the end of the dynamic motion and σ'_{vo} is the initial vertical effective stress. Where $r_u > 0.7$ the layer is considered to be liquefied (Beaty and Perlea, 2011).

The vibro-compacted fill layer under some of the structures including the Main Breakwater showed signs of the onset of localised liquefaction when subjected to a N2 and N3 level earthquake. The build-up of pore-water pressures is demonstrated in Figure 10 for the Caisson Breakwater undergoing an N2 Level earthquake. This structure is designed with the soil layers possibly liquefying, while residual deformations of such structures are within the limits to comply with the performance-based design criteria.

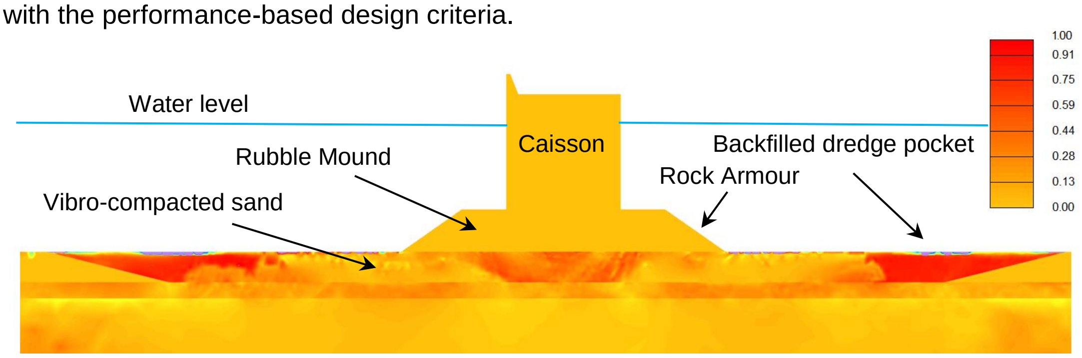

Figure 10. Pore water pressure ratio (r_u) contours for N2 Level earthquake.

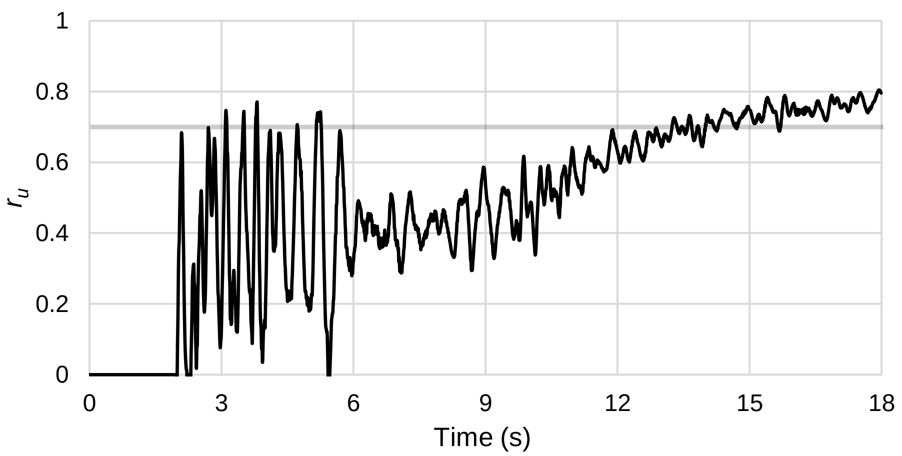

Figure 11. Pore-water pressure ratio (r_u) build-up in UBC3D-PLM materials.

Coupled SSI - a sustainable approach

Coupled SSI aided the structural design of the caissons by estimating more realistic spring values and incorporating the nearfield seismic effects into the models. The use of a coupled SSI approach enabled the risk of liquefaction to be incorporated rather than completely eliminated or assessed separately, providing a more economical design. Modelling the soil in the analysis developed greater confidence and accuracy in the calculation of post-earthquake displacement and soil amplification factors in relation to the uncoupled analysis where soil amplification factors are typically obtained directly from design codes. To ensure that the coupled and uncoupled sets of analysis were compatible, the accelerations of the caisson were compared. Very good correlation was found and therefore the uncoupled analysis was validated for the extraction of results for structural design. The application of non-linear Hardening Soil Strain (HSsmall) soil models over Mohr-Coulomb soil models in the coupled SSI approach proved to be slightly over-predicting ground movements, validating the use of such constitutive soil models to capture stress-dependent stiffness behaviour.

Confidence gained in the uncoupled analysis through calibration with the coupled analysis enabled the steel reinforcement of the caisson to be optimised by approximately 10%. In addition, ground improvement requirements including dredging, replacement and vibro-compaction were reduced through the implementation of coupled soil-structure interaction analysis rather than separated pseudo-static and structural problems. Providing savings of approximately 500,000 m^3 of quarried 1-500kg rock fill for the main Caisson Breakwaters alone.

Conclusions

The Nador West Med case study shows that coupled SSI can provide considerable benefit to projects. It can ensure accuracy and confidence in the quantification of post-earthquake performance criteria for performance-based seismic design and at the same time it is cost-effective in terms of structural and geotechnical design quantities. This makes the rather complicated dynamic coupled SSI analyses method very attractive for large scale infrastructure projects where large quantities of material may be required. For the specific project the overall seismic design strategy achieved approximately 10% of savings in steel reinforcement for the concrete caissons (Figure 11), 500,000m^3 saving of breakwater quarry fill and over 2,200,000m^3 of ground improvement, when compared with more conventional seismic analysis and design methods.

Figure 11. Construction of breakwater caissons under way.

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