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Seismic retrofitting optimization of underground parking garage using an efficient nonlinear-analysis based design strategy

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ABSTRACT: This paper presents the design strategy for the seismic retrofitting of a parking garage aimed at the optimization of the strengthening measures. The design strategy considers the execution of nonlinear time-history analyses (NLTH) including the soil and the foundation to realistically determine the demand. To significantly reduce the calculation time, a combination of PLAXIS and OpenSees models is used. The PLAXIS model helps to verify that the assumptions made in the OpenSees model are appropriate for a selected number of cases. Once the OpenSees model is satisfactory, it is updated to consider more realistic structural behavior and further used to compute the NLTH drift demand. Ultimately, thousands of nonlinear pushover analyses (NLPO) are performed in order to optimize the design considering the drift demand from the NLTH analyses as target. Material strengthening measures and calculation time were reduced significantly by applying the proposed strategy.

1 INTRODUCTION

When an existing structure is assessed regarding earthquakes, considering updated hazards and standards, it is in the interest of the owner of the structure to find the ultimate limit of the structural capacity and the required retrofitting measures in order to save costs. By using nonlinear time-history (NLTH) finite-element analyses, a more accurate description of the loading and behavior of the materials can be achieved, increasing the potential to reach this goal. In the case of an underground structure, soil should be included in the analyses, because the soil-structure interactions effects are important. Despite such a model can be built, the total necessary calculation time to consider several set of motions, sensitivity in soil strength, existing structure evaluation, alternative strengthening measures and design optimization will most likely exceed practical engineering time frames.

The underground parking garage Boterdiep in Groningen, with an area of 290x120 m² is the largest single story parking garage in the Netherlands. Due to induced earthquakes, which are occurring frequently since its construction was completed, seismic retrofitting is required. The assessment is performed on the shortest cross-section of the garage, which has dimensions 100x120 m² and has 221 columns. The main vertical bearing components are the concrete columns and the piled foundation. The lateral support is provided by the soil, the sheet pile walls, the columns, the concrete shear walls and the piles. There is also a building on top of the garage placed on base isolation to reduce the lateral loads to the structure and the garage. From a preliminary design phase, it is already determined that the columns require strengthening, mainly because their lateral displacement capacity is only 12 mm.

The goal of the project was to optimize the strengthening measurements needed to increase the lateral displacement capacity of the columns. Considering a single earthquake

motion, a total of 5100 analyses are required given the large amount of variable column properties: 13 geometries, 11 vertical loads, 2 orientations and 15 configurations of FRP-layers used as strengthening measure. On top of this, the Dutch code NPR 9998 (2015) requires to consider either 7 or 11 sets of motions (using a smaller load factor in the latter case) and furthermore three soil stiffness and strength scenarios: Upper bound (UB), Best estimate (BE) and Lower bound (LB). This means that the optimization of the design, complying with all the code requirements for NLTH analysis, could require as much as 110,000 analyses.

Due to the large amount of analyses for this project, an efficient design strategy is needed. The adopted strategy consists out of four parts:

1. Idealization of the parking garage as a simplified discrete OpenSees model.
2. Verification of the OpenSees model against a continuum PLAXIS 2D model and update (if necessary).
3. Calculation of the lateral displacement demand using the OpenSees model by means of NLTH analyses.
4. Optimization of strengthening measures of the concrete columns by means of NLPO analyses.

The paper focuses on the verification of the simplified discrete OpenSees model, which consists of comparing NLTH results for a selected number of cases against a more complete continuum PLAXIS 2D model. Unfortunately the needed constitutive models for the columns and base isolation are not available in PLAXIS, for which a further simplified OpenSees model was verified. Findings coming from the optimization of the strengthening measures are also presented.

It should be emphasized that the results presented here correspond to the design conducted according to the NPR 9998 (2015). It is now known that the seismic hazard will reduce because of the declining gas production in the Groningen area. This fact has not been considered in this paper and hence any specific strengthening measure mentioned is no longer valid.

2 GEOTECHNICAL INVESTIGATION AND INTERPRETATION

2.1 *Geotechnical investigation*

The geotechnical investigation consists of 7 cone penetration tests (CPTs) and 1 borehole, performed at the southwest corner of the Boterdiep garage. The CPTs reach a depth of around 24 to 25m with respect to the ground surface level. Two CPTs also contain pore pressure u_2 measurements. The borehole reached 3.0m depth, providing visual classification of the shallow layers and the phreatic level, which is found to vary between NAP +0.67m and NAP -1.83m (NAP = Average sea level at Amsterdam).

2.2 *Geotechnical interpretation*

Layer identification and parameter interpretation are based on cone tip resistance q_c , friction ratio R_f and pore pressure u_2 measurements. Layers are characterized using a version of Robertson soil behavior chart (Robertson 1990, Robertson 2010) adapted to the local Dutch conditions (CUR 1993). The saturated unit weight γ_{sat} , the undrained shear strength s_u and the drained friction angle Φ' are determined from the recommended relation with the q_c as prescribed in the Dutch geotechnical code NEN 9997-1 (2016). In case of sands with low fine contents, the undrained shear strength is considered to be equal to the drained shear strength. The shear wave velocity V_s is estimated using a regional correlation developed by Arup (2015) specifically for the province of Groningen.

The soil stratigraphy used for design is presented in Table 1, together with the most relevant design parameters. In the shallowest 5.8m of the profile, the soil is predominantly sand

Table 1. Representative soil stratigraphy and best estimate (BE) parameters

Layer No.	Level (m NAP)	Soil type (CUR 1993)	γ_{sat} (kN/m ³)	Vs (m/s)	su (kPa)	ϕ (deg)
1	+1.2 to -0.3	Sand, silty, medium dense	17.1	117	10	25.0
2	-0.3 to -1.55	Loam, sandy, medium dense	17.5	130	40	32.5
3	-1.55 to -3.8	Sand, silty, medium dense	18.5	175	19	29.0
4	-3.8 to -6.8	Loam, sandy, dense	18.1	170	70	32.5
5	-6.8 to -11.8	Clay, clean, stiff	18.5	173	80	17.5
6	-11.8 to -16.8	Clay, clean, stiff	19.1	201	110	17.5
7	-16.8 to -28.8	Clay, clean, stiff	19.4	223	140	17.5

and silty loam with medium dense compaction. The layer between 5.8m to 8.0m depth can be characterized as dense sandy loam (known locally as *Keileem*). From 8.0m until the final investigated depth, a layer of stiff clay is found, (known locally as *Potklei*). For design purposes, the ground surface level is taken as NAP +1.2m and the groundwater table as NAP -0.3m.

Table 1 is obtained by compiling the interpretation of the three most typical CPT profiles, computing the parameters as the mean values between them. In order to completely define the dynamic behavior of the soil layers, shear modulus degradation G/G_0 curves and the corresponding damping curves are computed using the relations developed by Darandeli (2001).

3 VERIFICATION OF THE DISCRETE OPENSEES MODEL

As mentioned in the introduction, the verification of the simplified discrete OpenSees model consists of comparing results for a selected number of cases against a more complete continuum PLAXIS 2D model. In this section, the PLAXIS model is first presented, afterwards the OpenSees model is presented and ultimately their results are compared.

3.1 Continuum PLAXIS 2D model

Figure 1 shows the PLAXIS 2D model, which represents the cross-section in the shortest direction of the underground parking garage. It consists of a single story with a height of 2.7m. The model actually extends from $x = -160\text{m}$ to $x = 160\text{m}$ and from $y = +1.00\text{m}$ to $y = -29.0\text{m}$, but only the central part is shown (remaining part is used to avoid lateral reflections). Hence, the model has dimensions of $L = 320\text{ m}$ and $H = 31\text{m}$. The top slab is supported by columns and the bottom slab is founded on piles. At each side of the basement, the soil is retained by a sheet pile wall whose tip extends to a level deeper than the bottom slab.

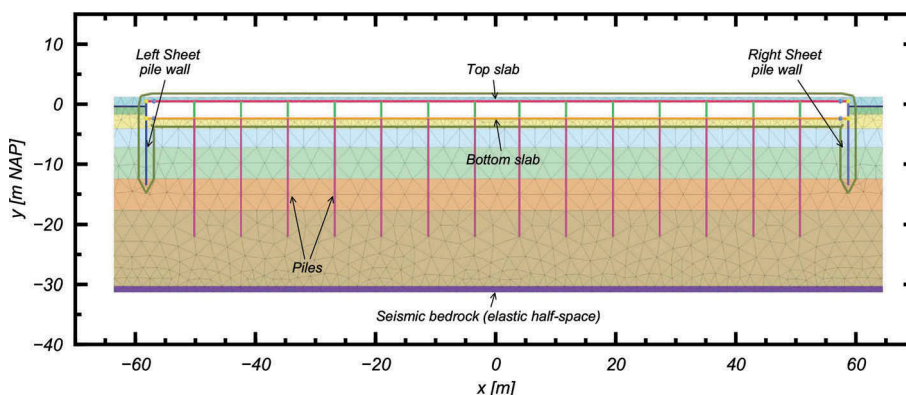


Figure 1. PLAXIS 2D model (zoomed between $X = -70$ to 70m ; vertical direction is complete)

Regarding soil elements, the Hardening Soil Small-strain (HSS) model is used for all layers, with drainage type Undrained (A). The soil parameters given in Table 1 are the basis for the HSS model. However, the undrained shear strength nor the complete shear modulus degradation curves are explicit inputs for this constitutive model. The cohesion c , unloading/reloading modulus E_{ur} and the reference strain $\gamma_{0.7}$ were varied for each layer to match its corresponding shear modulus degradation curve and undrained shear strength.

The pile foundation actually comprises pile groups under each column of 4 Vibro 410/450mm (2x2 group, c.t.c. distance of 1.2m). The distance between the pile groups is 7.8m in both horizontal directions. Each pile group is modelled as a single embedded row beam element. The moment of inertia of the pile is multiplied by 4 (number of piles in the group) and divided by the tributary length of 7.8m, to consider correctly the 3D behavior of the foundation using the proposed 2D section. Pile-to-pile interaction is not included in the model (columns yield much earlier).

Regarding structural elements, the columns are concrete round 450mm with 4 ϕ 16mm steel reinforcement. They are represented as a plate with linear elastic parameters or an M-Kappa diagram. The tributary perpendicular-to-paper length of the columns is also 7.8m. The left and right sheet pile walls are Larssen 603K and 601 sections (respectively) and are represented as plates with linear elastic parameters. The connections between the sheet pile walls and the top and bottom slabs are hinged with zero strength, i.e. they are free to rotate. All remaining connections are fixed. To the interface between the soil and the bottom slab (practically) zero strength is assigned, such that the design only relies on the piles and the sheet pile walls regarding seismic horizontal loading.

The boundary conditions are free-field boundaries at both horizontal limits of the model and compliant base at bottom of the model (NAP -31.2m). The latter is the location of the seismic bedrock, which is represented as an elastic halfspace (with parameters $V_s = 360$ m/s and $\gamma_{sat} = 20$ kN/m³). Design-spectrum consistent motions are input as acceleration time-histories at the seismic bedrock. The mass source is the self-weight of all the elements plus the mass of the building (3.67 Mg/m²), which is lumped at the top slab.

The mesh of the model contains approximately 40,000 nodes and 4600 elements. Element size was selected based on comparison of the surface motions between a PLAXIS 2D soil column (without structure) versus 1D site response analysis made in DEEPSOIL (very good agreement). Moreover, it was checked that the pore pressures and total stresses behind the sheet pile walls was between reasonable values. A total of 2000 time steps are required to solve (10.0s motion at time increments of 0.005sec). Using a processor i7-6700HQ CPU @ 2.60GHz, RAM memory of 32.0 GB and a solid-state hard drive, a single input motion takes around 2 hours to be computed.

3.2 Discrete OpenSees model

The discrete OpenSees model (Figure 2) aims to reproduce the results of the continuum PLAXIS model with sufficient accuracy (such that the overall safety is not compromised) having the advantage of a significantly smaller calculation time. In order to achieve this, the OpenSees model should be a reduction of the PLAXIS model and must include the relevant features of the physics of the problem.

From the physics perspective, the controlling motion mechanism of the seismic behavior is the drift between the top and bottom slabs. It is assumed that it is sufficient to consider: (a) the behavior of the system only in the horizontal direction (x direction); (b) free-field soil displacement time-histories at the level of the top slab (Node 10) and at the bottom slab (Node 11) as input motions. They are outputs obtained from the PLAXIS model in the far field ($X = -155$ m) at NAP +0.5m to NAP -2.2m, respectively. Alternatively, these motions could be more easily computed from 1D site response analysis; (c) the behavior of several elements of the same type as a single element.

Radiation damping elements serve to completely dissipate the compressional waves occurring between the free-field and the surrounding soil (Node 12 and 13). Such waves are generated due to the interaction between the top, bottom slab and sheet pile wall with the surrounding soil. This interaction is modelled by using the so-called (in this paper) soil-structural elements: *soil*

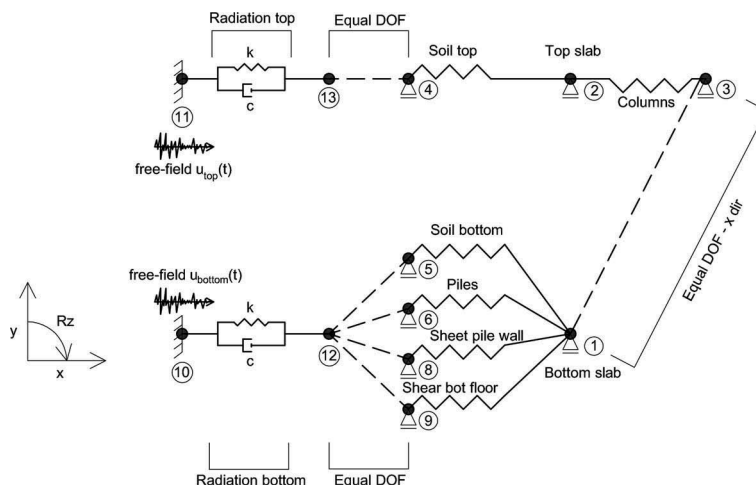


Figure 2. Visualization of the discrete OpenSees model

Table 2. Elements in OpenSees model

Name	Element type	Uniaxial Material	Method for parameters
Soil top, bottom	ZeroLength	HyperbolicGapMaterial	Rankine pressures
Piles, Sheet pile walls	ZeroLength	PySimple1	Pushover**
Shear bottom slab-soil	ZeroLength	ElasticPP	Gazetas and Tassoulas (1987)
Columns	ZeroLength	Elastic/ElasticPP	M-Kappa
Radiation top, bottom*	ZeroLength	Elastic	Gazetas and Dobry (1984)

* Input free-field soil displacements at these nodes

** D-Sheet Piling (2016) used: Bernoulli beam discretization; earth pressures Brinch-Hansen and Christensen (1961) for single pile, Culmann (1866) for sheet pile wall.

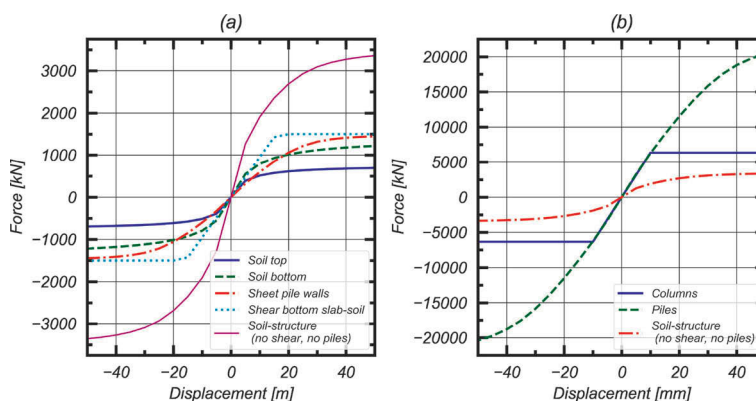


Figure 3. Backbone curves of the structural and soil-structural elements: (a) soil-structural elements excluding the piles; (b) columns, piles and the sum of soil-structural elements.

top (i.e. soil behind sheet pile wall at top slab level), *soil bottom* (i.e. soil behind sheet pile wall at bottom slab level), *piles*, *sheet pile walls* below bottom slab level, *shear bottom slab-soil* (i.e. between the bottom slab and the soil). A single element is used to consider the behavior of the structural columns that connect the top and bottom slab. All the aforementioned radiation, soil-structural and structural elements are *ZeroLength* elements (i.e. the distance between the nodes is zero). The materials assigned to these elements and the method used to

derive the required parameters are listed in Table 2. The resulting backbone curves are shown in Figure 3 (with exception of the radiation elements which are linear elastic). In all cases the forces are equal to the sum of all elements present is 7.8m length perpendicular to the paper.

Equal DOF (degree of freedom) constraints are added between the surrounding soil nodes and the soil-structural elements and between the columns and the bottom slab. The purpose of these constraints is to facilitate parameter input and the visualization of force distribution.

Note that the top and bottom slab are actually represented as nodes. The mass of both slabs is considered, while their stiffness is disregarded. The mass of the building on top of the garage is lumped into the top slab. The mesh of the model contains approximately 13 nodes, 8 elements and 6 constraints. Using a processor with specifications mentioned in the previous subsection, a single input motion (2000 time steps) takes around 15 seconds to be computed.

3.3 Result comparison

A total of six scenarios were compared between the discrete OpenSees model and the continuum PLAXIS model. Table 3 summarizes the key features of each scenario. It can be observed that variation is made regarding input motion, column elastic or inelastic behavior, connection behavior between the sheet pile wall and the slabs and the occurrence (or not) of shear between the bottom slab and the soil.

Results of the comparison are shown in Figure 4. From the drift time-histories plots, good agreement is found between the OpenSees and PLAXIS results. Both in terms of frequency content an peaks. The most salient time-history is the one for Scenario 4. In this scenario, a permanent

Table 3. Scenarios compared between OpenSees and PLAXIS models

Scenario ID	Input motion ID	Columns	Floor-Sheet pile wall connections	Shear bottom slab-soil
1	11y	Elastic	Hinged	Yes
2	11y	Elastic	Hinged	No
3	11y	Elastic	Fixed	No
4	11y	Inelastic	Hinged	No
5	6x	Elastic	Hinged	No
6	4x	Elastic	Hinged	No

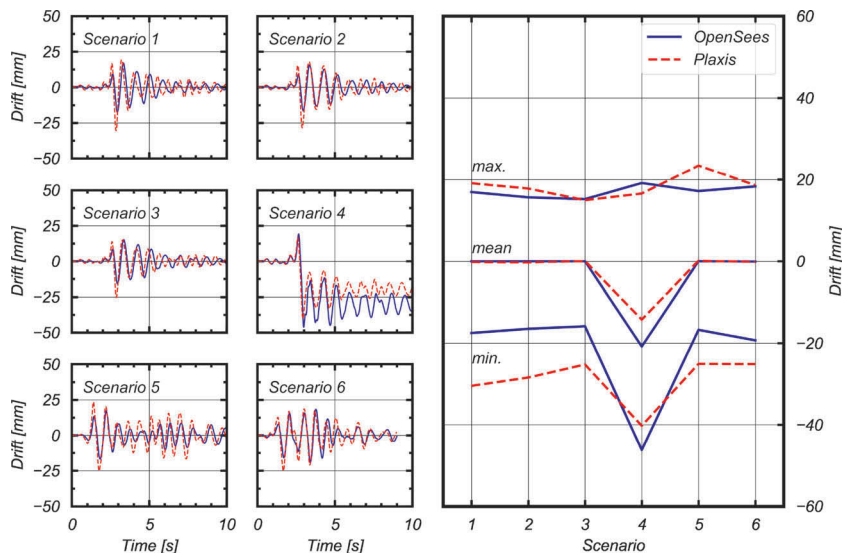


Figure 4. Result comparison between OpenSees and PLAXIS models regarding drift between slabs.

displacement of 20 mm occurs. The reason is that the columns *yield* at $t = 3.0$ s. Yielding of the columns cannot occur in the other models because the behavior of the columns is elastic.

In the plot drift vs. scenario, the maximum, mean and minimum drift per scenario are presented. It is observed that the maximum and mean values are in very good agreement, while the minimum values are underestimated in the OpenSees model by 5 to 10mm (moderate agreement) in the scenarios with elastic columns and overestimated in the OpenSees model by 5mm in the scenarios with inelastic columns (good agreement). Because the OpenSees model will be used with inelastic columns for the final design, the model is considered to be successfully verified against the continuum.

An important remark is that the level of agreement presented here was not found directly from the original parameters computed using the methods listed in Table 2. An adjustment of the radiation top and bottom elements was necessary. This highlights the importance of verifying the simplified discrete model against a more complete continuum model and updating it if necessary. Therefore, it is strongly recommended to perform this verification.

4 FINDINGS FROM THE DESIGN OPTIMIZATION

The verified Opensees model is used to calculate the lateral displacement demand (after updating column behavior and base isolation) by means of NLTH analyses. A total of 3 analysis are performed, resulting in a *NLTH demand* of 40mm. This demand is used as target for the design optimization by means of NLPO analyses. A total of 5100 NLPO analysis are performed to achieve the optimized design (each analysis takes approximately 1.70s).

Without considering the NLTH analysis and hence disregarding the soil-structure interaction, it is in principle not possible to compute the lateral demand. It could be argued that the design spectral displacement at large periods is a *conservative demand*. From Figure 5(a), it can be observed that this demand is approximately 70mm.

For the typical column (concrete round 450mm with 4 ϕ 16mm steel reinforcement) under a vertical load $F_v = 3000$ kN, the optimization process is repeated for the conservative demand of 70mm. In Figure 5(b), the results are contrasted against the NLTH demand of 40mm in terms of pushover curves. It is observed that at least 8 FRP layers are required to withstand the NLTH demand, while at least 20 FRP layers are required for the conservative demand. This means a saving in material costs of around 60% when using a NLTH approach instead of a conservative approach. This could be translated into a saving of 20% of the total cost of strengthening measures. Savings could be even greater, if it is taken into account that 20 FRP layers are not feasible to be apply to a column, meaning that a more expensive strengthening measure must be assessed (e.g. use of braces).

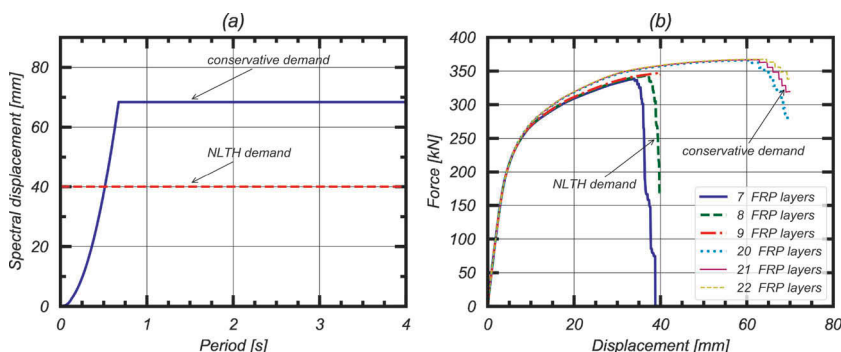


Figure 5. (a) Design displacement response spectrum indicating the NLTH demand and a conservative demand; (b) Nonlinear pushover curves for typical column under vertical load $F_v = 3000$ kN and different amount of FRP layers. They correspond to the optimal solution for the NLTH demand and a conservative demand is indicated.

The total needed calculation time for the design optimization is only 2.4 hours. This is very small relative to the 10,200 hours that would be needed if the same 5100 analyses would be performed in PLAXIS. Overall, the design strategy managed to reduce the calculation time by a factor of 5000.

5 CONCLUSIONS AND RECOMMENDATIONS

The design strategy followed for the seismic retrofitting of the underground parking garage Boterdiep, aimed at the optimization of the strengthening measures was presented. The strategy consists of four parts: (1) idealization of the parking garage as a simplified discrete OpenSees model, (2) verification of the OpenSees model against a continuum PLAXIS 2D model, (3) calculation of the lateral displacement demand using the OpenSees model by means of NLTH analyses and (4) optimization of strengthening measures of the concrete columns by means of NLPO analyses.

The OpenSees model assumes that it is sufficient to consider the behavior of the system only in the horizontal direction and that the free-field soil displacement time-histories can be used as input motions of the model. It also groups the behavior of several elements of the same type as a single element. This reduces the mesh size from 40,000 nodes to 13 nodes.

From the result comparison, it is concluded that the simplification of a continuum PLAXIS model into a discrete OpenSees model can be successful. A point of attention is that sufficiently good agreement was found only after updating the initially computed parameters for radiation damping elements of the OpenSees models. Performing the optimization with the simplified discrete OpenSees model with the proposed design strategy, produced great benefits: savings of 60% in material costs and reduction the total calculation time from 10,000 to 2.4 hours.

When trying to idealize the seismic behavior of an underground structure a discrete model, it is strongly recommended to perform the verification against a more complete continuum model. Ideas for further improvements of the discrete model can come from: (a) more accurate parameter computation of the radiation damping elements, (b) modelling the radiation damping elements using nonlinear materials and (c) replacing the radiation damping elements by the 1D soil-column instead.

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