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MODELLING NON-LINEAR SOIL-STRUCTURE INTERACTION FOR DYNAMIC EARTHQUAKE ANALYSES

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ABSTRACT

The research presented in this paper provides a clear insight in the current state of modelling soil-structure interaction in Non-Linear Time History (NLTH) analyses using the finite element analyses package DIANA. NLTH analyses are performed to assess the earthquake resistance of existing buildings in Groningen, The Netherlands. The paper discusses some important theories on soil-structure interaction effects and how these effects are captured in the models. It provides background on and discusses the use of the Hardin Drnevich constitutive soil model, half space boundary conditions, radiation and material damping, mesh configurations and discrete non-linearities in the form of interfaces and springs. In addition, two cases are presented and discussed, which include an object with a shallow foundation and an object with a pile foundation.

Keywords: Non-Linear Time History Analyses; Soil-Structure Interaction; DIANA; Fully Coupled Analyses; Groningen

1. INTRODUCTION

The northern part of The Netherlands is subject to the occurrence of induced earthquakes that result from the extraction of natural gas from the Groningen gas field. Most of the buildings have never been designed to resist earthquakes. A lot of them consist of unreinforced masonry structures. Soil conditions are variable and include clay, silt, peat and sand deposits. The earthquakes have a short duration time and are shallow in comparison to tectonic earthquakes.

Royal HaskoningDHV performs research on the structural integrity of existing buildings in Groningen and advises on mitigating measures in terms of structural reinforcements. For some objects, Non-Linear Time History (NLTH) analyses are performed to assess the earthquake resistance of structures using near collapse criteria. These analyses are performed with the Finite Element Analysis (FEA) software package DIANA.

This paper discusses the results of recent research on modelling Soil-Structure Interaction (SSI) for direct NLTH analyses. In direct analyses both the soil and structure are included in one finite element model. To describe the non-linear dynamic behavior of a building during an earthquake, it is essential to also include the non-linear behavior of the foundation system and the underlying and surrounding soil. The general objective of this research is to model the effects of SSI as accurately as possible with the available means.

In addition, this paper provides a brief introduction on the theory of SSI, a description of how SSI is included in the DIANA models and the results of SSI effects in NLTH analyses.

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2. SOIL-STRUCTURE INTERACTION

Soil-structure interaction, as described throughout this paper, is the dynamic interaction between a building structure, its foundation and the underlying and surrounding soil during an earthquake. SSI effects can be differentiated into inertial interaction effects, kinematic interaction effects and the deformations of the foundation elements. In NLTH analyses it is essential to capture these effects to obtain realistic behavior of the structure and foundation.

2.1 Inertial interaction

The effects of inertial soil-structure interaction can be explained by comparing a fixed base analysis with a flexible base analysis. A fixed base consists of rigid supports. A flexible base incorporates the stiffness and damping properties of the soil and foundation elements. Two phenomena can be observed when comparing a fixed base analysis with a flexible base analysis: period lengthening of the complete system and change of frequency dependent damping.

The period lengthening of the system occurs because the translational and rotational stiffness decreases when the stiffness of the soil and the foundations elements are incorporated in the analyses. This causes a shift in the spectral response of the building. The damping for the entire system will be higher for a flexible base than for a fixed base analysis, resulting in an overall lower response of the structure. Two types of damping are responsible for this higher damping ratio: material damping and radiation damping. (NEHRP 2012)

2.2 Kinematic interaction

During an earthquake, stiff foundation elements in a softer soil cause different motions at foundation level when compared to free-field motions. This results in a kinematic interaction between the structure and the soil. Effects that cause this deviation are base slab averaging and embedded effects.

Base slab averaging can occur when foundations are so large and stiff that they level out the motions of the earthquake. This is comparable to what happens to a large vessel on ocean waves. The motions of the vessel are averaged by the vessels stiffness.

Basements, foundation piles and almost all shallow foundations are embedded in the soil and cause kinematic interaction during an earthquake. Theory states that foundation-level motions are reduced, because ground motions generally reduce with depth below the free surface. In Groningen however, very soft layers in the top of the soil profile may cause damping. This may result in higher motions at deeper levels than at free-field levels, in which case the response at foundation level would also be higher. Finally, kinematic interaction between the soil and piles can cause damage to the piles. (NEHRP 2012)

2.3 Foundation deformations

The stiffness of the foundation is an essential part of soil-structure interaction in NLTH analyses. It influences global and local effects of SSI. Globally, it determines the period lengthening of the system, as discussed in section 2.1, and therefore the response of the building at base level. Locally, it has its effects on the distribution of forces in the building's structure. This is illustrated by a small example.

Imagine a shear wall in a building. Due to its stiffness, it is likely the wall is subject to large horizontal forces during an earthquake. These forces are transferred to the foundation. If the foundation fails as a result, the wall will undergo displacement, changing the entire force distribution in the building. In an ordinary situation, one could design a foundation such that it will be able to withstand these forces. However, in the case of an existing building the foundation is already present. Therefore, when performing an NLTH analyses for an existing building, the maximum capacity of the foundation and its stiffness must be an input value of the model.

3. MODELLING NLTH ANALYSES IN DIANA

DIANA is a finite elements analyses software package. It provides constitutive models for structural and geotechnical non-linear dynamic analyses. Therefore, it is possible to perform direct analyses of an earthquake. Figure 1 shows a non-linear DIANA model of a building structure and a soil block. The purpose of these direct analyses is to include three types of behavior in the analyses: the non-linear ground response, the non-linear behavior of the structure and soil-structure interaction.

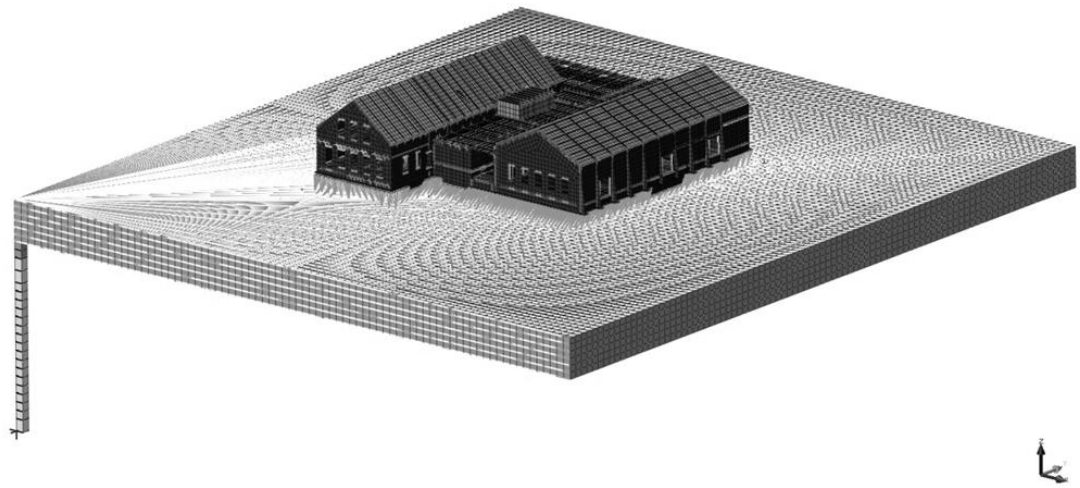

Figure 1. Typical non-linear DIANA model of a building structure and a soil block.

3.1 Non-linear ground response

The non-linear ground response is obtained with a soil block. The soil block consists of a 3D soil block on top of a 1D soil column, see Figure 1. The purpose of this column is to reduce the number of elements and computation time of the analyses.

3.1.1 Hardin Drnevich Soil Model

DIANA incorporates the Hardin-Drnevich constitutive model to model the dynamic response of a soil column in earthquake conditions. The Hardin-Drnevich constitutive model is a linear elastic soil model with a non-linear shear stress - shear strain relationship. As a result, it can reliably model the propagation of earthquake waves through the soil mass, where hysteresis in shear results in damping during cyclic loading, but it does not capture non-linear behavior in compression and tension. The Hardin-Drnevich model behaves according to the so-called extended Masing rules:

- For initial loading, the shear stress - strain relationship is prescribed by a backbone curve.
- When reloading or unloading from the initial loading occurs, the shear stress-strain relationship forms a loop, which is obtained by scaling the backbone curve by a factor two.
- If the previous maximum shear strain is exceeded, the shear stress - strain relationship again follows the backbone curve.
- If the hysteresis loop intersects a previous loading or unloading curve, the shear stress - strain relationship follows that previous curve.

In the Hardin–Drnevich model, the relationship between shear stress τ and shear strain γ is defined by:

$$\tau = \frac{G_{max}\,\gamma}{1 + \frac{\gamma}{\gamma_r}} \tag{1}$$

in which γ_r represents the characteristic shear strain. The maximum tangent shear modulus G_{max} is calculated from:

$$G_{max} = \frac{E}{2(1 + \upsilon)} \tag{2}$$

where E and υ are the initial Young's modulus and Poisson's ratio, respectively. The normal stress – strain relationship is fully linear and defined by:

$$\sigma = \frac{E}{\varepsilon} \tag{3}$$

The advantage of the Hardin-Drnevich soil model is that it converges relatively quickly in numerical calculations. The main disadvantage is that it is unable to describe realistic elasto-plastic soil behavior:

- Compression gives an elastic response only based on the defined Young's modulus. There is no explicit shear strength criterium;
- No difference is made between the soil response in compression or tension, although soil can absorb no or only a small amount of tension;
- The shear stress - strain relationship is hyperbolic and approaches a limiting undrained shear strength at large strains as defined in the backbone curve. The parameters are defined beforehand and are not dependent on the normal stress conditions in the model. This not a problem for 1D ground response analyses, but it is for a coupled analysis where the dynamic behavior of a structure influences the stress conditions in the soil.

Because of the limitations of the Hardin-Drnevich soil model, all non-linear soil behavior at the foundation interface must be modelled in a discrete manner. This can be achieved by applying interface elements with non-linear properties between foundation elements and the soil.

3.1.2 Half space boundary conditions

Half space boundary conditions are introduced to prevent waves from reflecting at the edges of the soil block. Two types of boundary conditions are distinguished: spring-dampers and tyings.

At the bottom of the soil block a spring and a damper are placed in x-, y- and z-direction. The spring stiffness and damper properties are determined according to Lysmer and Kuhlemeyer (1969). The formulas for the horizontal damper constant (c_h), vertical damper constant (c_v), horizontal spring stiffness (k_h) and vertical spring stiffness (k_v) are presented below.

$$c_h = v_s \rho A \tag{4}$$

$$c_v = v_p \rho A \tag{5}$$

$$k_h \cong 0 \tag{6}$$

$$k_v = \frac{W}{\Delta_l} \tag{7}$$

$$W = hg\rho A \tag{8}$$

$$\Delta_l = \frac{W}{\left(\frac{A}{\rho v_s^2 2\left(1+\left(\frac{\rho v_p^2 - 2\rho v_s^2}{2\rho v_s^2 + 2\left(\rho v_p^2 - 2\rho v_s^2\right)}\right)\right)h}\right)} \tag{9}$$

Where:

v_s	shear wave velocity
v_p	pressure wave velocity
ρ	average density of the soil block
A	area of the soil block
W	weight of the soil block
h	height of the soil block
g	gravity constant
Δ_l	deformation of the soil below the soil block

A tying is a feature in DIANA that links the displacements of a master node to the displacement of a single or multiple slave nodes. The models in DIANA have limited dimensions, typically 100m by 100m. It is assumed that shear and pressure waves have a constant propagation within this area. All nodes at the edge of soil block are tied for each mesh layer in x-, y- and z-direction. This is illustrated in Figure 2. The edges of the soil block are modelled at least 25 meters from the outer edges of the object. This is a sufficient distance to allow waves, caused by the kinematic interaction between the structure and the soil, to attenuate before returning to the building.

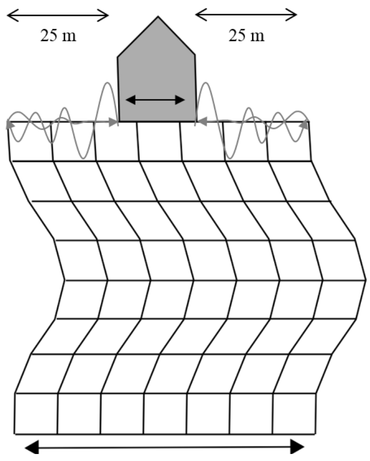

Figure 2. Schematic mesh configuration on the soil block with tied outer nodes.

3.1.3 Dynamic earthquake load

The dynamic earthquake load is introduced at the bottom of the soil block, at the same node where the springs and dampers are connected. The load is applied as forces that consist of a velocity $\dot{u_h}(t)$ and $\dot{u_v}(t)$ times a damper constant c_h and c_v. Both are scaled with the peak acceleration at bedrock level $a_{g,ref}$. The formulas are given below.

$$F_h(t) = a_{g,ref} c_h \dot{u}_h(t) \tag{10}$$

$$F_v(t) = a_{g,ref} c_v \dot{u}_v(t) \tag{11}$$

3.1.4 Mesh configuration of the soil block

The mesh of the soil consists of cubic solid elements. In horizontal direction, the mesh size of the soil is 1.0 m by 1.0 m. In vertical direction, the size depends on the soil type. Softer soils need a finer mesh to describe the propagation of shear waves. Typically, the vertical mesh size is between 0.5 m and 2.0 m. This is a very coarse mesh configuration for a soil block, but considered acceptable. Verification analyses have shown that the DIANA ground response corresponds with the ground response in other packages like Plaxis and Deepsoil.

Triangular and trapezium shaped elements are avoided. They behave slightly stiffer than cubic shaped elements, which can result in a higher local strain. When this strain exceeds the maximum of the defined stress strain curve, it causes numerical divergence and the analyses will fail.

The coarse mesh of cubic shaped elements helps to make the complex NLTH analyses more stable and reduces computation time to about a week.

3.1.5 Embedded effects

Most objects analyzed with DIANA are embedded in the soil. It is complicated to include these embedded effects into the Hardin Drnevich soil model. The linear elastic model is not able to model failure. Also, it is hard to generate a mesh of solid elements that align with the mesh of the structure that consists of shell elements. Therefore, the soil block is modelled to the level of the bottom of the foundation beam or foundation strip and the surrounding soil is replaced with a uniform surface load. The passive soil pressure against the sides of the foundation is neglected. This is based on engineering judgement. This approach is considered valid to a depth of maximum 2 m. For basements, an alternative solution is required.

3.2 Soil-structure interaction

For some soil-structure interaction effects, there is a difference between shallow and pile foundations. Other effects, such as radiation damping, were discussed earlier.

3.2.1 Shallow foundations

To overcome the intrinsic limitations of the Hardin Drnevich soil constitutive model, discrete non-linearity is added by means of an interface between the soil and the foundation slab.

A foundation slab consists of shell elements with an average dimension of 0.3 m. The coarse mesh of the soil and the mesh of the foundation slab are connected with tyings. The 3D soil block has a maximum thickness of 5m to limit computation times. The interface is located between the tyings and the mesh of the foundation slab. This is illustrated in Figure 3.

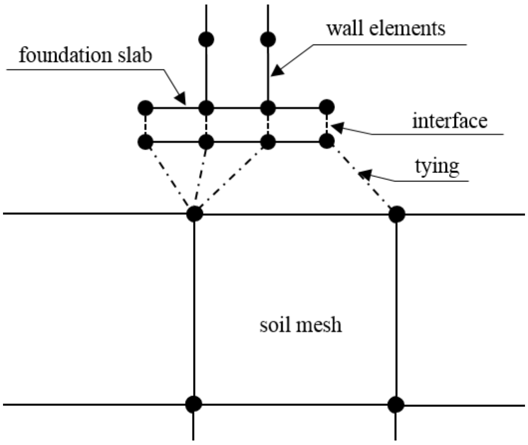

Figure 3. Mesh configuration shallow foundations in DIANA

The specified interface properties are critical to the behavior of the shallow foundation. The interface must be able to describe non-linear plastic behavior in vertical direction. In horizontal direction, the interface must have a coulomb friction resistance depending on the vertical stress level.

In vertical direction, the maximum stress is equal to the maximum bearing capacity. The tension capacity of the foundation slab is almost equal to zero. The stiffness (k) is determined based on the target that the natural frequency, at which the building behaves as a one-degree-of-freedom system, must be higher than 50 Hz in horizontal and vertical direction. At that frequency, the energy in the signal is approximately equal to zero. There is no physical explanation for the interface stiffness. The only purpose of the interface is to limit the bearing capacity of the foundation. The higher stiffness is applied to eliminate numerical noise.

$$k = m(50 \cdot 2\pi)^2 \tag{12}$$

Where m represents the mass of the building.

The interface behaves linearly elastic until it reaches the failure stress level. For smaller strains, the interface does not include hysteretic material damping. Therefore, 5% Rayleigh damping between 0.1 Hz and 30.0 Hz is added to the interface. Because this is a model for direct analyses, base-slab averaging is automatically included in the model.

Stress displacement behavior of the soil, the interface and the combined behavior are illustrated in Figure 4.

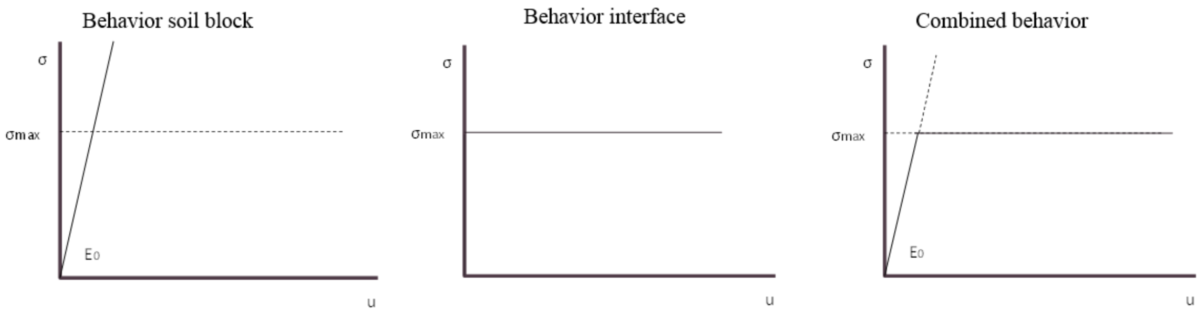

Figure 4. Stress displacement behavior of the soil, the interface and the combined behavior.

3.2.2 Pile foundations

Piles in DIANA are modelled with embedded reinforcement elements. These are interface elements that assign their properties to the elements they cross. One of the biggest advantages of embedded elements is that they do not need to align with the mesh of the soil. This makes the coarse mesh possible, which saves computation time and leads to a more stable analysis. A disadvantage is that these elements are not able to describe the non-linear behavior of a concrete pile. Like shallow foundations, pile foundations also require a discrete non-linearity. It would be desirable to create an interface around the pile that can describe non-linear plastic behavior, but this feature was not available in DIANA. Therefore, a simplified configuration was achieved with springs between the top of the pile and the foundation beam. The bottom of the pile is tied in vertical direction to its actual pile tip level. The pile configuration in DIANA is illustrated in Figure 5.

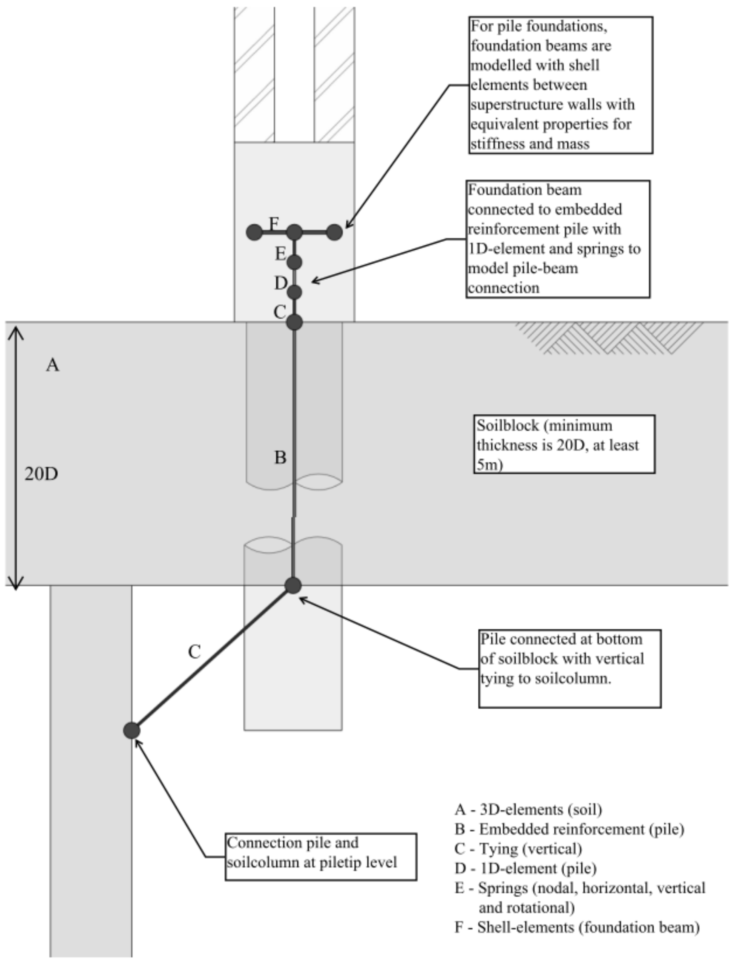

Figure 5. Schematization pile modelling in DIANA.

The piles are not modelled over their complete length. At a depth of 20 times the diameter of the pile the rotations due to inertial forces from the building are approximately zero. Therefore, the 3D soil block has a thickness of 20 times the diameter of the pile. At deeper levels, earthquakes can cause damage to piles as well, but DIANA does not allow the modelling of these kinematic interaction effects yet.

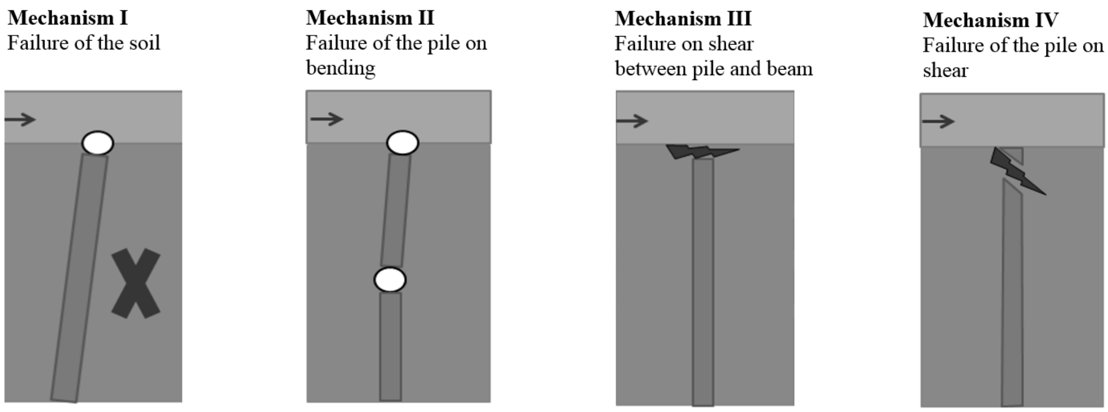

Figure 6. Four failure mechanism for pile foundations during earthquake loading.

The springs at the top of the pile have a vertical and horizontal component and are named f_{crit}, where ‘crit’ represents the critical value at which the spring behaves perfectly plastic. The spring is able to dissipate energy via hysteretic loops, but for smaller strains a damping factor of 5% for material damping is included. In vertical direction, the maximum bearing capacity is based on the undrained soil strength, including strength degradation and liquefaction effects. Potential buckling effects of the pile are not included in the model. For lateral loading, four failure mechanisms are considered, as illustrated in Figure 6. Prior to the NLTH analyses, the governing failure mechanism is determined. The force needed for this failure mechanism is the critical value of the f_{crit} spring.

4. SSI RESULTS OF NLTH ANALYSES

The results of two objects analyzed with a NLTH analysis are discussed.

4.1 Specification of case objects

Table 1 provides specifications of these objects and Figure 7 shows their 3D models. In this paper, the result of one input motion are evaluated. During the actual analyses, seven signals are applied to obtain the behavior of the structure.

Table 1. Specification of objects analyzed with NLTH analyses (NPR 9998:2015)

	Object 1	**Object 2**
Foundation type	Shallow	Piles
$A_{g;ref}$ [g]*	0.36	0.12
Number of stories	3	1
Material load bearing walls	Unreinforced masonry	Unreinforced masonry
Number of signals	7	7
Duration earthquake signal [s]	10.0	10.0

* EUROCODE 8: A_{gR} (NEN-EN 1998-1 2015)

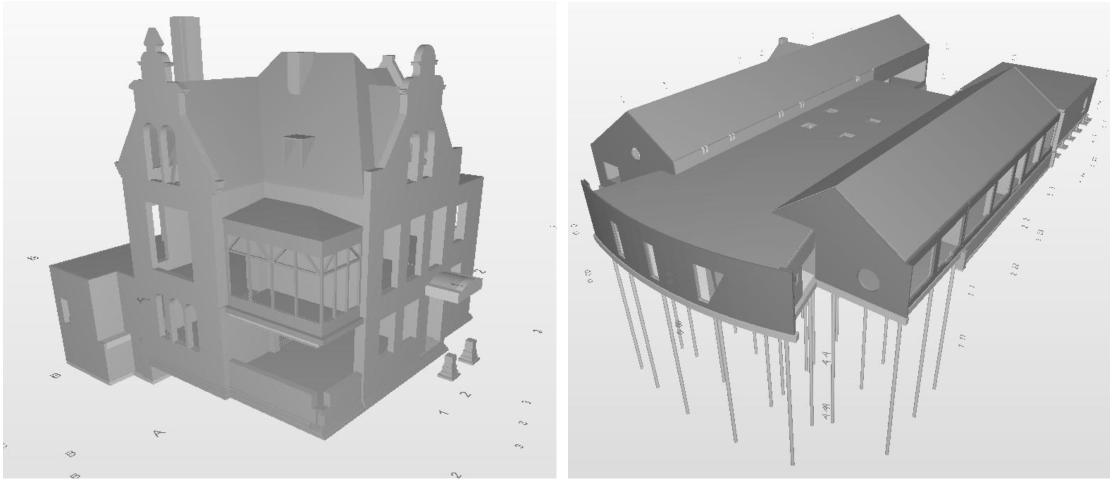

Figure 7. 3D models of objects analyzed with NLTH analyses: Object 1 (left), Object 2 (right).

The soil properties for both objects are given in Table 2 and Table 3. The properties are for upper bound analyses, because this situation is governing for the structure in most cases. The results in this paper are also for upper bound analyses. During the actual retrofit studies, lower bound and best estimate analyses are also considered. The unified time signal in x-direction is shown in Figure 8.

Table 2. Soil Properties for HD-model for Object 1

Top level [m]	**Bottom level [m]**	**Description**	**Weight [kN/m^3]**	$\boldsymbol{v_s^{(1)}}$ **[m/s]**	$\boldsymbol{G_0^{(1)}}$ **[N/m^2]**	$\boldsymbol{\nu^{(1)}}$ **[-]**	$\boldsymbol{E^{(1)}}$ **[N/m^2]**
0.0	-1.5	Clay	16.6	98	1.63E+07	0.4979	4.87E+07
-1.5	-4.0	Clay	15.1	104	1.67E+07	0.4976	4.98E+07
-4.0	-9.25	Clay	15.0	124	2.35E+07	0.4966	7.02E+07
-9.25	-12.75	Sand	17.1	190	6.29E+07	0.4918	1.88E+08
-12.75	-16.5	Sand	18.4	253	1.20E+08	0.4854	3.57E+08
-16.5	-21.0	Sand	20.3	340	2.39E+08	0.4729	7.04E+08
-21.0	-25.5	Sand	21.1	395	3.36E+08	0.4627	9.81E+08
-25.5	-30.0	Sand	19.9	368	2.75E+08	0.4680	8.06E+08

(1): Upper bound values (NPR 9998:2015)

Table 3. Soil Properties for HD-model for Object 2

Top level [m]	**Bottom level [m]**	**Description**	**Weight [kN/m^3]**	$\boldsymbol{v_s^{(1)}}$ **[m/s]**	$\boldsymbol{G_0^{(1)}}$ **[N/m^2]**	$\boldsymbol{\nu^{(1)}}$ **[-]**	$\boldsymbol{E^{(1)}}$ **[N/m^2]**
0.0	-1.0	Clay	16.3	91	1.37E+07	0.4982	4.12E+07
-1.0	-1.5	Clay	17.6	124	2.77E+07	0.4966	8.29E+07
-1.5	-2.25	Sand	19.0	200	7.76E+07	0.4910	2.31E+07
-2.25	-12.0	Sand	18.7	247	1.16E+08	0.4861	3.45E+08
-12.0	-18.25	Sand	19.4	293	1.70E+08	0.4802	5.03E+08
-18.25	-23.75	Loam	18.7	261	1.30E+08	0.4844	3.86E+08
-23.75	-30.0	Sand	20.3	371	2.85E+08	0.4674	8.37E+08

(1): Upper bound values (NPR 9998:2015)

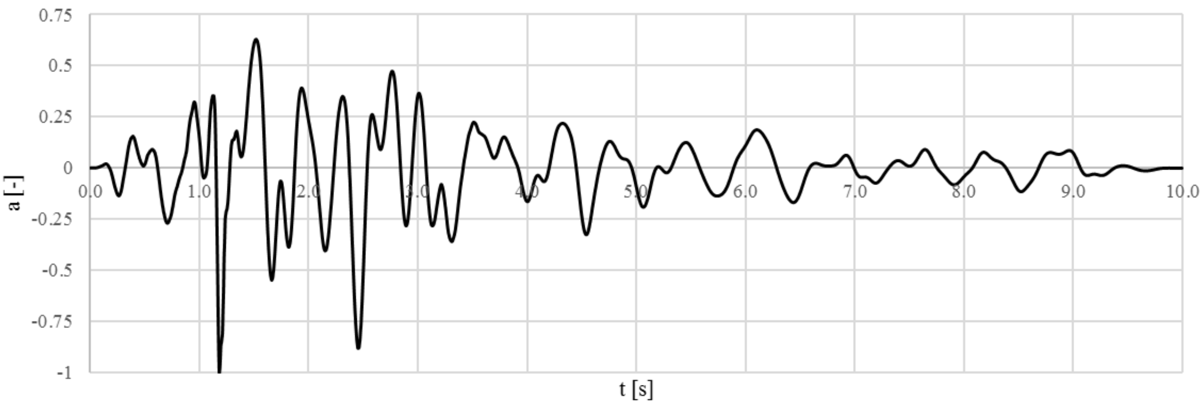

Figure 8. Time signal in x-direction. (NPR 9998:2015)

4.2 Comparison between earthquake responses

A period shift and reduction of the base shear is a good indication of inertial SSI effects. Similarly, the response of the base of a building to an earthquake, compared with the far field response, provides an indication of SSI effects in an NLTH analysis.

Figure 9 and Figure 10 show the responses to an earthquake signal at multiple nodes in the NLTH analysis for Objects 1 and 2 respectively. The response spectra are generated with a damper constant of 5% of the critical damping. The dotted line describes the far field response. This is a point where the structure has no influence on the response. In the DIANA models, this is a point at the edge of the soil block. The dashed and the continuous lines represent locations at the edge of the building, at ground level and at foundation level respectively. The differences between these two spectra show the effects of the interfaces and springs, as described in Section 3.

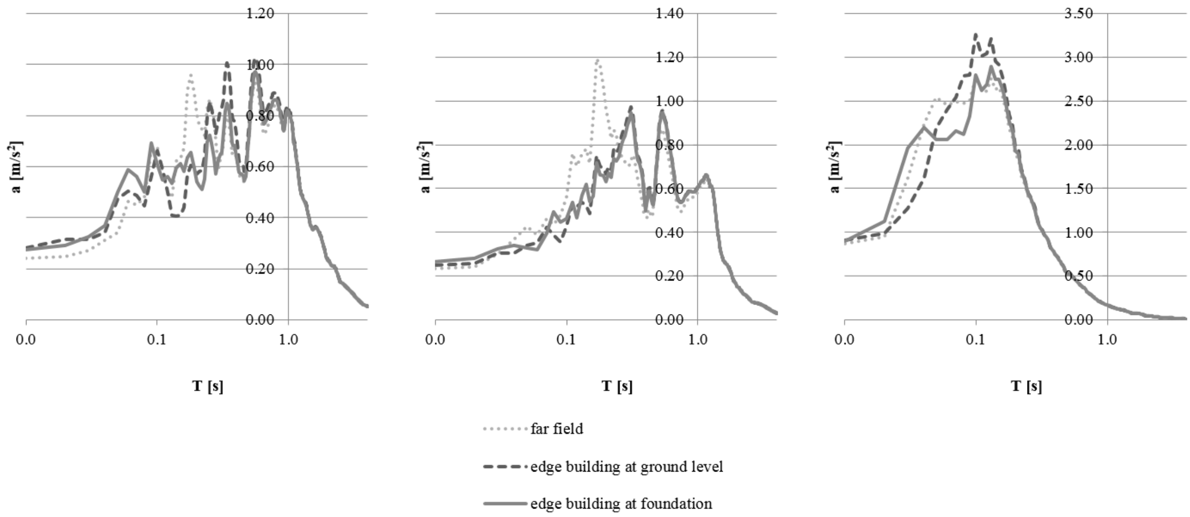

Figure 9. Response spectra Object 1 in x-direction (left), y-direction (middle) and z-direction (right)

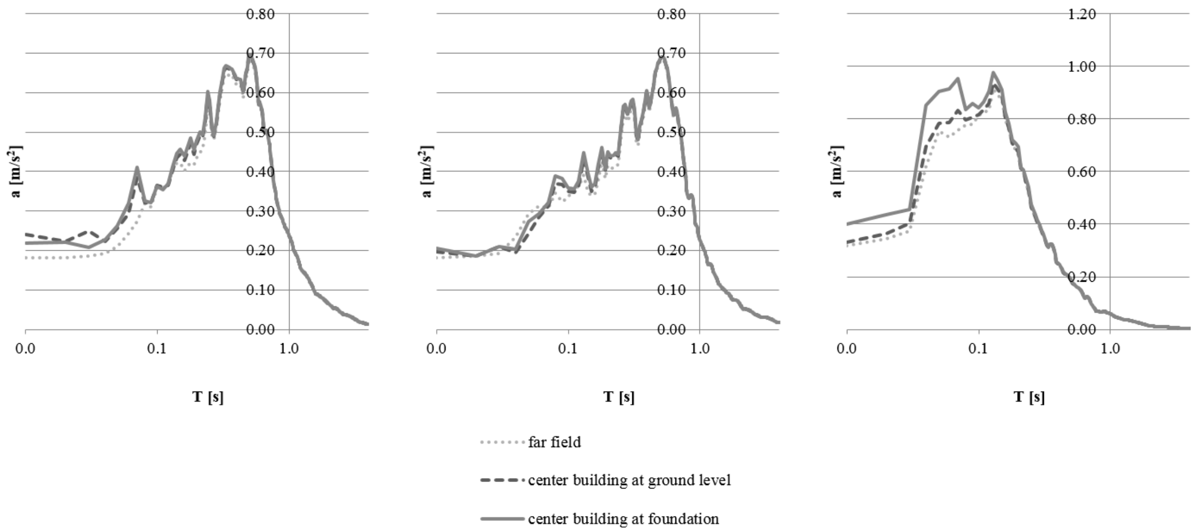

Figure 10. Response spectra Object 2 in x-direction (left), y-direction (middle) and z-direction (right).

Object 1 shows a slight period shift. In horizontal direction and for smaller periods the response of the building is lower than the far field response. In vertical direction, there are differences between the continuous and the dashed lines. This indicates that a part of the interface reached its critical limit. For a near collapse situation, failure of the foundation can be acceptable if the forces can be redistributed.

The response spectra of Object 2 are closer together. There is no noticeable period shift and almost no damping. This means that the response of the building is almost the same as the far field response. This might be due to the fact that the intensity of the input signal is a lot smaller than for Object 1. In addition, Object 2 is lighter than Object 1. The heavier the structure, the larger the inertial effects.

6. CONCLUSIONS AND RECOMMENDATIONS

The research showed that it is possible to perform non-linear time history analyses in DIANA that include a direct approach for modelling soil-structure interaction. Due to the limitations of the Hardin Drnevich soil model, discrete non-linearities are necessary to be able to model plastic failure of foundation elements. This can be achieved by applying springs and interfaces with non-linear plastic properties between the soil and the foundation elements.

A new soil model called Modified Mohr Coulomb Small Strain is currently being developed. This model will be able to capture the dynamic behavior of soils in a more realistic manner, but will also require a much finer mesh. The increase in computation time could be compensated with the explicit solver that is also under development.

User defined sub-routines can be developed if a feature is not available, which may describe the non-linear behavior of the foundation elements and the soil around it more realistically. Additional research is required before these can be implemented.

For now, the results show behavior corresponding with theory and research will continue as there are still many buildings in Groningen to be reinforced.

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