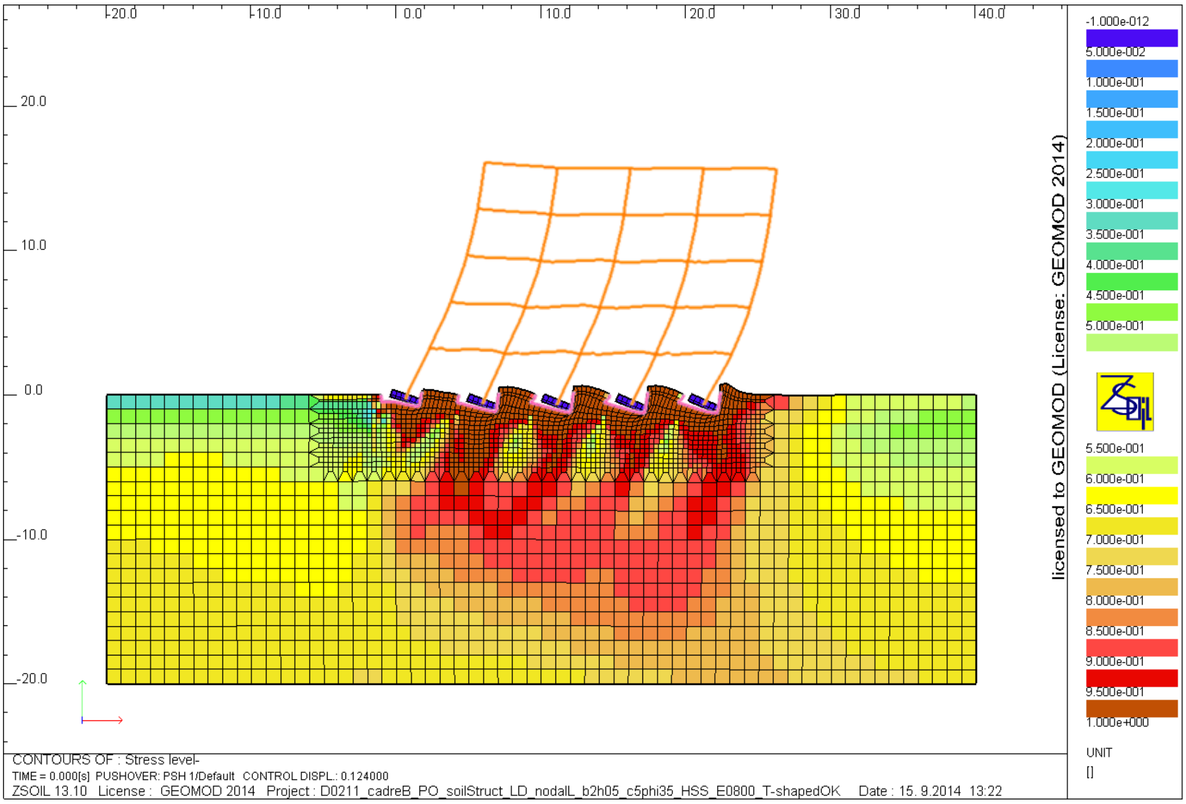
Nonlinear pushover analyses

Taking soil into account



Commissioned by the Federal Office for the Environment (FOEN)

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Imprint

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1. Introduction

In May 2014, GeoMod ingénieurs conseils SA has received a contract from the Federal Office for the Environment (FOEN) in order to evaluate the importance of taking soil structure interaction into account when performing nonlinear pushover analyses.

1.1 Study objectives and methodology

This study should provide answers to the following questions:

- What is the necessity and potential benefit of taking soil structure interaction into account when performing finite element seismic assessment of structures (mainly buildings and bridges) through nonlinear pushover analyses?
- Which constitutive laws and which characteristic values of the soil parameters should be used to model the soil, so that results remain on the conservative side according to the principles of the buildings codes?

In order to answer the first question, it has been decided to compare the results of the following different pushover analyses on an example 5-storey building [1], performed with the finite element software package ZSOIL [2]:

- 2D structural only analysis, with clamped boundary conditions at terrain level
- 2D structure + soil analysis, considering two types of foundations:
    - Shallow isolated foundations
    - Rigid mat foundation

The comparison of the results is focused on the bending moment distribution and the maximum chord rotation for the critical structural elements when the roof top displacement reaches the displacement demand (target displacement). The detailed assessment of the seismic safety of the example building is beyond the scope of this study. Whenever possible, a comparison of the compliance factor values for the critical structural elements with and without taking soil into consideration is given.

An estimation of the influence of the foundation flexibility on the bending moment distribution in the elastic domain has also been performed in order to estimate the influence of the introduction of foundation flexibility in classical seismic force based verification analyses.

To answer the second question, a parametric study has been conducted on the 2D structure + soil analysis with shallow isolated foundations, with the following varying parameters:

- Constitutive law used to model the soil: Hardening Soil with small strain extension model [3] vs. Mohr-Coulomb model
- Soil parameters: use of different elastic moduli E and friction angles ϕ for the Mohr-Coulomb model
- Seismicity level: Z3b (a_g = 1.6 m/s^2) vs. Z2 (a_g = 1.0 m/s^2)

Remark: the validation of the pushover approach with and without soil has been performed on another example structure by comparing results with the ones given by a time history analysis considering as seismic input an accelerogram compatible with the pushover demand spectrum. Details can be found in [4] and [5].

1.2 Problem description

Classical « structural only » pushover analysis assumes that the structure has a rigid foundation and usually doesn't consider soil-structure interaction: the foundation is replaced by a fixed boundary condition at the soil-structure interface. This approach is generally (but not always) on the safe side. When the rigidity and/or the failure mode of the foundation system contributes to a substantial amount of the horizontal displacement of the structure under seismic loading, taking soil structure interaction into consideration will be generally beneficial for the verification of the structural safety of structural elements (reduction of forces in the structure). Sometimes neglecting soil structure interaction is not on the safe side (for example for structures that are sensitive to P-Δ effects, for structures founded on foundations with very different rigidities, or for foundations on soft soils). It is then mandatory to include soil structure interaction in the modelization of the problem. Several research teams have addressed this question in [6] (see Figure 1.1).

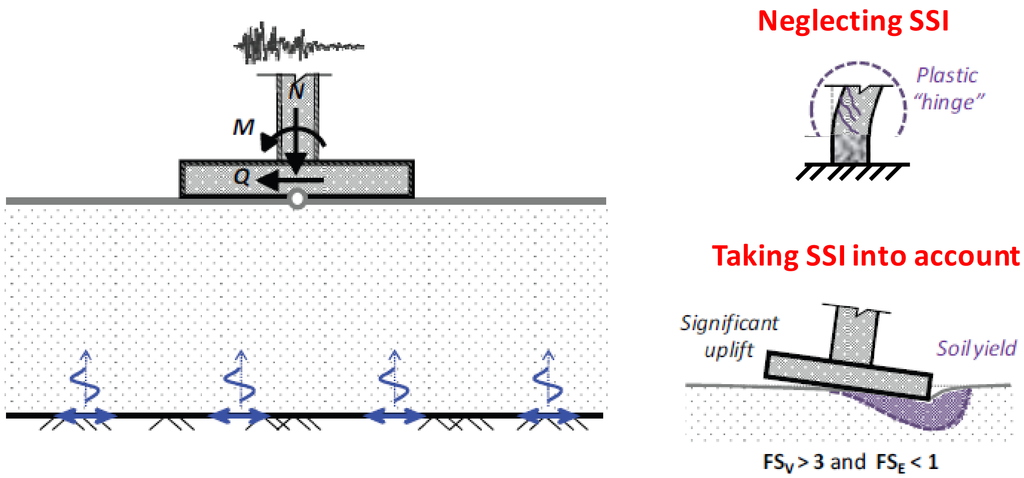


Figure 1.1 Taking soil structure interaction into account (modified, from [6])

In the pushover method, the capacity curve of the structure is obtained by imposing a force distribution to the structure and raising it monotonously. The top displacement is then related to base shear V_b (see Figure 1.2). With the classical structural only pushover method, raising the external force will increase the bending moment at the base of the structure until it reaches a yield plateau (red curve in Figure 1.2). Considering the structure-foundation-soil system, the foundation will transmit the bending moment at the base of the structure to the soil underneath the foundation and two situations can occur [4]:

- either the plastic moment of the soil is greater than the plastic moment of the structure (dotted green curve) and the global response (dotted black curve) will be close to the structural only response: V_b^{max}(structure+soil) will be roughly equal to V_b^{max}(structure)
- or the soil's parameters (or the foundation type and dimensions) are such that the soil's plastic moment under the foundation is smaller than the structure's plastic moment, and the global response (plain black curve) will be bounded by the soil's behavior (plain green curve) which will trigger a rocking phenomenon: the bending moment at the base of the structure will not reach its plastic value, and V_b^{max}(structure+soil) will smaller than V_b^{max}(structure)

In conclusion, the influence of taking the foundation and soil into account on the force-displacement capacity curve can a priori be expected to be anywhere, from negligible to significant, depending on various factors: soil type, foundation type and dimensions, structural stiffness.

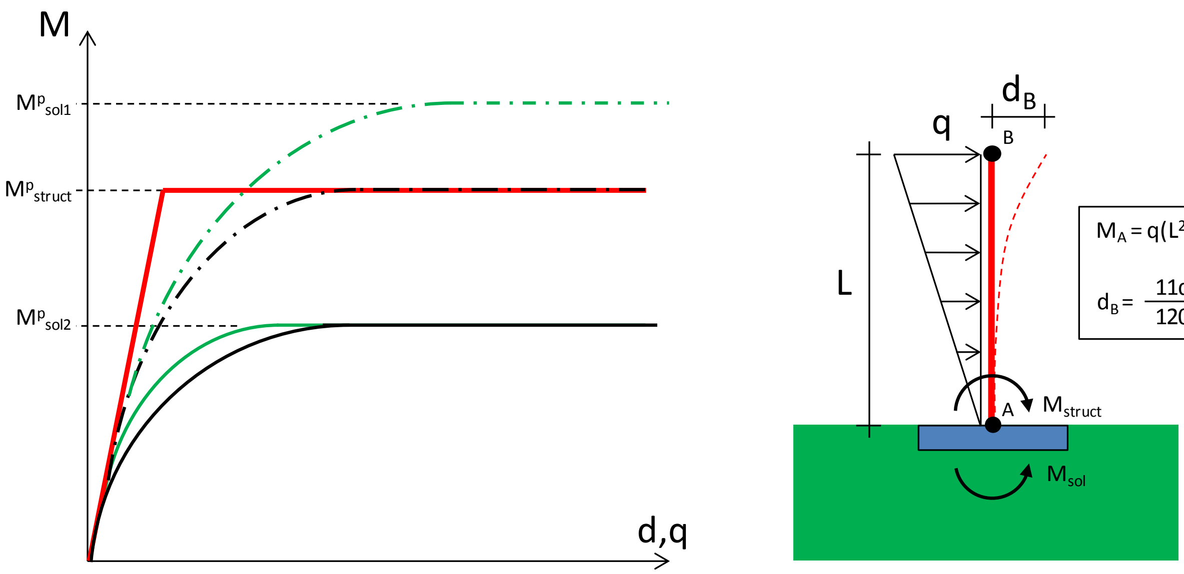


Figure 1.2. Structure-foundation-soil system: moment displacement curves [4]

2. Example building description

2.1 Structure

Figures 2.1 to 2.4 summarize the characteristics of the example 5-storey reinforced concrete building taken from [1] as a reference for this study. For subsequent 2D analyses and for the sake of simplicity, only frame B will be considered (assumption: frame B is repeated every 5 m).

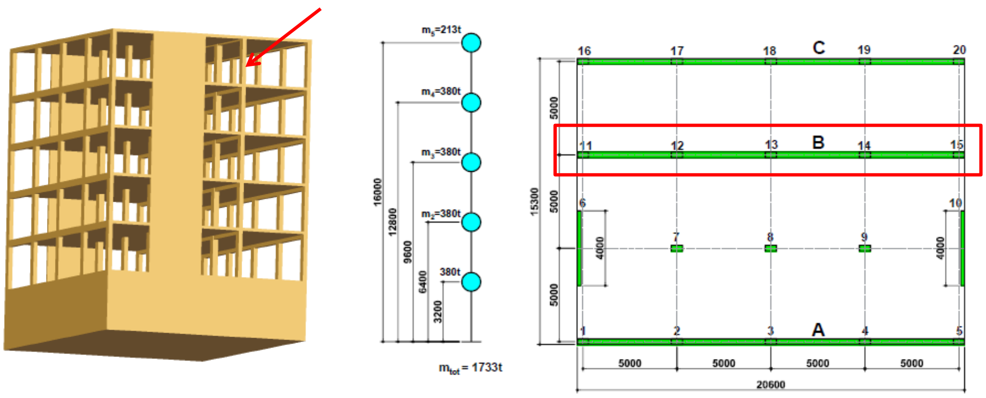
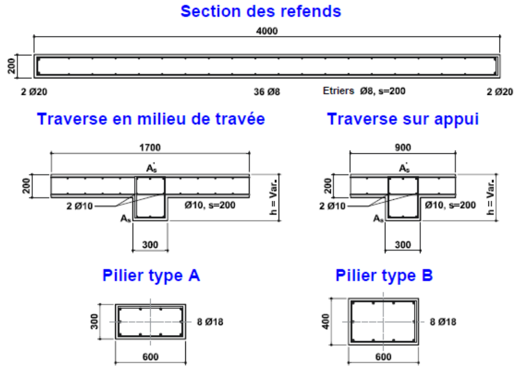


Figure 2.1. Example building geometry

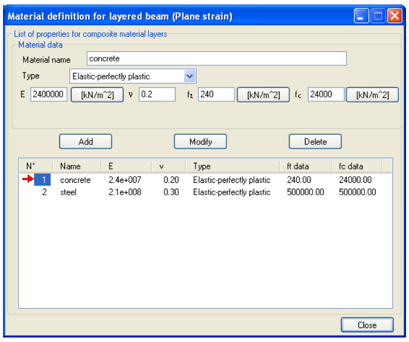


Type & Localisation	h [mm]	A_s' [-]	A_s [-]
1: Cadre A+C, mi-travée, en général:	0.40	2Ø20	3Ø14
2: Cadres A+C, sur appui, en général:	0.40	3Ø20	2Ø14
3: Cadres A+C, mi-travée, toit:	0.30	2Ø14	3Ø12
4: Cadres A+C, sur appui, toit:	0.30	3Ø14	2Ø12
5: Cadre B, mi-travée, en général:	0.45	2Ø22	3Ø22
6: Cadre B, sur appui, en général:	0.45	3Ø22	2Ø22
7: Cadre B, mi-travée, toit:	0.30	2Ø18	3Ø14
8: Cadre B, sur appui, toit:	0.30	3Ø18	2Ø14

Tableau 1: Hauteur et renforcement des traverses.

4.1.4 Zone de risque sismique, classe d'ouvrage et classe du sol de fondation

Figure 2.2. Example building RC members characteristics



4.1.3 Propriétés des matériaux

Le bâtiment est réalisé en béton BH300. Selon le cahier technique SIA 2018 Tableau 5 et la norme SIA 262 article 3.1.2.2.4, on peut admettre la résistance à la compression suivante:

$$f_{ck} = 1.25 \cdot 19.2 = 24\text{MPa} \quad (18)$$

$$f_{cd} = 24/1.5 = 16\text{MPa} \quad (19)$$

$$E_c = 24\text{GPa} \quad (20)$$

Le diagramme contrainte-déformation du béton correspond à la courbe 3 de la figure 8 à gauche. Le type de l'acier d'armature utilisé est du IIIA, et selon SIA 2018 Tableau 6 les propriétés suivantes peuvent être retenues:

$$f_{sk} = 450\text{MPa}\,,\; f_{tk} = 550\text{MPa}\,,\; (f_t/f_s)_k = 1.22 \quad (21)$$

$$f_{sd} = 390\text{MPa} \quad (22)$$

Classe de ductilité B, $\varepsilon_{uk} = 0.050$ (23)

Poids propres: $\gamma_k = 25\text{kN/m}^3$ (14)

Surcharges habitation: $q_k = 5\text{kN/m}^2$ (15)

Surcharges toit: $q_k = 1\text{kN/m}^2$ (16)

Charges utiles habitation: $q_k = 2\text{kN/m}^2\,,\; \psi_2 = 0.3$ (17)

Figure 2.3. Material parameters (ZSOIL input, given data) and loads

2.2 Seismic action and soil conditions

Figure 2.4 summarizes the parameters that determine seismic action according to SIA 261 [7].

4.1.4 Zone de risque sismique, classe d'ouvrage et classe du sol de fondation

Dans le cadre de cet exemple, on admet les conditions suivantes pour l'évaluation sismique:

Zone de risque sismique: Z3b, $a_{gd} = 1.6\text{m/s}^2$ (24)

Classe d'ouvrage: CO I, $\gamma_f = 1.0$ (25)

Classe du sol de fondation: C (26)

Figure 2.4. Seismic zone, importance factor and soil class

According to [7], soil class C corresponds to sand, gravel or moraine deposits with N_{SPT} between 15 and 50. For this study a value of $N_{SPT} = 40$ has been chosen. The following set of characteristic mean parameters are estimated:

$$\gamma = 20\ \text{kN/m}^3,\ E(\text{static}) = 80'000\ \text{kPa (see Figure 2.5)},\ c_k = 5\ \text{kPa},\ \phi_k = 35°$$

Figure 3.24: Performance of empirical correlations from Table 3.5 compared with typical values for the "static" modulus E_s from Table 3.4.

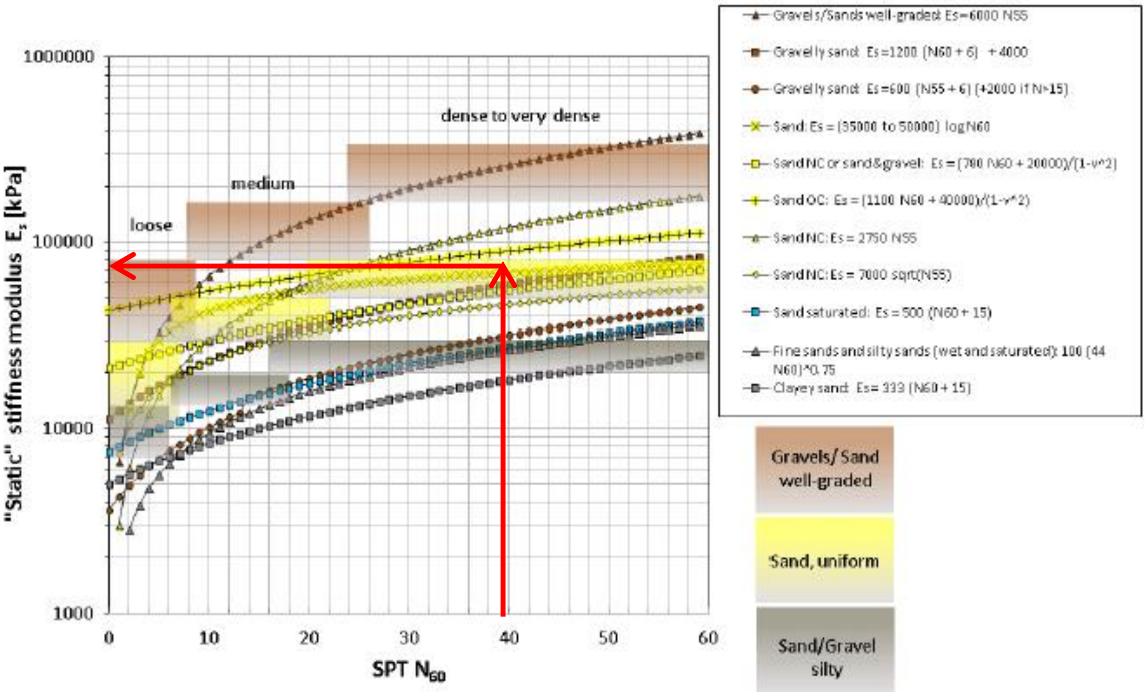


Figure 2.5. Soil's elastic modulus estimation [3]

2.3 Foundation assumptions

Figure 2.1 shows that the example building has a rigid basement floor. In order to study the influence of the foundation type, two configurations are scrutinized (see Figure 2.6):

- Continuous thick slab (h = 0.5 m, E = 20 GPa)
- Isolated shallow foundations under each column: b = 2 m, h = 0.5 m, E = 20 GPa

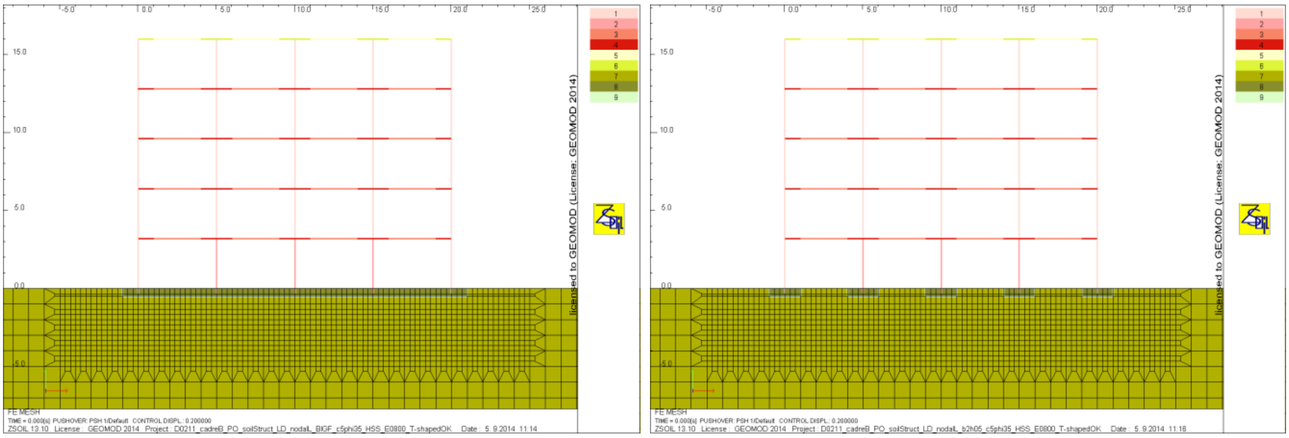
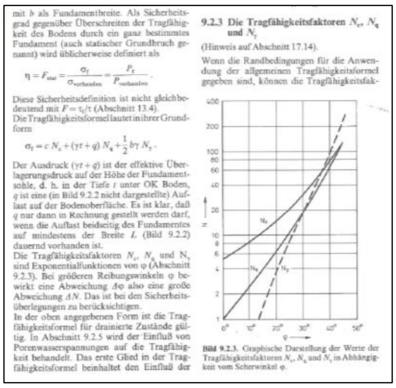


Figure 2.6. Foundation types: thick slab (left) and isolated shallow foundations (right)

Remarks:

- As this 2D FE model assumes plane strain conditions, the isolated shallow foundations case actually corresponds to a strip footing foundation type
- The bearing capacity of the isolated shallow foundations under static loading (Figure 2.3) has been verified (see Figure 2.7 and [8])



mit b als Fundamentbreite. Als Sicherheitsgrad gegenüber Überschreiten der Tragfähigkeit des Bodens durch ein ganz bestimmtes Fundament (auch statischer Grundbruch genannt) wird üblicherweise definiert als

$$\eta = F_{stat} = \frac{\sigma_f}{\sigma_{vorhanden}} = \frac{P_f}{P_{vorhanden}}.$$

Diese Sicherheitsdefinition ist nicht gleichbedeutend mit $F = \tau_f/\tau$ (Abschnitt 13.4).
Die Tragfähigkeitsformel lautet in ihrer Grundform

$$\sigma_f = c\,N_c + (\gamma t + q)\,N_q + \frac{1}{2} b \gamma\,N_\gamma.$$

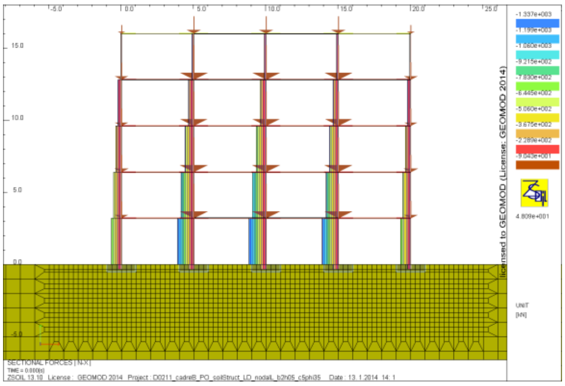
Der Ausdruck $(\gamma t + q)$ ist der effektive Überlagerungsdruck auf der Höhe der Fundamentsohle, d. h. in der Tiefe t unter OK Boden, q ist eine (in Bild 9.2.2 nicht dargestellte) Auflast auf der Bodenoberfläche. Es ist klar, daß q nur dann in Rechnung gestellt werden darf, wenn die Auflast beidseitig des Fundamentes auf mindestens der Breite L (Bild 9.2.2) dauernd vorhanden ist.
Die Tragfähigkeitsfaktoren N_c, N_q und N_γ sind Exponentialfunktionen von φ (Abschnitt 9.2.3). Bei größeren Reibungswinkeln φ bewirkt eine Abweichung $\Delta\varphi$ also eine große Abweichung ΔN. Das ist bei den Sicherheitsüberlegungen zu berücksichtigen.
In der oben angegebenen Form ist die Tragfähigkeitsformel für drainierte Zustände gültig. In Abschnitt 9.2.5 wird der Einfluß von Porenwasserspannungen auf die Tragfähigkeit behandelt. Das erste Glied in der Tragfähigkeitsformel beinhaltet den Einfluß der

9.2.3 Die Tragfähigkeitsfaktoren N_c, N_q und N_γ

(Hinweis auf Abschnitt 17.14).

Wenn die Randbedingungen für die Anwendung der allgemeinen Tragfähigkeitsformel gegeben sind, können die Tragfähigkeitsfak-

Bild 9.2.3. Graphische Darstellung der Werte der Tragfähigkeitsfaktoren N_c, N_q und N_γ in Abhängigkeit vom Scherwinkel φ.



Footing bearing capacity

Say b = 2 m, t = 0.5 m
σ_{fail} = 5 kPa x 46 + 10 kPa x 33 + 20 kPa x 40 = 1360 kPa

Most loaded column: 1340 kN (service) x 1.5 => Nd = 2000 kN

σ_d = 2000 kN / 2 m / 1 m' = 1000 kPa

OK !

Figure 2.7 Bearing capacity of isolated shallow foundations (from [8])

3. 2D pushover analyses of frame B

Remark: for the analyses in this report, it is assumed that a brittle shear failure of the frame elements can be excluded.

3.1 Structural only analysis

Figure 3.1 illustrates the 2D structural FE model corresponding to frame B. Column and slab sections are introduced in the model according to Figure 2.2 and 2.3.

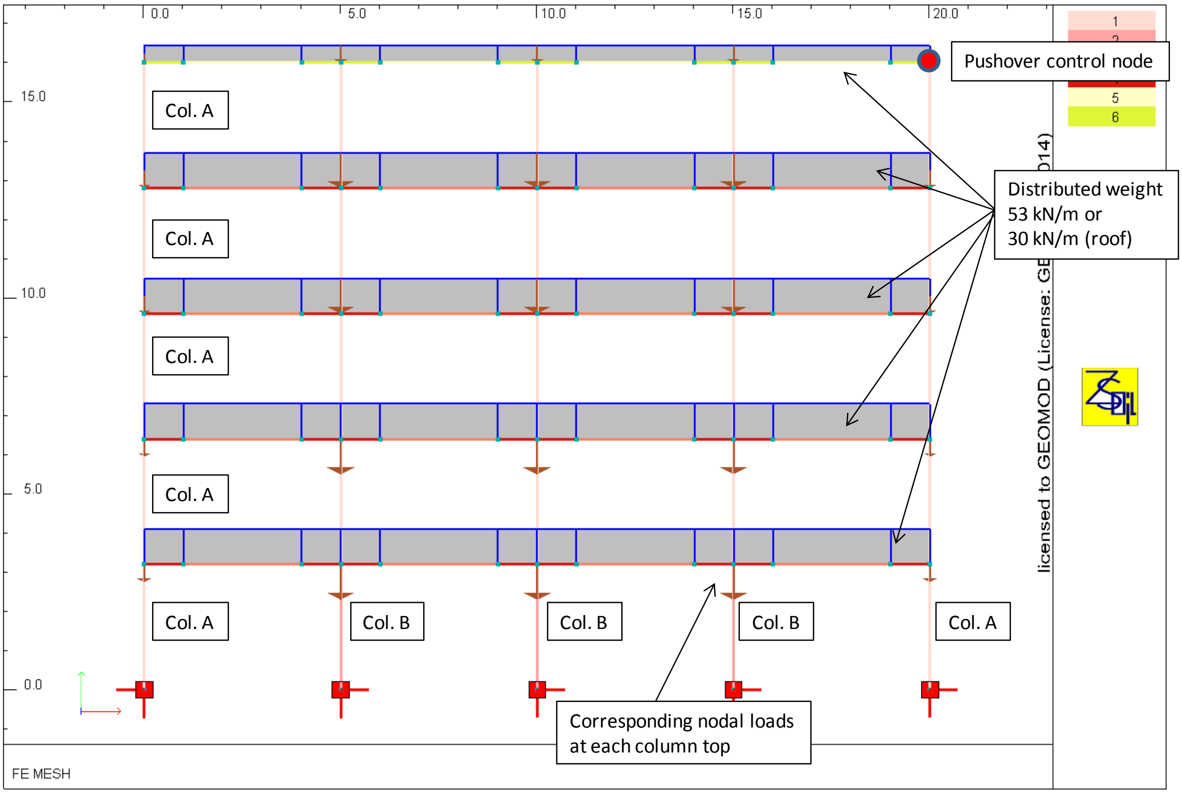


Figure 3.1 2D structural FE model

An initial state is first performed, applying gravity loads (nodal loads corresponding to the distributed weight at each column's top[2]), and then a classical pushover analysis is conducted, using a uniform force pattern. The resulting capacity curve is given in Figure 3.2. Total shear at the base reaches about 900 kN, which is in good agreement with the total horizontal force given in [1], Fig. 18.

[2] it is thus assumed for the sake of simplicity that bending moments in the structural elements are 0 for the gravity load case (which in reality is not the case, especially for the beam elements).

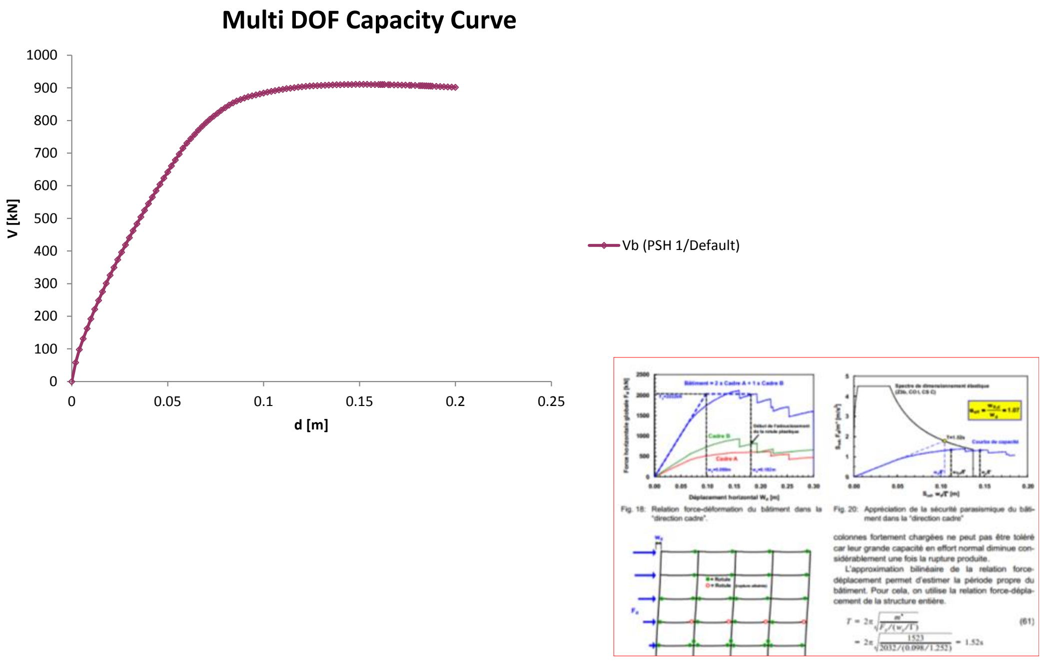


Figure 3.2 Structural only pushover analysis, frame B. Capacity curve and comparison with [1]

Figure 3.3 illustrates the demand spectrum, both in conventional and ADRS formats, corresponding to Eurocode 8 type 1 spectrum: here a ground acceleration of 0.16 g is considered, with importance factor = 1.0 and ground type = C.

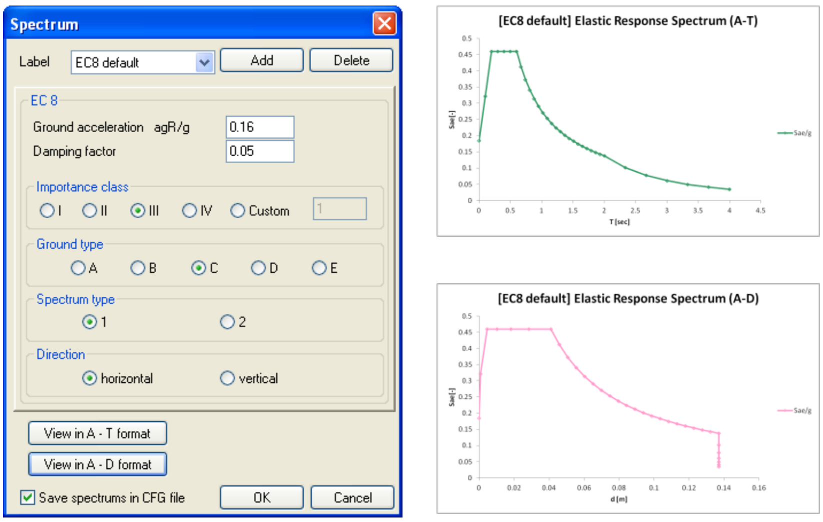


Figure 3.3 Seismic demand

In Figure 3.4, the 1DOF capacity curve (blue) and the elastic demand spectrum (pink) are plotted in the same graph, and yield the 1DOF target displacement. For this, according to N2 theory [10], the capacity curve is bi-linearized (brown line) and the demand spectrum is reduced (nonlinear red spectrum). Figure 3.5 summarizes the pushover analysis and yields the MDOF target displacement (or displacement demand): 8.8 cm.

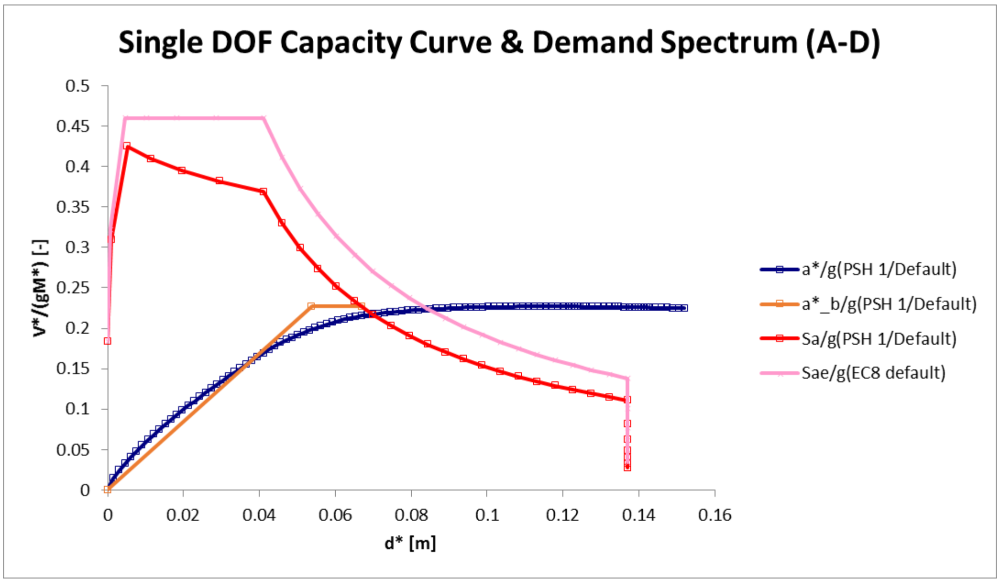


Figure 3.4 Structural only pushover analysis, frame B. Capacity and demand spectra

	Pushover analysis report	
Item	Unit	PSH 1/Default
MDOF Free vibr. period........T	[s]	0.596785
SDOF Free vibr. period.......T*	[s]	0.976961
SDOF equivalent mass.........M*	[kg]	311299
Mass participation factor Gamma	-	1.31488
Bilinear yield force value..Fy*	[kN]	692.6537
Bilinear displ. at yield....Dy*	[m]	0.053794
Target displacement.........Dm*	[m]	0.067052
SDOF displacement demand....Dt*	[m]	0.06706
Energy......................Em*	[kN*m]	27.81364
Reduction factor.............qu	-	1.246607
Demand ductility factor......mi	-	1.246607
Capacity ductility factor...miC	-	1.246464
MDOF displacement demand.....Dt	[m]	0.088176

Figure 3.5 Structural only pushover analysis, frame B.
Pushover analysis summary and extraction of target displacement

Figure 3.6 illustrates the deformed mesh of the structure corresponding to the top roof target displacement of 8.8 cm, and corresponding bending moments are given in Figure 3.7 (|max|: 482 kNm).

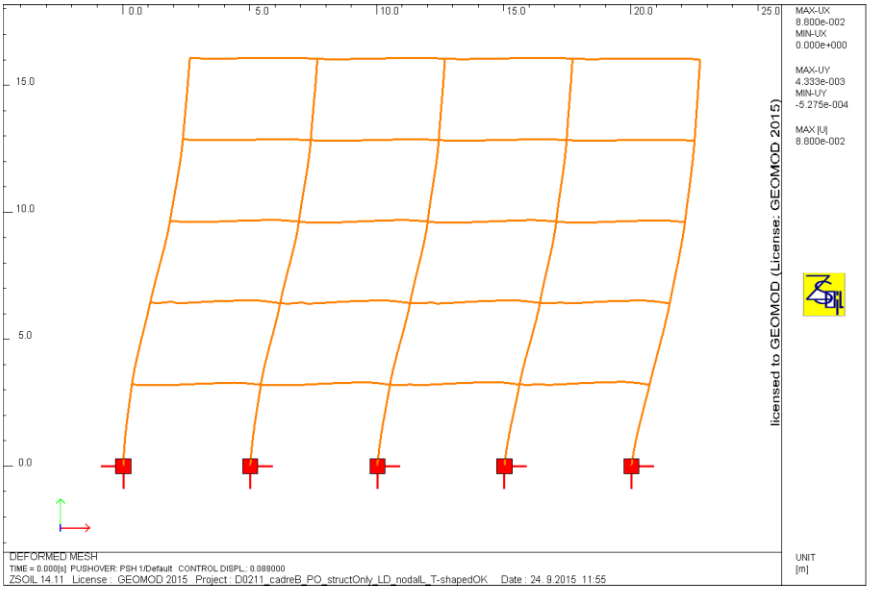


Figure 3.6 Structural only pushover analysis, frame B

Deformed mesh corresponding to target displacement dt = 8.8 cm

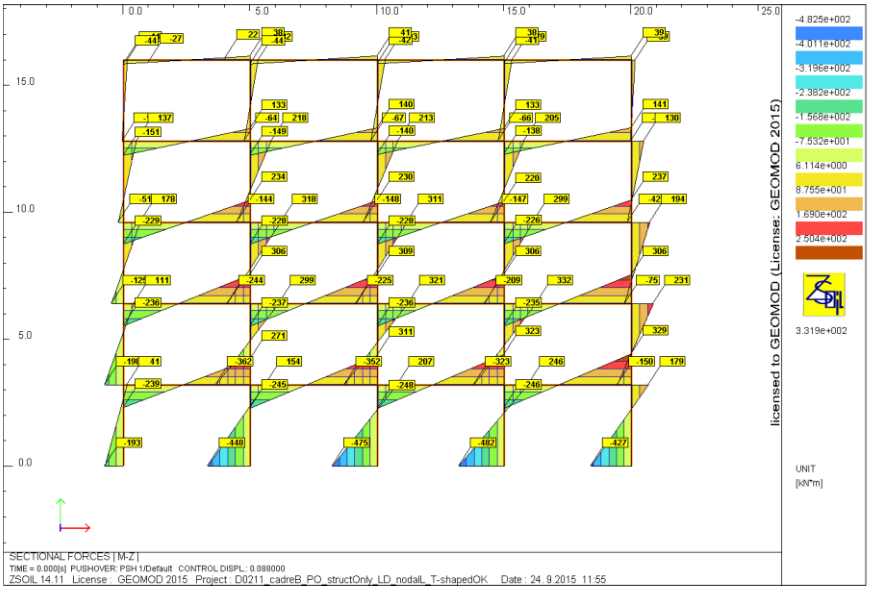


Figure 3.7 Structural only pushover analysis, frame B

Bending moment distribution corresponding to target displacement dt = 8.8 cm

The displacements, chord rotations, inter-storey drifts derived from Figure 3.6 or internal forces represented in Figure 3.7 should be compared with limit values of resistance or deformation in order to assess the seismic safety of the structure. A specific chapter of this report is dedicated to this point (see chapter 4).

3.2 Structure + soil analysis

3.2.1 Shallow isolated foundations

Figure 3.8 illustrates the 2D structure + soil FE model corresponding to frame B on isolated foundations (see also Fig. 2.6, right). In the reference case, the HSS constitutive law is used to model the soil, with soil C characteristic parameters according to §2.2. As for the structural only case, an initial state is first performed, applying gravity loads (nodal loads corresponding to the distributed weight of the structure at each column's top + distributed soil's unit weight all over the soil domain), and then a pushover analysis is conducted, using once again a uniform force pattern applied to the structure only (multiplier for the soil's mass matrix = 0).

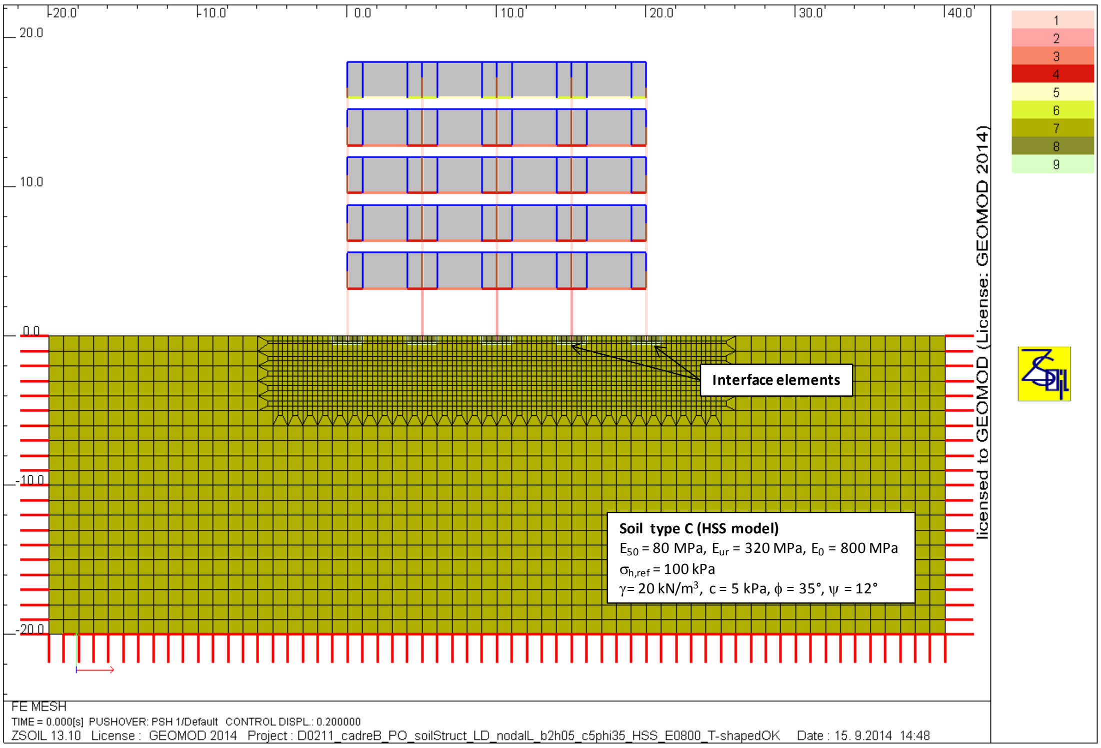


Figure 3.8 2D structure + soil FE model (shallow isolated foundations)

The resulting capacity curve is given in Figure 3.9 and in Figure 3.10, the 1DOF capacity and demand spectra are plotted in the same graph, yielding the 1DOF target displacement. Figure 3.11 summarizes the pushover analysis and yields the MDOF target displacement (or displacement demand): 12.3 cm (higher than the 8.8 cm obtained in the structural only case, due to the foundation's flexibility and the rocking's phenomenon).

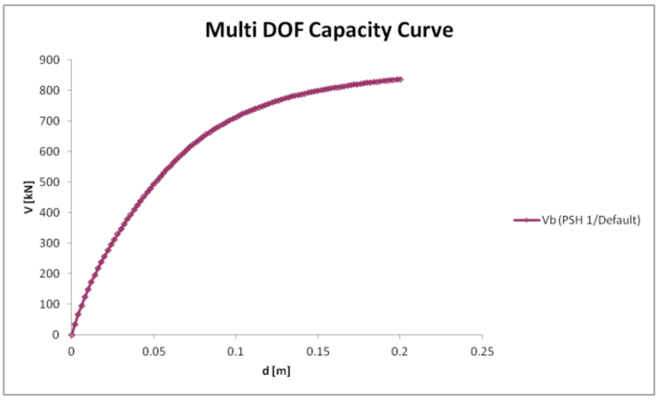


Figure 3.9 Structure + soil pushover analysis, frame B (shallow isolated foundations)
Capacity curve

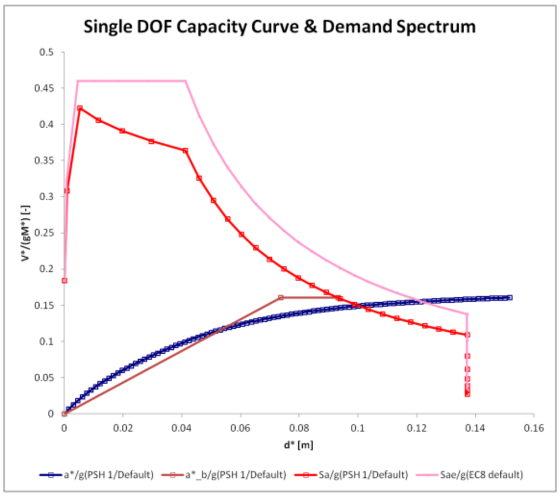


Figure 3.10 Structure + soil pushover analysis, frame B (shallow isolated foundations)
Capacity and demand spectra

	Pushover analysis report	
Item	Unit	PSH 1/Default
MDOF Free vibr. period........T	[s]	0.840514
SDOF Free vibr. period.......T*	[s]	1.359520061
SDOF equivalent mass.........M*	[kg]	402954
Mass participation factor Gamma	-	1.31941
Bilinear yield force value..Fy*	[kN]	634.3961679
Bilinear displ. at yield....Dy*	[m]	0.073708332
Target displacement.........Dm*	[m]	0.093231987
SDOF displacement demand....Dt*	[m]	0.09324022
Energy.....................Em*	[kN*m]	35.76587353
Reduction factor............qu	-	1.264988878
Demand ductility factor......mi	-	1.264988878
Capacity ductility factor...miC	-	1.264877177
MDOF displacement demand.....Dt	[m]	0.123022079

Figure 3.11 Structure + soil pushover analysis, frame B (shallow isolated foundations)
Pushover analysis summary and extraction of target displacement

Figure 3.12 illustrates the deformed mesh of the structure corresponding to the top roof displacement of 12.3 cm, as well as the stress level in the soil (SL = 1 in brown corresponds to a plastic state). Corresponding bending moments are given in Figure 3.13 (max: -258 kNm at the bottom of the lower columns, and +316 kNm at the top of the lower columns): soil plastification and associated rocking phenomenon are found to reduce significantly the maximal bending moment found in the structural only case (-482 kNm).

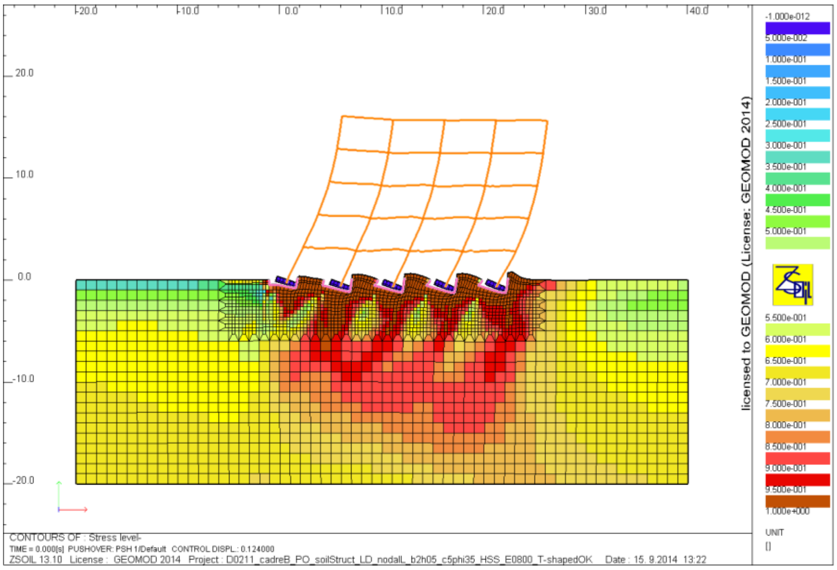


Figure 3.12 Structure + soil pushover analysis, frame B (shallow isolated foundations)
Deformed mesh (x 50) and stress level corresponding to target displacement dt = 12.3 cm

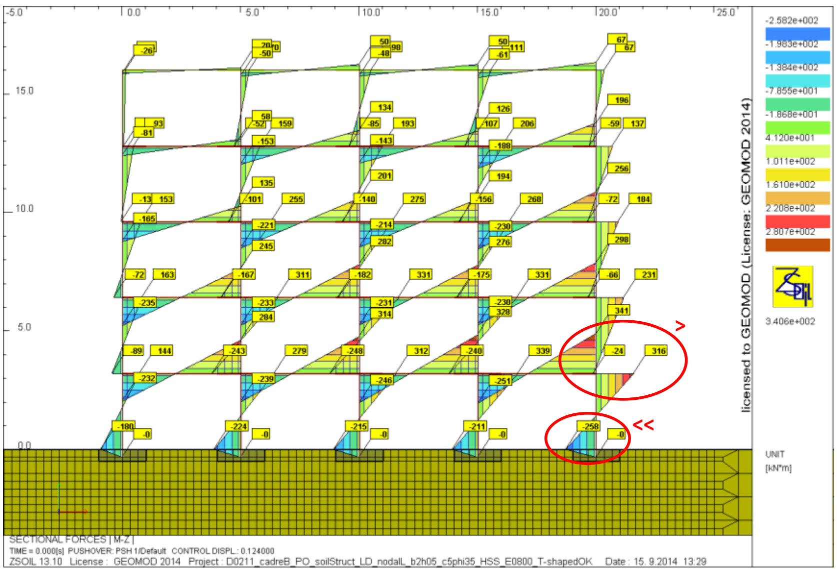


Figure 3.13 Structure + soil pushover analysis, frame B (shallow isolated foundations)
Bending moment distribution corresponding to dt = 12.4 cm

3.2.2 Thick slab foundation

Figure 3.14 illustrates the 2D structure + soil FE model corresponding to frame B on thick slab foundation (see also Fig. 2.6, left).

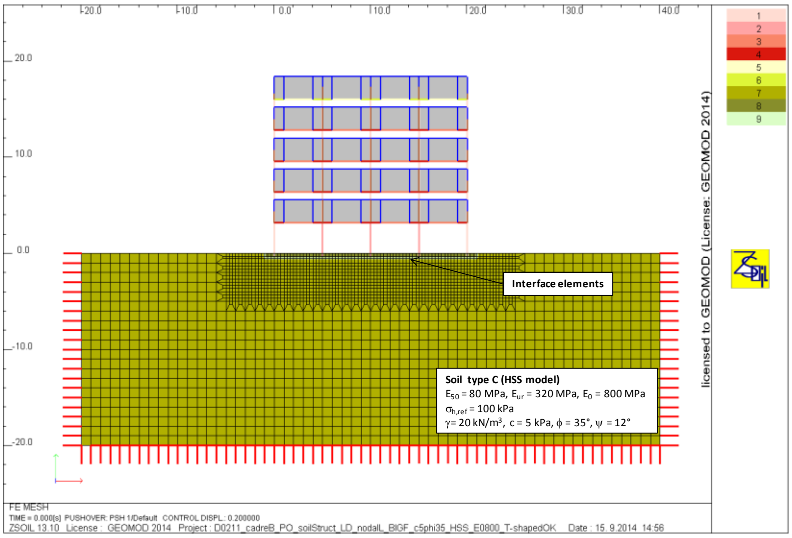


Figure 3.14 2D structure + soil FE model (thick slab foundation)

The resulting capacity curve is given in Figure 3.15 and in Figure 3.16, the 1DOF capacity and demand spectra are plotted in the same graph, yielding the 1DOF target displacement. Figure 3.17 summarizes the pushover analysis and yields the MDOF target displacement (or displacement demand): 11.2 cm.

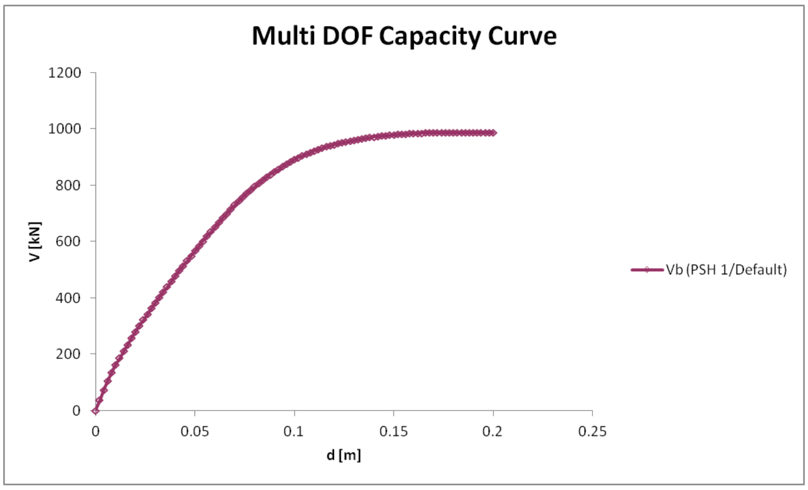


Figure 3.15 Structure + soil pushover analysis, frame B (thick slab foundation)

Capacity curve

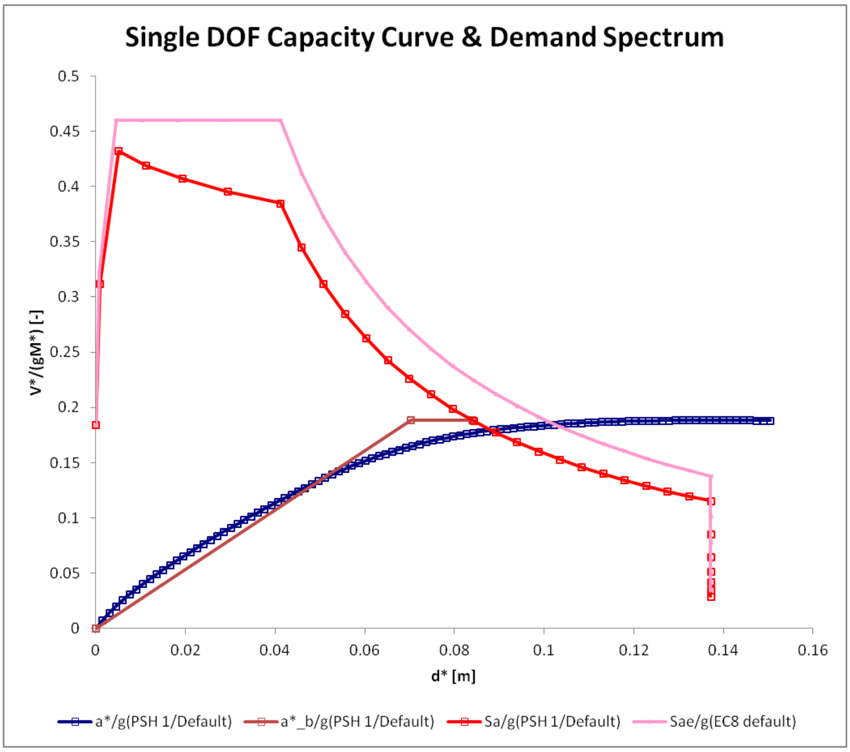


Figure 3.16 Structure + soil pushover analysis, frame B (thick slab foundation)
Capacity and demand spectra

Pushover analysis report

Item	Unit	PSH 1/Default
MDOF Free vibr. period........T	[s]	0.836596
SDOF Free vibr. period.......T*	[s]	1.225429744
SDOF equivalent mass.........M*	[kg]	401745
Mass participation factor Gamma	-	1.33014
Bilinear yield force value..Fy*	[kN]	742.5613829
Bilinear displ. at yield....Dy*	[m]	0.070307067
Target displacement.........Dm*	[m]	0.084025659
SDOF displacement demand....Dt*	[m]	0.084032062
Energy.....................Em*	[kN*m]	36.29055307
Reduction factor.............qu	-	1.195215021
Demand ductility factor......mi	-	1.195215021
Capacity ductility factor...miC	-	1.195123946
MDOF displacement demand.....Dt	[m]	0.111774407

Figure 3.17 Structure + soil pushover analysis, frame B (thick slab foundation)
Pushover analysis summary and extraction of target displacement

Figure 3.18 illustrates the deformed mesh of the structure corresponding to the top roof displacement of 11.2 cm, as well as the stress level in the soil. Corresponding bending moments are given in Figure 3.19 (max: -482 kNm at the bottom of the lower columns): this time, the thick slab's thickness prevents the rocking phenomenon to happen and the maximal bending moments in both the structural only and the structure + soil cases are found to be equal.

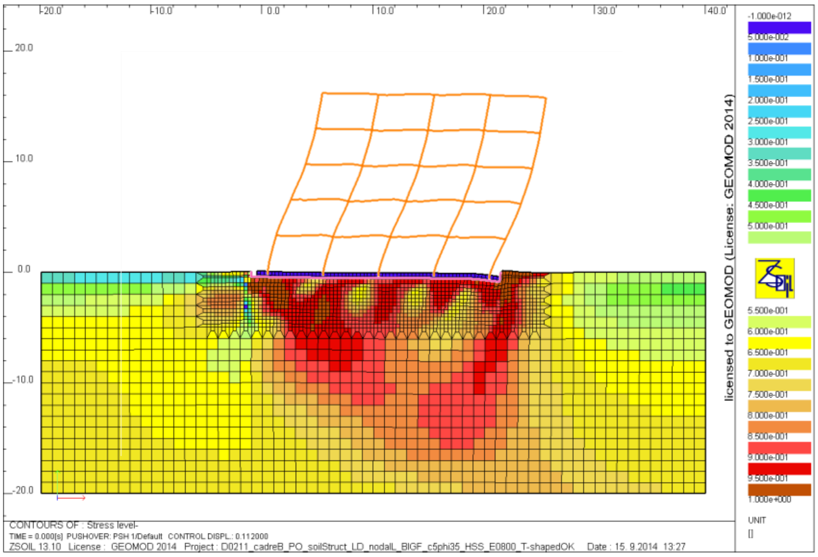


Figure 3.18 Structure + soil pushover analysis, frame B (thick slab foundation)
Deformed mesh (x 50) and stress level in soil corresponding to dt = 11.2 cm

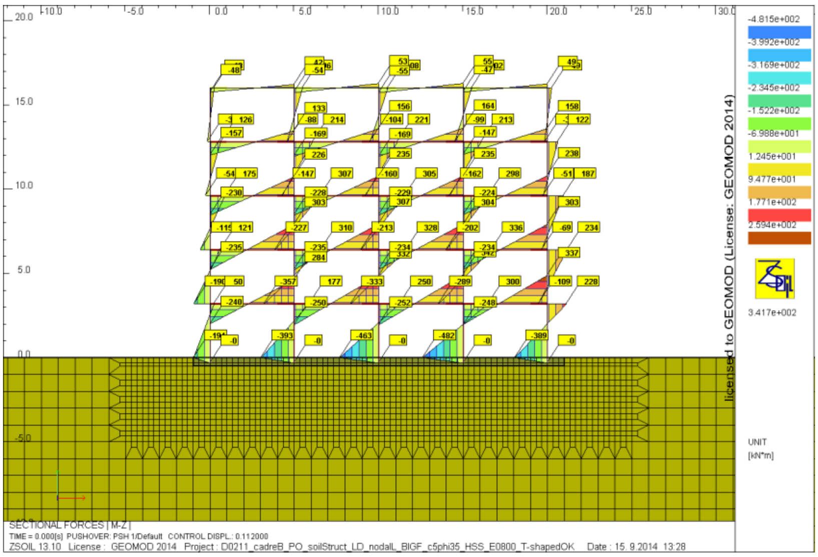


Figure 3.19 Structure + soil pushover analysis, frame B (thick slab foundation)
Bending moment distribution corresponding to target displacement dt = 11.2 cm

3.2.3 Parametric studies

Different parametric studies have been conducted on the soil + structure case, with isolated foundations:

- Use of Mohr-Coulomb constitutive model instead of HSS, with ϕ(M-C) = ϕ(HSS) = 35° and E(M-C) = E_0(HSS) = 800 MPa
- Within the Mohr-Coulomb model, use E = 120 MPa (corresponding to somewhere between loading and unloading moduli) instead of 800 MPa
- Within the Mohr-Coulomb model, use ϕ = 33° instead of 35°
- Change seismic demand (ground acceleration of 0.10g instead of 0.16g)

Figure 3.20 gives the capacity curves corresponding to the different computed cases.

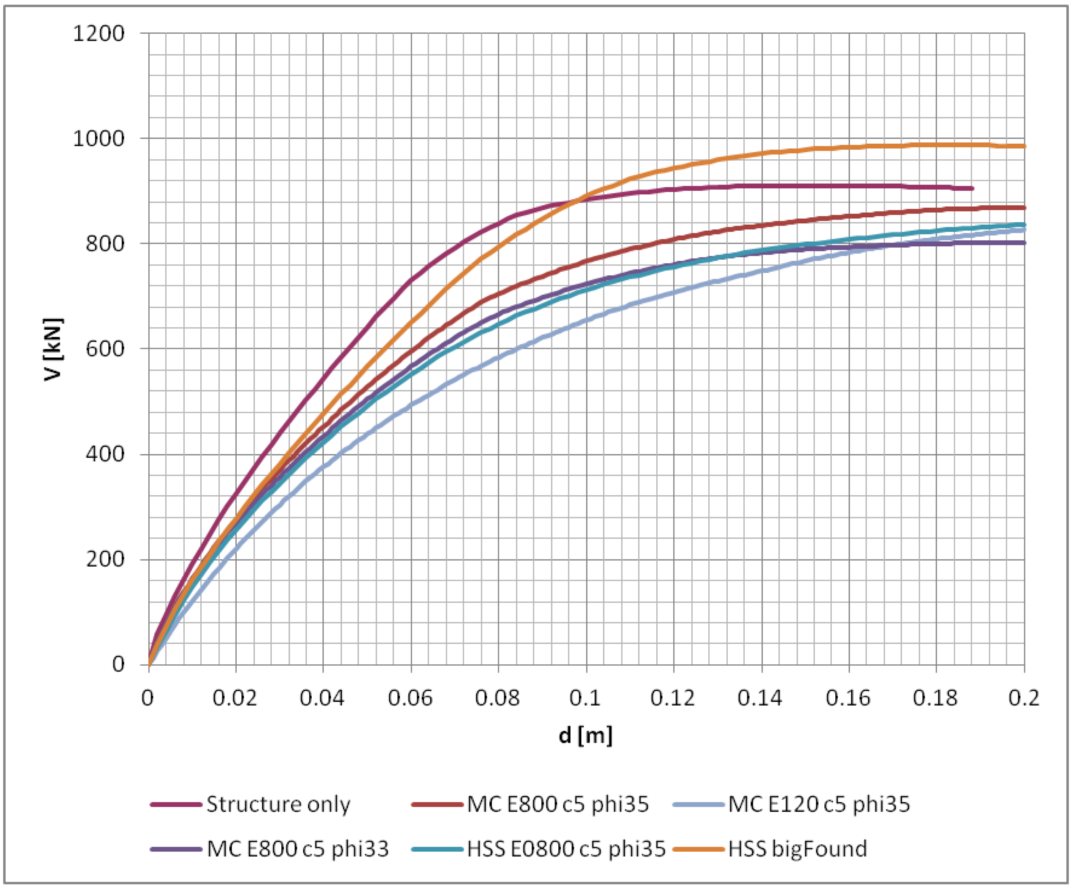


Figure 3.20 Pushover analysis of frame B
Comparison of capacity curves, parametric study

Tables 3.1 (a_g = 0.16g) and 3.2 (a_g = 0.10g) summarize the pushover results for the aforementioned studies.

Item	Unit	Struct only	MC E800 phi35	MC E120 phi35	MC E800 phi33	HSS E0800 phi35	HSS E0800 phi35 big foundation
MDOF Free vibr. period........T	[s]	0.596785	0.800796	1.07811	0.802771	0.840514	0.836596
SDOF Free vibr. period.......T*	[s]	0.976960966	1.271963381	1.607603057	1.294799843	1.359520061	1.225429744
SDOF equivalent mass.........M*	[kg]	311299	394034	464753	394251	402954	401745
Mass participation factor Gamma	-	1.31488	1.31923	1.42099	1.31906	1.31941	1.33014
Bilinear yield force value..Fy*	[kN]	692.6537468	658.7684476	581.8733938	608.3042528	634.3961679	742.5613829
Bilinear displ. at yield....Dy*	[m]	0.053793915	0.068515459	0.081960426	0.065522972	0.073708332	0.070307067
Target displacement.........Dm*	[m]	0.067052149	0.08726019	0.110229905	0.088783015	0.093231987	0.084025659
SDOF displacement demand....Dt*	[m]	0.067059847	0.087268925	0.110265418	0.088789118	0.09324022	0.084032062
Energy......................Em*	[kN*m]	27.813644	34.91634853	40.2945535	34.07813426	35.76587353	36.29055307
Reduction factor............qu	-	1.246606565	1.273711457	1.345349493	1.355083803	1.264988878	1.195215021
Demand ductility factor......mi	-	1.246606565	1.273711457	1.345349493	1.355083803	1.264988878	1.195215021
Capacity ductility factor...miC	-	1.246463461	1.273583962	1.344916207	1.354990655	1.264877177	1.195123946
MDOF displacement demand.....Dt	[cm]	8.8	11.5	15.7	11.7	12.3	11.2
Mmax lower columns at Dt	[kNm]	482	344	411	345	337	482
Gain on Mmax	[%]	-	-28.8	-14.9	-28.6	-30.2	-0.2

Table 3.1 2D Pushover analysis of frame B. Parametric study, for ag = 0.16 g

Item	Unit	Struct only	MC E800 phi35	MC E120 phi35	MC E800 phi33	HSS E0800 phi35	HSS E0800 phi35 big foundation
MDOF Free vibr. period........T	[s]	0.596785	0.800796	1.07811	0.802771	0.840514	0.836596
SDOF Free vibr. period.......T*	[s]	0.991272432	1.303588674	1.622768732	1.315012813	1.397791253	1.241586529
SDOF equivalent mass.........M*	[kg]	311299	394034	464753	394251	402954	401745
Mass participation factor Gamma	-	1.31488	1.31923	1.42099	1.31906	1.31941	1.33014
Bilinear yield force value..Fy*	[kN]	692.6537468	658.7684476	581.8733938	608.3042528	634.3961679	742.5613829
Bilinear displ. at yield....Dy*	[m]	0.055381509	0.071964865	0.083514103	0.067584683	0.077916598	0.072173227
Target displacement.........Dm*	[m]	0.152105135	0.151603587	0.140746944	0.151623126	0.151582904	0.150360112
SDOF displacement demand....Dt*	[m]	0.042527486	0.055865578	0.069565869	0.05637374	0.059930249	0.053223862
Energy......................Em*	[kN*m]	86.17608697	76.16756855	57.59958461	71.67696703	71.44861827	84.85508727
Reduction factor............qu	-	1	1	1	1	1	1
Demand ductility factor......mi	-	3.576631238	2.713720882	2.023218375	2.689605581	2.529322108	2.825050759
Capacity ductility factor...miC	-	2.746496776	2.106633393	1.685307491	2.243453971	1.945450767	2.083322554
MDOF displacement demand.....Dt	[m]	5.6	7.4	9.9	7.4	7.9	7.1
Mmax lower columns at Dt	[kNm]	421	316	330	311	319	408
Gain on Mmax	[%]	-	-24.9	-21.6	-26.1	-24.2	-3.1

Table 3.2 2D Pushover analysis of frame B. Parametric study, for ag = 0.10 g

Reading the results, it is difficult to be conclusive on whether to use min. or max. characteristic values of strength and stiffness soil parameters in order to be on the safe side. Smaller values of E, c and ϕ lead to a bigger rocking phenomenon and hence lower the bending moment at the bottom of the lower columns, but Table 3.3 shows we should be careful about the value at the top of these columns, as the soil's, foundation's and floor's stiffness all play a role in the column bending moment distribution.

M(Dt) [kN/m]	Structural only	HSS thick slab $E_0 = 800$ MPa $\phi = 35°$	HSS isolated $E_0 = 800$ MPa $\phi = 35°$	M-C isolated $E = 800$ MPa $\phi = 35°$	M-C isolated $E = 800$ MPa $\phi = 33°$	M-C isolated $E = 120$ MPa $\phi = 35°$
lower columns max. top	246	300	339	312	338	411
lower columns max. bottom	482	482	258	331	282	173

Table 3.3 Moment in lower columns (at top/bottom) corresponding to target displacement

Conclusions:

- Sensibility analyses with characteristic soil parameters taken from their minimal to their maximal values should be performed in order to remain on the safe side.
- Neglecting SSI for the design of frame structures on isolated footings is not on the safe side as some structural elements could be potentially under-designed. For the seismic verifications of such structures, the critical structural elements could be misidentified and structural measures implemented at the wrong place.

4. Considerations on seismic assessment with/without soil

4.1 Definitions

According to [1] (introduction and §4.2.3) or [11] (§9.1), the seismic verification of a structure is based on the determinant compliance factor α_{eff} for all structural members (including foundation elements). The compliance factor can be computed:

- Either force based by dividing the seismic action for which the design resistance of the determinant structural member is reached by the verification value of the seismic action (in some simple cases the force based compliance factor can be obtained by dividing the design resistance R_d by the effect of the considered actions E_d)

- Or displacement based by dividing the displacement capacity of the structure $w_{R,d}$ by the displacement demand due to the seismic action w_d (the target displacement obtained by the pushover method in our case). The displacement capacity $w_{R,d}$ is computed based on the ultimate displacement capacity w_u divided by a partial factor γ_D as $w_{R,d} = w_u/\gamma_D$. The partial factor γ_D has a value of 1.3 for reinforced concrete structures. The displacement based compliance factor can be computed also for each structural member as the deformation capacity of the structural member divided by the deformation demand for the structural member. A displacement based approach is only possible if brittle failure modes can be excluded.

Remarks:

- Determining a displacement based compliance factor requires a careful analysis to determine at which point the structure has reached its ultimate capacity (see also [1]).

- Though theoretical failure loads can be retrieved through numerical modeling for simple frames (see Figures 4.1 and 4.2), the "numerical" global displacement capacity w_u of a complex structure can be difficult to estimate.

- Typically, for RC structures, chord rotations $\theta_{R,d}$ and θ_d are compared **at the member's** (column, wall, beam) **level** (see [11], §6.2.1).

In the following chapters, two different approaches are briefly developed to assess the influence of soil structure interaction on the compliance factor of the example building based on the previous considerations:

- Force based, by using the elastic part of the push-over capacity spectrum, similarly to a linear replacement forces method (§4.2)
- Deformation based, by comparison of column's chord rotations (§4.3)

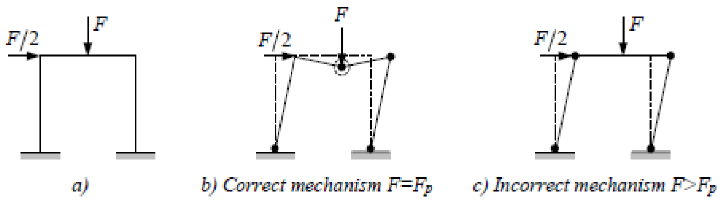


Fig. 3.2: Simple example of a mechanism.

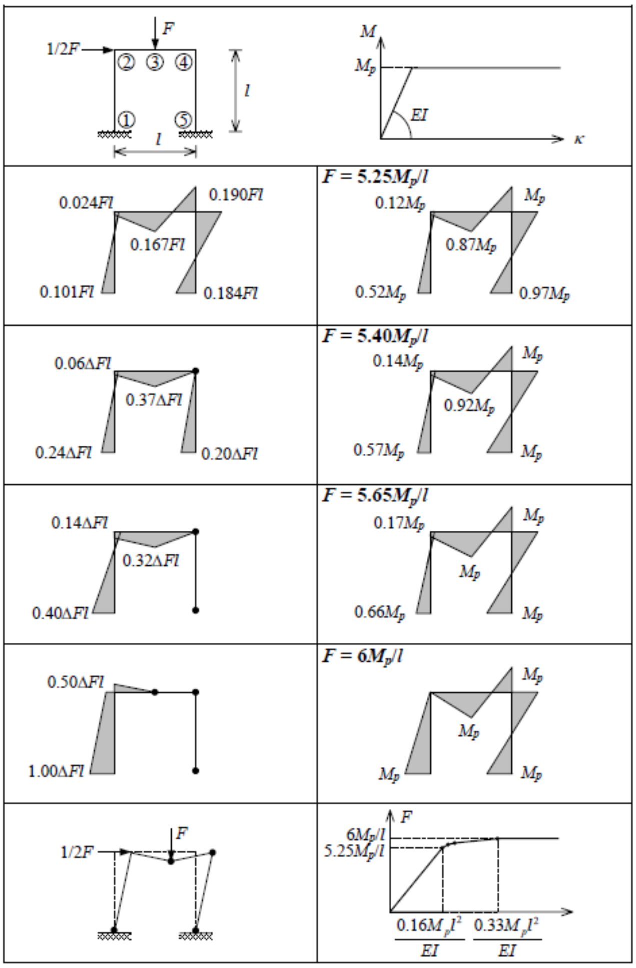


Fig. 2.14: Elastic-plastic analysis of a portal frame.

Figure 4.1. Theoretical elastic-plastic analysis of a portal frame (from [13])

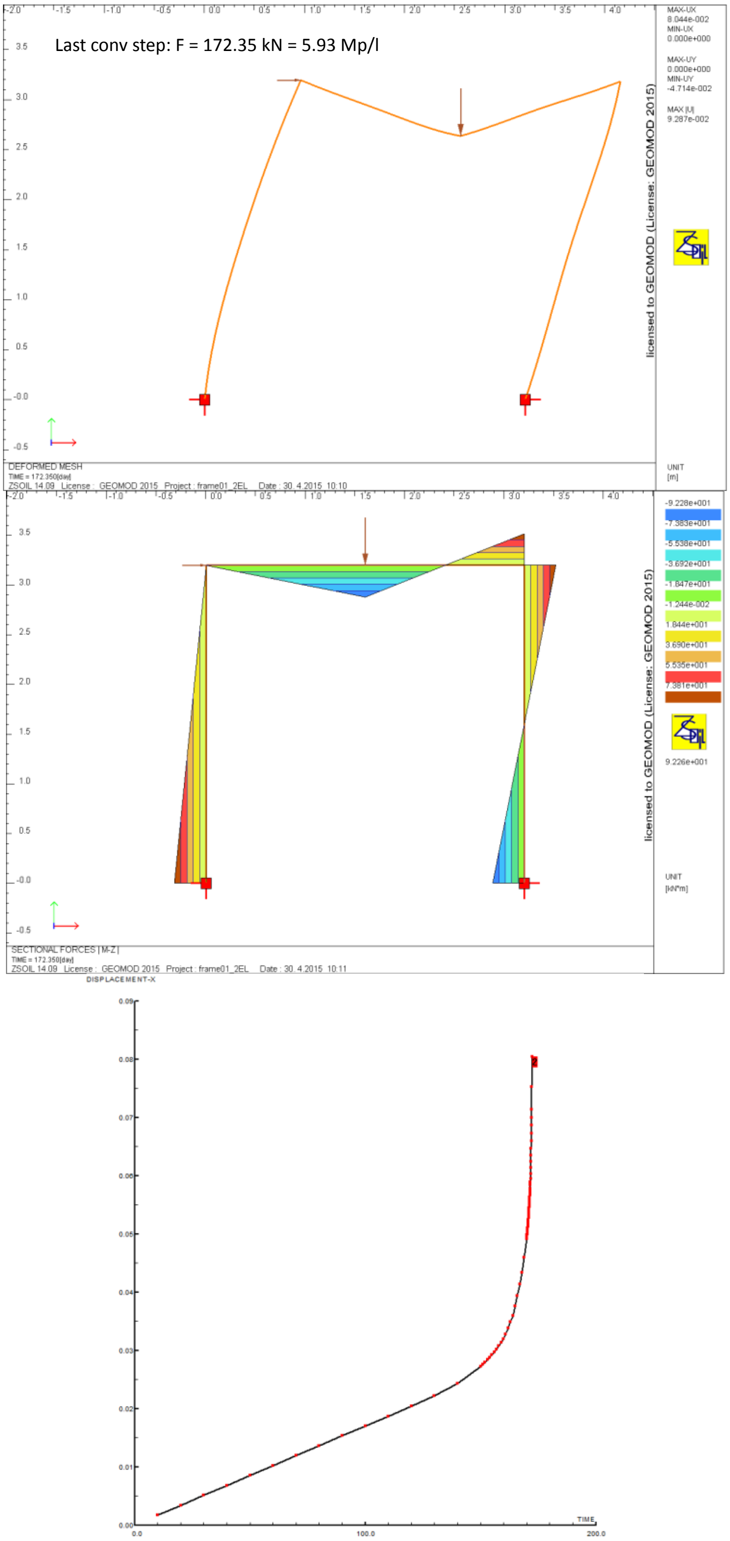


Figure 4.2. Elastic-plastic analysis of a portal frame with ZSOIL

4.2 Use of the elastic part of the push-over capacity spectrum

According to [12], the following method could be used in order to evaluate the impact of taking soil into account in a classical force based linear replacement forces method:

- Choose a reference point in the **elastic part** of the pushover capacity spectrum, and store corresponding reference internal efforts E_{ref}
- Compute a "global" scaling factor: $f = S_{ae,demand} / (S_{ae,ref})$ (see Figure 4.3)
- Compute the force-based verification internal efforts E_d by multiplying E_{ref} by the scaling factor f and then dividing the result by the appropriate behavior factor q.
- Compute α_{eff} force based

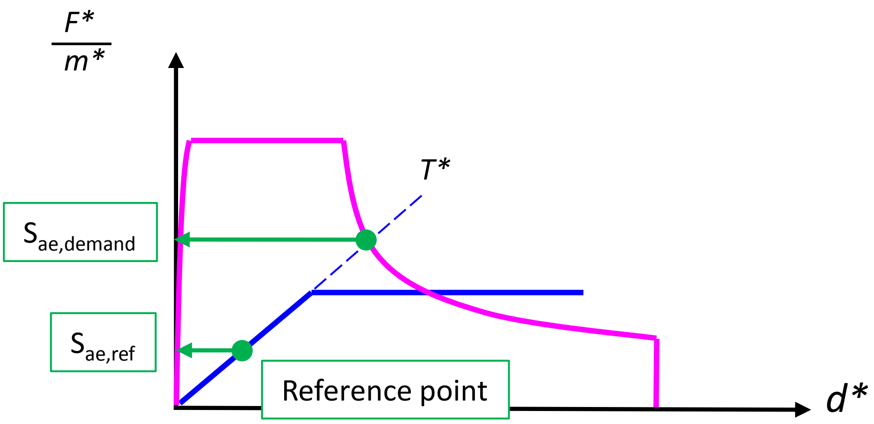


Figure 4.3. Definition of factor f

Applying this method to the building described in sections 3.1 (structure only, see Figures 4.4 and 4.5) and 3.2.1 (structure + soil on shallow isolated foundations, see Figures 4.6 and 4.7) leads to the design bending moments comparison given in Table 4.1.

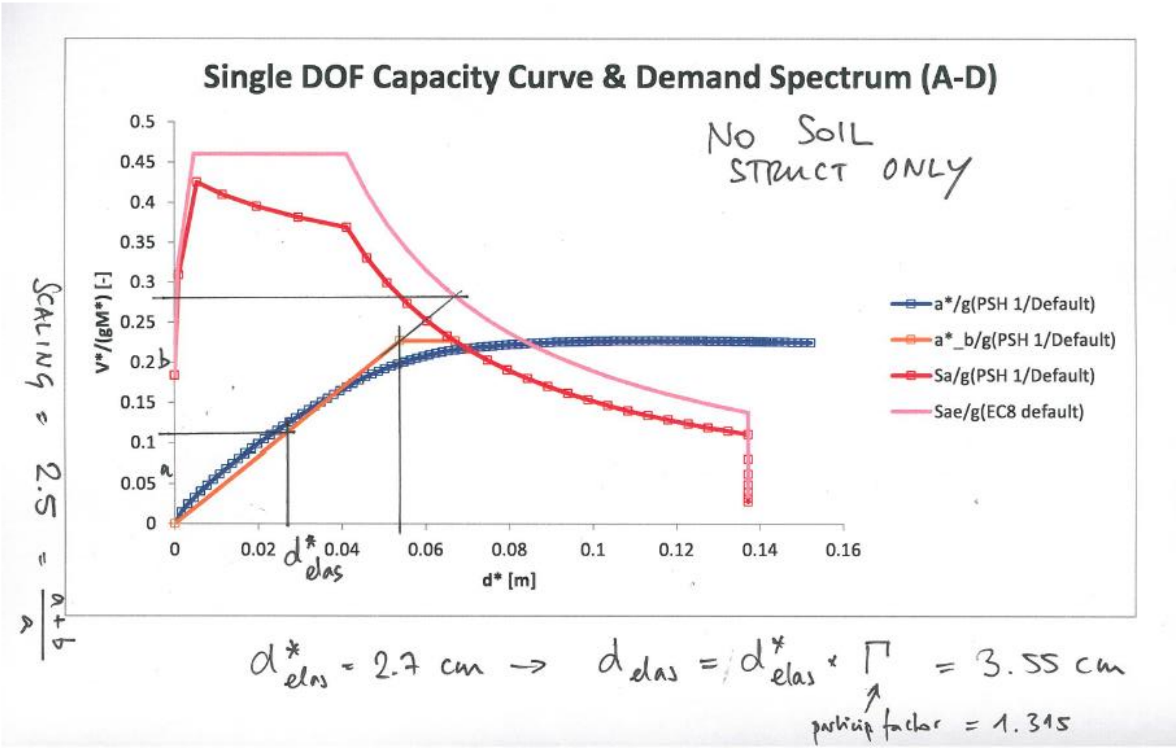


Figure 4.4. Definition of factor f and corresponding global displacement (structure only)

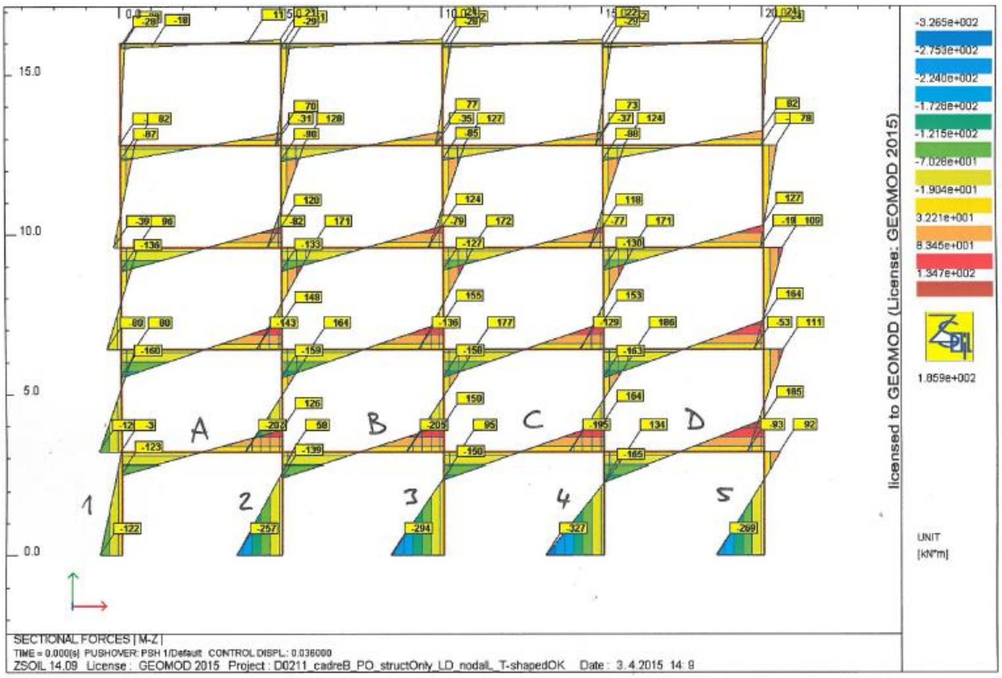


Figure 4.5a. Corresponding reference bending moments (structure only)

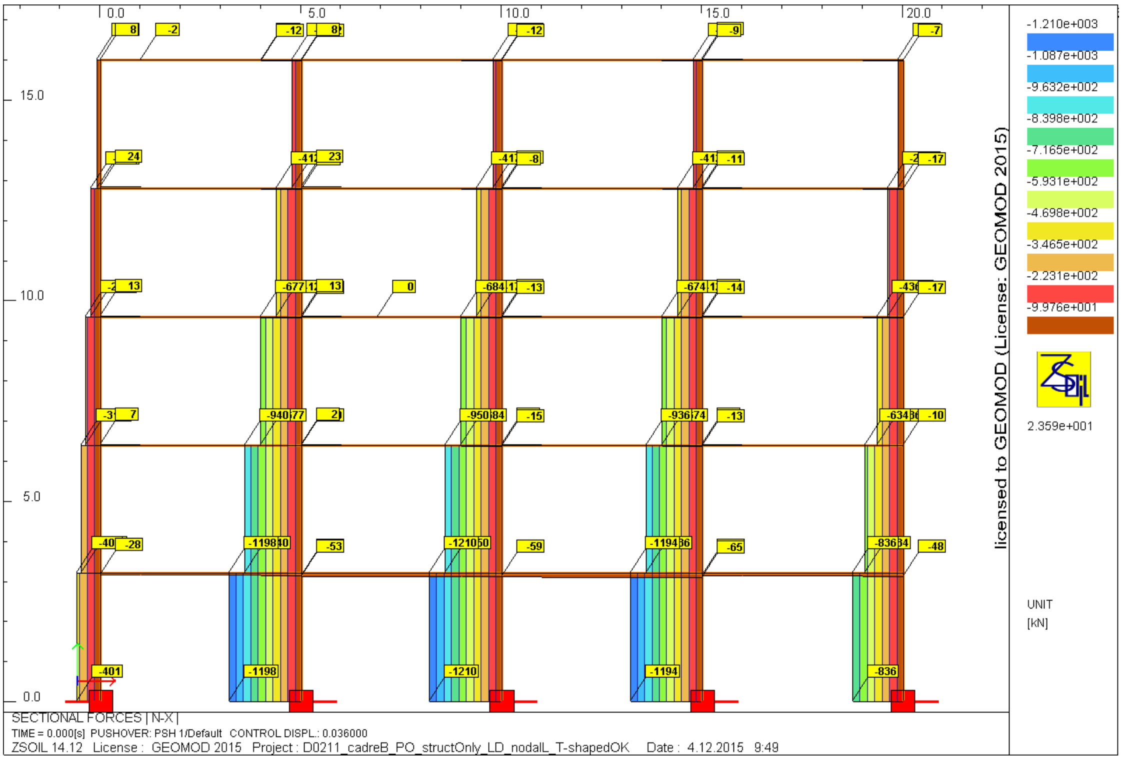


Figure 4.5b. Corresponding reference normal forces (structure only)

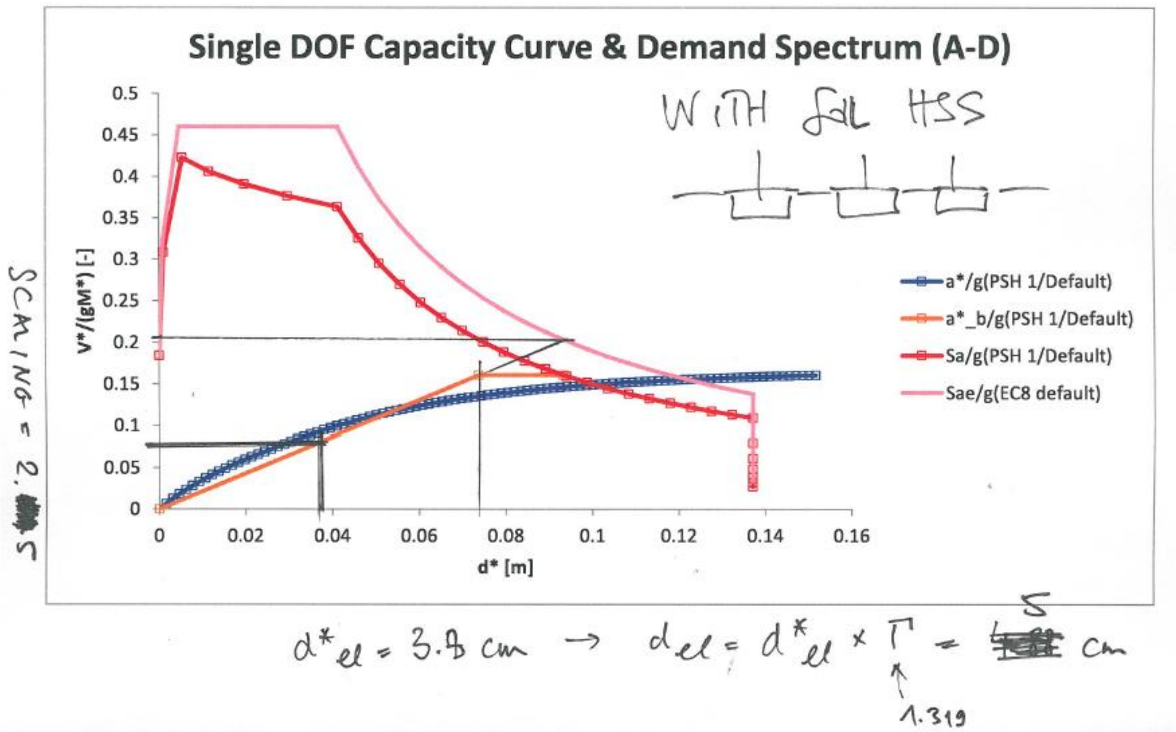


Figure 4.6. Definition of factor f and corresponding global displacement (structure+soil)

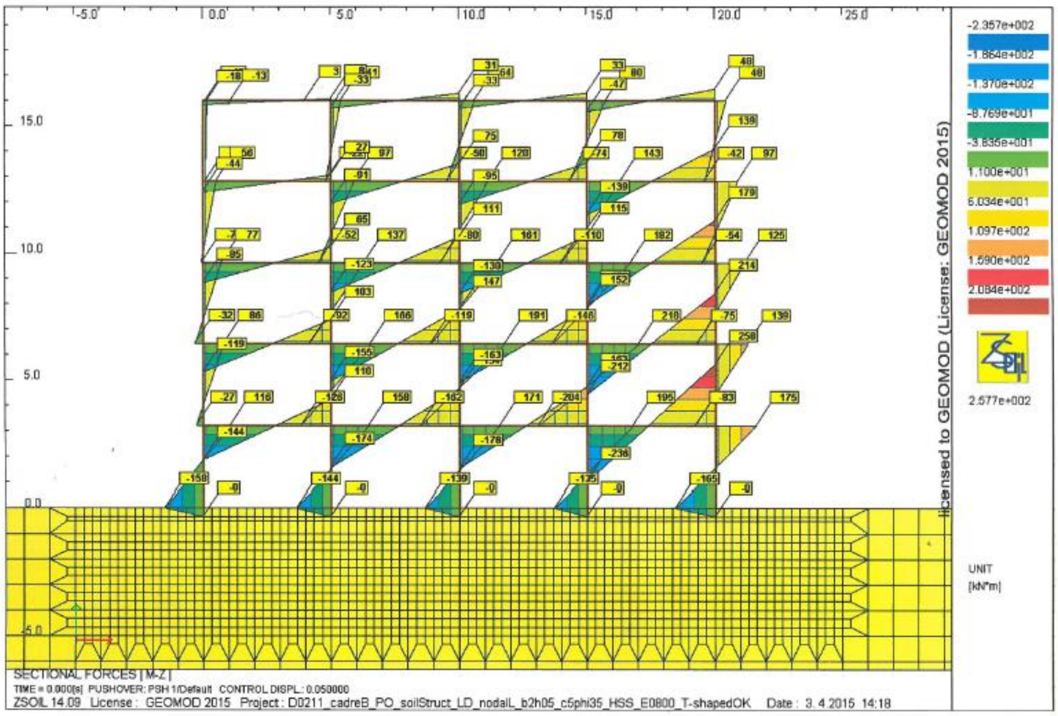


Figure 4.7a. Corresponding reference bending moments (structure+soil)

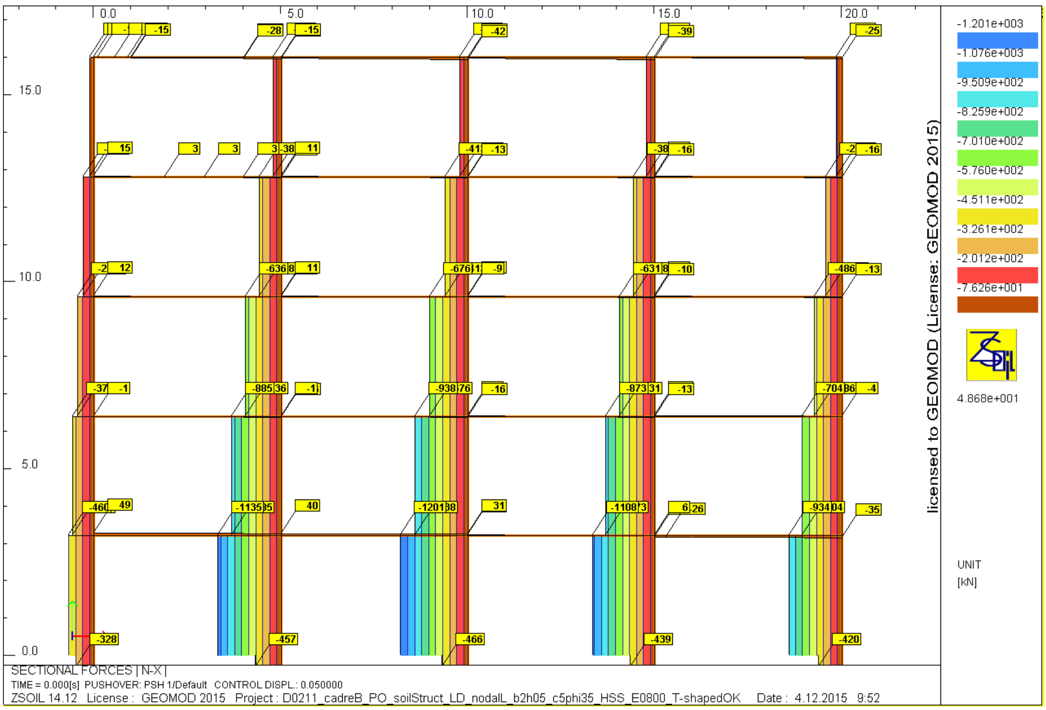


Figure 4.7b. Corresponding reference normal forces (structure+soil)

		structure only		structure+soil	
		d_el	3.6 cm	d_el	5 cm
		scaling	2.5	scaling	2.5
		M(d_el)	**M(scaled)**	M(d_el)	**M(scaled)**
Column		[kNm]	**[kNm]**	[kNm]	**[kNm]**
1	bottom	-122	**-305**	-158	**-395**
1	top	-3	**-8**	116	**290**
2	bottom	-257	**-643**	-144	**-360**
2	top	58	**145**	158	**395**
3	bottom	-294	**-735**	-139	**-348**
3	top	95	**238**	171	**428**
4	bottom	-327	**-818**	-135	**-338**
4	top	134	**335**	195	**488**
5	bottom	-269	**-673**	-165	**-413**
5	top	92	**230**	175	**438**
Beam					
A	left	-123	**-308**	-144	**-360**
A	right	126	**315**	110	**275**
B	left	-139	**-348**	-174	**-435**
B	right	150	**375**	134	**335**
C	left	-150	**-375**	-178	**-445**
C	right	164	**410**	163	**408**
D	left	-165	**-413**	-236	**-590**
D	right	185	**463**	258	**645**

Table 4.1. Comparison of bending moments for both "structure only" and "structure+soil" cases (E_{ref} and E_{ref} * f), without considering behavior factor q

Remark:

- Foundation flexibility is shown to drastically reduce the verification values of the bending moments at the columns bottom. However, in the structure+soil case, values at the top of the columns increase as soil, foundation and floor stiffness all play a role in the column bending moment distribution. In the structure + soil case, the verification values of the bending moments for the beam elements and the top part of the columns are systematically higher than for the structure only case.

4.2.1 Comparison of force-based compliance factors for columns and isolated footings

For the example building on isolated footings, a **behavior factor of q = 1.5** has been selected as the resistance of the soil is lower than the resistance of the structure (soil reaches design bearing capacity before columns reach their design structural resistance).

- In the structure only case: the lowest compliance factor for the columns is α_{eff} **= 0.80** (column 4; M_d = 545 kNm; N_d = 1’200 kN; M_{Rd} = 440 kNm)
- In the structure+soil case: the lowest compliance factor for the columns is α_{eff} **= 1.32** (column 5; M_d = 292 kNm; N_d = 1’200 kN; M_{Rd} = 387 kNm)

- In the structure only case: the lowest compliance factor for the isolated footings is α_{eff} **= 0.51** (footing 4; bearing capacity; N_d= 1’200 kN, M_d = 545 kNm, V_d = 240kN)
- In the structure+soil case: the lowest compliance factor for the isolated footings is α_{eff} **= 0.80** (footing 5; bearing capacity; N_d= 1’055 kN, M_d = 275kNm, V_d = 177kN)

Remarks:

- For the soil + structure case, only one set of characteristic values for the soil has been used. As explained in 3.2.3 a sensitivity analysis should be performed by varying the soil parameters between their upper and lower bounds.
- For the determination of the compliance factor of the columns the ratio M_{Rd} / M_d has been used as the moment for gravity loads only is assumed 0 in the structural elements (simplification for the sake of illustration). It is also assumed that shear failure is not determinant for the columns.
- For the determination of the compliance factor for the isolated footings, the more general definition of the compliance factor has been used.

4.3 Chord rotation of columns, with/without soil

Figure 4.8, taken from [11], illustrates graphically the computation of the chord rotation for reinforced concrete members.

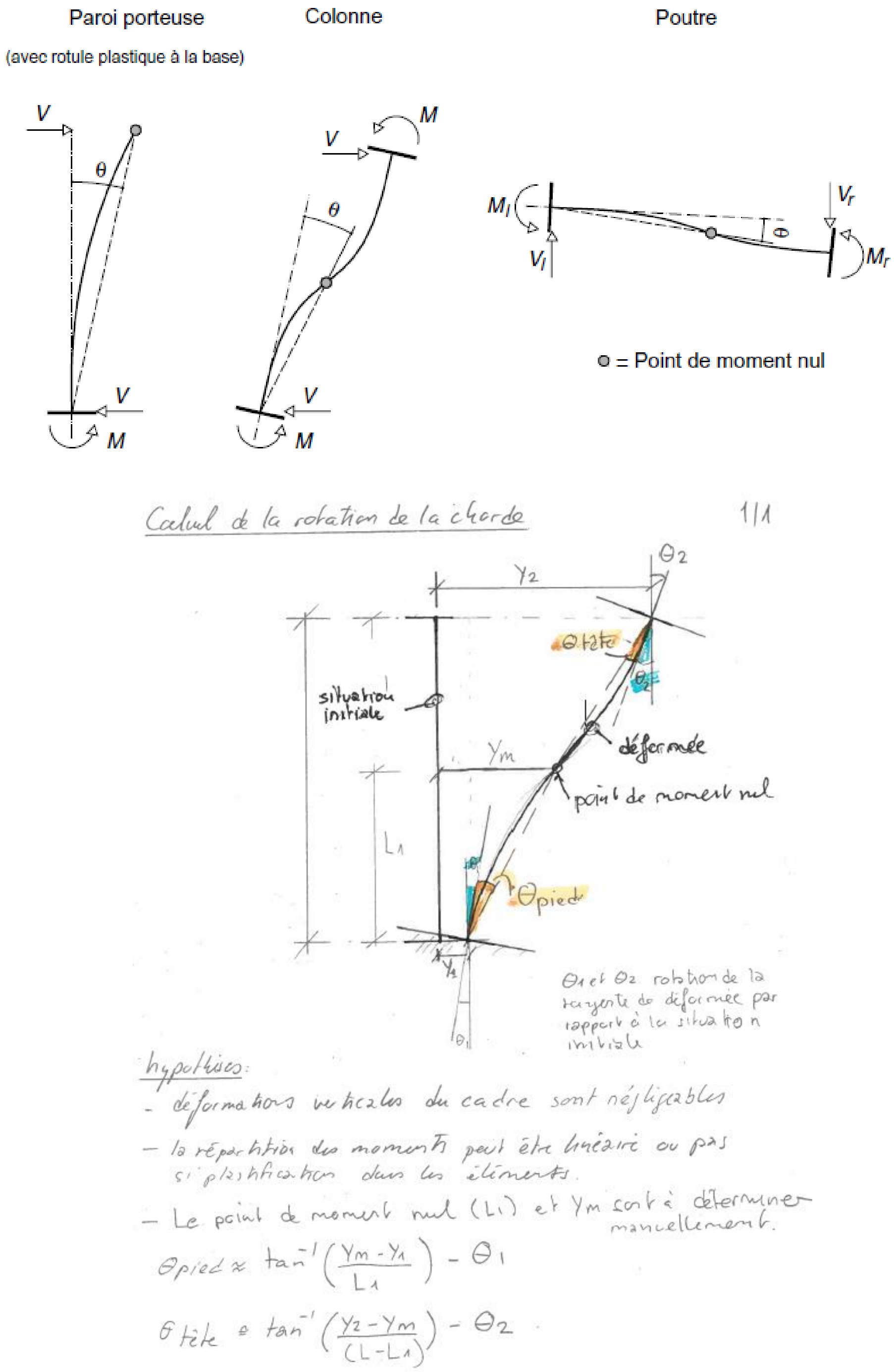


Figure 4.8. Chord rotation computation (taken from [11], top and [13], bottom)

According to Figure 3.5, the target displacement of the structure only case reaches 8.8 cm. The maximal chord rotation for the columns on the bottom floor is θ_d = 0.0073 rad (column 5, bottom). In the structure+soil case (shallow isolated foundations), Figure 3.11 indicates a target displacement of 12.3 cm. The maximal chord rotation for the columns on the bottom floor is θ_d = 0.0025 rad (column 5, bottom). The ultimate chord rotation for column 5 has been computed as θ_u 0.007 rad. Thus, the displacement based compliance factor for column 5 can be computed as:

- Structure only: $\boldsymbol{\alpha_{eff} = (0.007/1.3)/0.0073 = 0.74}$
- Structure+soil: $\boldsymbol{\alpha_{eff} = (0.007/1.3)/0.0025 = 2.15}$

In the structure+soil system, foundations are weaker than the structural elements (they reach their nominal resistance first). In this situation, it is not clear what criteria should be used in order to declare that the structure+soil system has reached its ultimate displacement capacity in a push-over analysis. This aspect has not been further investigated in this study.

Remark:

For the structure+soil case, only one set of characteristic values for the soil has been used. As explained in 3.2.3 a sensitivity analysis should be performed by varying the soil parameters between their upper and lower bounds.

5. Conclusions

This study shows that for the example building built on isolated shallow foundations, pushover analysis taking soil into account leads to maximum internal forces (or member curvatures) smaller than the ones obtained when neglecting soil (structural only pushover analysis with fixed boundary conditions).

The distribution of the bending moments in the structure is also substantially different, which shows that neglecting soil structure interaction for a frame structure on isolated footings can be potentially on the unsafe side for some structural elements.

In the example treated in this report, a reduction of 20% to 30% of the maximal bending moments in the critical columns on the ground floor is obtained (see Figures 3.21 and 3.22 and chapter 4.1), which leads to higher compliance factor for the structural members. On the other hand, if the building is built on a rigid foundation (for instance: on a thick continuous slab or on a pile raft with a stiff basement floor, see Figure 5.1), no significant reduction is observed.

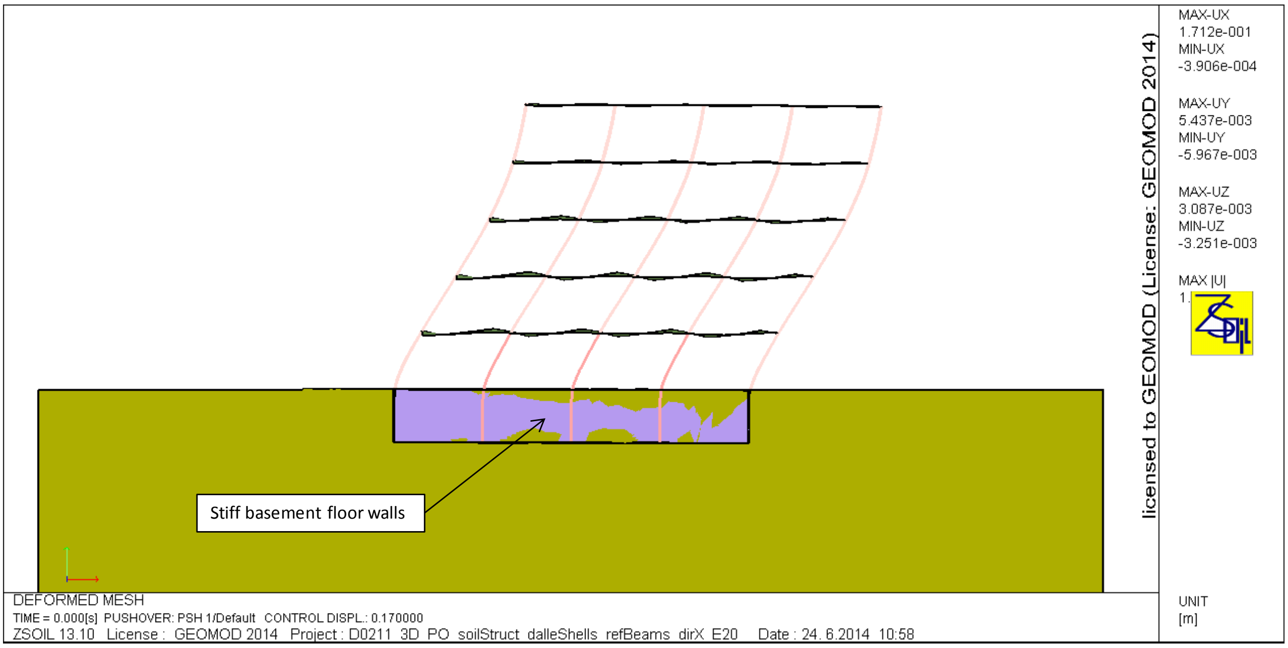


Figure 5.1 3D Pushover analysis in X direction, with stiff walls in basement
Deformed mesh corresponding approximately to target displacement dt = 17 cm

In order to trigger the foundation's rocking phenomenon, the soil must be modeled with an adequate elasto-plastic constitutive law. Simple (Mohr-Coulomb) or advanced (Hardening soil with Small strain extension) constitutive laws can be used. Characteristic (and not design) parameters (E, c, ϕ) given by the geotechnician should be introduced carefully in the soil's model, as well as interface elements. Sensitivity analyses should be performed on these characteristic values, as variations in the values of E and ϕ will decrease the internal loads or deformations for some structural members and increase them for others. In most cases it is not possible to know beforehand which combination is the determinant one. The N2 pushover method [10] used in this study is one of many nonlinear static procedures which have emerged in the last two decades in order to assess seismic safety of structures. Open topics that would be worth future research include:

- Testing the influence of soil structure interaction in the framework of other nonlinear static procedures (capacity spectrum method (or CSM) for instance)
- Comparing the results of nonlinear pushover static analyses to the ones of nonlinear time history analyses
- Analyze the influence of taking soil structure interaction for other relevant structures in Switzerland, such as bridge foundations.

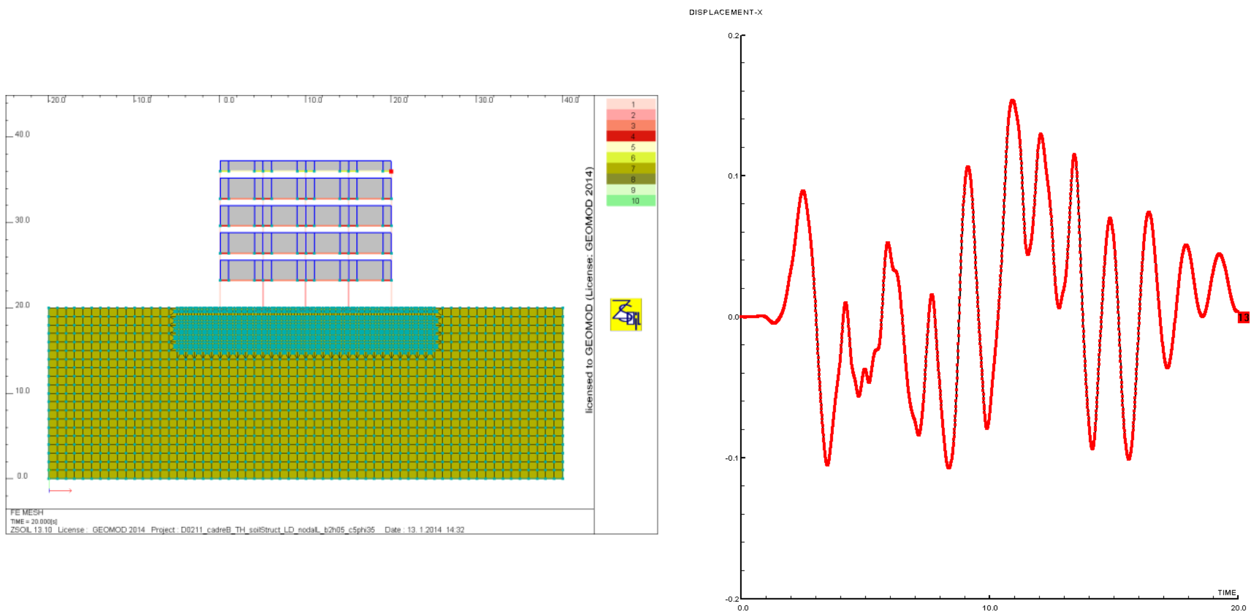


Figure 5.2 Structure + soil time history with DRM analysis, frame B (shallow foundations)
Reduced model top floor horizontal displacement
Hypothesis: seismic input (accelerogram) is coherent with demand spectrum

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