ZSoil

for geotechnics & structures

STATIC PUSHOVER ANALYSIS

Report 070202

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Contents

Chapter 1

INTRODUCTION

Modern seismic design codes allow engineers to use either linear or nonlinear analyses to compute design forces and design displacements. In particular, Eurocode 8 contains four methods of analysis: linear simplified static analysis, linear modal analysis, nonlinear pushover analysis and nonlinear time-history analysis. These methods refer to the design and analysis of framed structures, mainly buildings and bridges. The two nonlinear methods require advanced models and advanced nonlinear procedures in order to be fully applicable by design engineers. This report gives an introduction to the use of PUSHOVER analysis with ZSOIL.

Displacement-based and force-base elements are used in this study. The first is a classical two-node, displacement-based, Euler-Bernoulli frame element. The second is a two-node, force-based, Euler Bernoulli frame element. The main advantage of the second element is that it is "exact" within the relevant frame element theory. This implies that one element per frame member (beam or column) is used in preparing the frame mesh, thus leading to a reduction of the global number of degrees of freedom. The complete theory for the force-based element can be found in (Spacone, 1996).

The nonlinear response of a 2D model of an existing building is presented as an illustration. The building is a residential two-storey reinforced concrete building in Bonefro, Italy. It is representative of typical residential building construction in Italy in the 1970's and 1980's. The design spectrum for the building was obtained from EC8 using the local soil properties and the peak ground acceleration given by the new Italian seismic map.

This work presents the nonlinear pushover procedure, which is based on the N2 method developed by Fajfar (Fajfar, 1999). The procedure is illustrated on a 2D model of an existing building and the data structure to perform such analyses in ZSOIL is presented. More details and comparisons with dynamic analysis can be found in (Belgasmia & al. 2006)

Chapter 2

NONLINEAR FRAME ANALYSIS METHODS IN EUROCODE 8

In this section the nonlinear pushover procedures given by Eurocode 8 are presented. Pushover analysis is described in §4.3.3.4.2. of EC8 Part 1. According to EC8, pushover analysis may be used to verify the structural performance of newly designed buildings and of existing buildings. In particular, pushover analysis may be used for the following purposes:

- to verify or revise the overstrength ratio values α_u/α_1. The definition of the overstrength ratio is recalled with the aid of Window 2-1. If the structure is pushed with a lateral load distribution of constant shape and increasing intensity, α_u/α_1 is the ratio between the base shear $\alpha_u V_b$ corresponding to the formation of a mechanism and the base shear $\alpha_1 V_b$corresponding to the formation of the first plastic hinge. The overstrength ratio is used in linear analysis to compute the behaviour factor q (see EC8 Part 1 §3.2.2.5, 5.2.2.2, 6.3.2, 7.3.2) which allows to obtain nonlinear design spectrum for inelastic analysis starting from the linear design spectrum;

Window 2-1: Overstrength ratio

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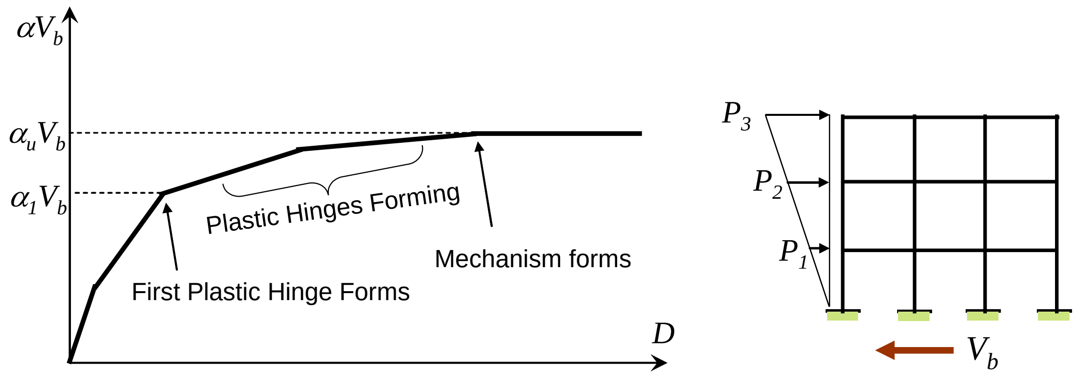

Overstrength ratio

Window 2-1

- to estimate the expected plastic mechanisms and the damage distribution;
- to assess the structural performance of existing or retrofitted buildings for the purposes of EN 1998-3 (EC8 Part 3);
- as an alternative to the design based on linear-elastic analysis which uses the behaviour factor q. In this case, the target displacement found from the pushover analysis should be used as the basis for the design;
- EC8 Part 1 §4.3.3.4.2.1 adds that buildings not conforming to the regularity criteria of EC8 shall be analyzed using a spatial (3D) structural model. Two independent analyses with lateral loads applied in one direction only may be performed. No indications are given in EC8 or in the published literature on how to perform a pushover analysis with two loads distributions applied simultaneously in two orthogonal directions, therefore it is assumed here that in a pushover analysis the structure is pushed with loads applied in one horizontal direction at a time (the vertical seismic is typically neglected in buildings). Indications are given on how to combine the effects of the actions applied separately in two horizontal directions (EC8 Part 1 §4.3.3.5). For buildings conforming to the regularity criteria of EC8 the analysis may be performed using two planar models, one for each principal direction horizontal direction.

The structural element models and the resulting structural model of the overall building are very similar for pushover and nonlinear time-history analysis. The only difference lies in the need to have cyclic models for the time-history analysis.

The initial steps of both nonlinear procedures are identical: construction of the nonlinear frame model and application of the gravity loads. The gravity loads remain constant during the nonlinear analysis (both static and dynamic). The application of the gravity loads is schematically shown in Window 3-1. The value of the constant gravity loads is given by EC8. This initial step is quite important because it may change the initial state of the structure. In a reinforced concrete building, for example, the gravity loads typically induce cracking in beams and apply high axial forces on columns.

Window 2-2: Gravity loads

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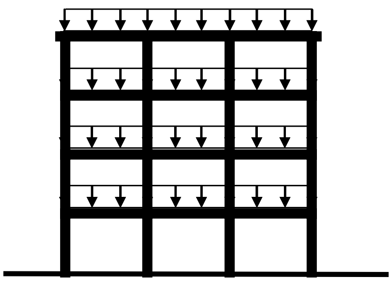
$$G_k + P_k + \sum_i \left(\psi_{2i} Q_{ki}\right)$$



Application of constant gravity loads

Window 2-2

Finally, EC8 Part 1 §4.3.3.4.1 states that the seismic action in nonlinear methods shall be applied in both positive and negative directions (depending on the symmetry of the structure).

Chapter 3

NONLINEAR STATIC PUSHOVER ANALYSIS ACCORDING TO EUROCODE 8

The Nonlinear Static Pushover Procedure in EC8 follows the N2 method developed by Fajfar (1999). The method consists of applying constant load shapes to the building model. The load shapes represent the lateral loads applied by the ground motion. The load intensity is increased in a pseudo-static manner. The structure model can be planar (2D) or spatial (3D), depending on the regularity characteristics of the building. The load pattern, on the other hand, is always applied in one direction only. For analyses with input ground motion in more than one direction, for example input ground motion in the x and y directions, combination rules are given by EC8.

The nonlinear pushover analysis consists of applying monotonically increasing constant shape lateral load distributions to the structure under consideration. The structure model can be either 2D or 3D. In particular, EC8 states that for buildings with plan regularity, 2D analysis of single plane frames can be performed, while for buildings with plan irregularity a complete 3D model is necessary. Given that the nonlinear methods are particularly interesting for existing buildings, which are rarely regular, a 3D model is required in most cases.

The N2 method was developed using a shear building model, i.e. a frame model with floors rigid in their planes. Furthermore, vertical displacement are typically neglected in the method and only the two horizontal ground motion components, x and y, are considered. Extension to the general case of a fully deformable frame is straightforward. The N2 method consists of applying two load distributions to the frame:

- a "modal" pattern, that is a load shape proportional to the mass matrix multiplied by the first elastic mode shape,
$$\mathbf{P}^1 = \mathbf{M}\boldsymbol{\varphi}_1$$
- a "uniform" pattern, that is a mass proportional load shape,
$$\mathbf{P}^2 = \mathbf{MR}$$

where $\mathbf{M}$ is the mass matrix, $\boldsymbol{\varphi}_1$ is the first mode shape and $\mathbf{R}$ a vector of 1s corresponding to the degrees of freedom parallel to the application of the ground motion and 0s for all other dofs. In the N2 method φ_1 is normalized so that the top floor displacement is 1, i.e. $\varphi_{1,n} = 1$. The two load distributions are schematically shown in Window 3-1. The applied lateral load distributions are increased and the response is plotted in terms of base shear V_b vs. top floor displacement D (for example center of mass of the top floor)[1]. This is the so-called **pushover curve** or **capacity curve** (also shown schematically in Window 3-1).

Window 3-1: Nonlinear static pushover analysis according to Eurocode 8

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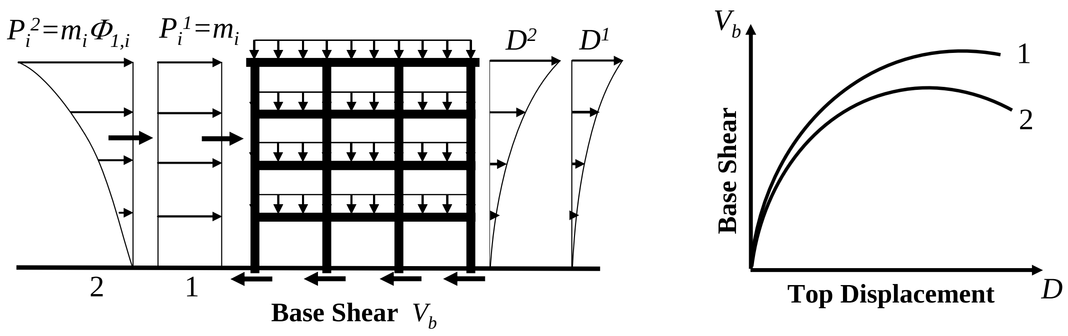

Load distribution for pushover analysis according to EC8 and pushover response curve.

Window 3-1

The N2 procedure transforms the response of the MDOF system into the response of an equivalent SDOF system. This is necessary in order to compare the building capacity curve of Window 3-1 with the demand, expressed in the design codes by the design spectra, which refer to SDOF systems.

3.1 Equivalent SDOF model and capacity diagram in Eurocode 8

As previously stated, Fajfar (1999) assumes that the building is a shear frame, i.e. the floors are rigid in their own plane. If the vertical displacements of the building are neglected, the floor displacements are given by the three degrees of freedom shown in Window 3-2. The degrees of freedom are typically taken at the center of mass. Note that the beams can deform outside the floor plane, that is the nodes have out-of-floor plane rotational degrees of freedom.

[1]Pushover forces are defined as follows: (masses*acceleration multiplier)*(loadpattern), load pattern is constant, linear or modal, multiplier is defined iteratively to achieve prescribed control node displacement (see APPENDIX C for the details of the nonlinear static procedure)

Window 3-2: Nonlinear static pushover analysis according to Eurocode 8

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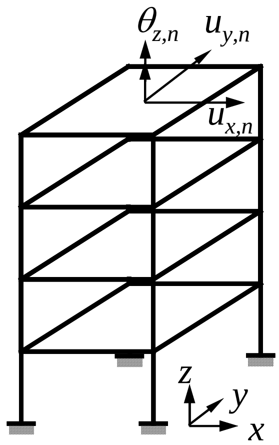

Rigid slab degrees of freedom in 3D shear frame (for simplicity, column axial deformability is neglected)

Window 3-2

The theoretical derivation of the transformation procedure is as follows. The equations of motion of a MDOF building subjected to base ground motion model is:

$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{F}(\mathbf{U}) = -\mathbf{M}\mathbf{R}a$$

where damping is neglected, $\mathbf{M}$ is the mass matrix (assumed diagonal in the original derivation of the N2 method), $\mathbf{U}$ and $\mathbf{F}$ are vectors representing relative displacements and internal forces, respectively, $\mathbf{R}$ is the influence vector and a is the ground acceleration as function of time, i.e. $a = a(t)$. a is given in one direction only. In the linear elastic case $\mathbf{F} = \mathbf{KU}$(where $\mathbf{K}$ is the structure stiffness matrix), in the nonlinear case $\mathbf{F}$ depends on the displacement history. For uni-directional ground motion, for example in the direction x, the influence vector $\mathbf{R}$ consists of 1s in correspondence to the dofs in the x direction, and 0s for all other dofs. For example, for the frame of Window 3-2, considering only the 12 dofs of the 4 stories, $\mathbf{R}^T = \{\mathbf{1}, \mathbf{1}, \mathbf{1}, \mathbf{1}\}$, with $\mathbf{1} = \{1\ 0\ 0\}$.

The first assumption (and approximation) of the N2 method is that the displacement $\mathbf{U}$ has a constant shape that does not change during the response to the ground motion:

$\mathbf{U} = \mathbf{\Phi} D_t$ or $\mathbf{U}(\mathbf{x},t) = \mathbf{\Phi}(\mathbf{x}) D_t(t)$, where $D_t(t)$ is the intensity at the pseudo-time t of the displacement shape $\mathbf{\Phi}$, where $\mathbf{x}$ indicates that the displacement shape depends on the degree of freedom location. For convenience, $\mathbf{\Phi}$ is normalized in such a way that the top-storey displacement is equal to 1, i.e.$\mathbf{\Phi}_n = 1$. This way $D(t)$ gives the top floor displacement at time t.

In the pushover analysis of frames with rigid floors, lateral loads are applied at the centre of mass of each storey. The vector of the lateral loads $\mathbf{P}$ is $\mathbf{P} = p\mathbf{\Psi} = p\mathbf{M}\mathbf{\Phi}$.

The magnitude of the lateral load is p, i.e. $p = p(t)$. The distribution of lateral loads is related to the assumed displacement shape $\mathbf{\Phi}$. This is the second assumption of the procedure. In more complex models with deformable slabs and with distributed masses at each node the load, the load vector $\mathbf{P}$ is applied to all degrees of freedoms with mass in the direction of the applied ground motion. Note that the displacement shape $\mathbf{\Phi}$ is needed only for the transformation from the MDOF system to the equivalent SDOF system of the nonlinear pushover procedure. In the general case of a 3D building, $\mathbf{\Phi}$ has nonzero components in the six dofs of each node.

From above equations it follows that in a shear frame the lateral force in the i-th storey is proportional to the component $\mathbf{\Phi}_i$ of the assumed displacement, weighted by the storey mass m_i. If the ground motion is applied in the x direction $P_i = pm_i\Phi_{xi}$

From statics it follows that $\mathbf{P} = \mathbf{F}$, that is the internal forces $\mathbf{F}$ are equal to the pseudo-static external forces $\mathbf{P}$.

By combining the above equations and by pre-multiplying by $\mathbf{\Phi}^T$, we obtain

$$\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}\ddot{D}_t + \mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}p = \mathbf{\Phi}^T\mathbf{M}\mathbf{R}a$$

The left term of Equation is then divided and multiplied by $\mathbf{\Phi}^T\mathbf{M}\mathbf{R}$ to obtain

$$\underbrace{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}_{m^*}\underbrace{\frac{\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}}{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}}_{\frac{1}{\Gamma}}\ddot{D}_t + \underbrace{\frac{\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}}{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}}_{\frac{1}{\Gamma}}\underbrace{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}p}_{V_b} = \underbrace{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}_{m^*}a$$

where m^* is the mass of the SDOF equivalent to the MDOF building

$$m^* = \mathbf{\Phi}^T\mathbf{M}\mathbf{R}$$

For a shear building and ground motion applied in the x direction:

$$m^* = \sum m_i\Phi_{x,i}$$

where $\Phi_{x,i}$ is x component of the modal shape vector for node i. The constant Γ controls the transformation from MDOF to SDOF and back:

$$\Gamma = \frac{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}{\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}}$$

For a shear building and ground motion in the x direction:

$$\Gamma = \frac{\sum m_i\Phi_{x,i}}{\sum m_i\Phi_{x,i}^2}$$

Γ is a factor that, for $\mathbf{\Phi}$ equal to the mode shape of one of the building's modes, corresponds to the mode participation factor. In the development of the N2 method, can be any reasonable deformed shape.

V_b is the base shear of the MDOF building in the direction of the ground motion, equal to:

$$V_b = \mathbf{\Phi}^T\mathbf{M}\mathbf{R}p$$

For a shear building and ground motion applied in the x direction:

$$V_x = p\sum m_i\Phi_{x,i} = \sum P_{x,i}$$

Window 3-3: Transformation from response of MDOF to equivalent SDOF

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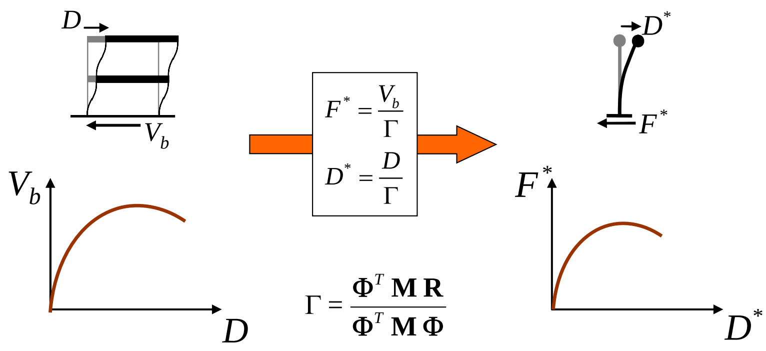

Capacity curve: transformation from response of MDOF to equivalent SDOF

Window 3-3

The equation the SDOF equivalent to the MODF building is thus obtained:

$$D^* = \frac{D_t}{\Gamma}, \qquad F^* = \frac{V_b}{\Gamma}$$

The above derivation allows the transformation of the MDOF pushover capacity curves of Window 3-1 into pushover curves for the equivalent SDOF system, as shown in Window 3-3

Both force and displacement axes are scaled by the same factor Γ. The stiffness of the system remains the same. Note that the transformation factor Γ depends on the shape $\mathbf{\Phi}$ of the assumed displacement shape, and is thus different for different choices of $\mathbf{\Phi}$. As shown in Window 3-1, two forms of loadings are suggested in EC8:

a) $\mathbf{\Phi} = \mathbf{\Phi}_1$, thus $\Gamma = \dfrac{\mathbf{\Phi}_1^T \mathbf{M}\mathbf{R}}{\mathbf{\Phi}_1^T \mathbf{M}\mathbf{\Phi}_1}$,

b) $\mathbf{\Phi} = \mathbf{R}$ thus $\Gamma = 1$.

3.2 Linearization of the capacity curve and comparison to demand spectrum

3.2.1 Linearization of the capacity curve

In order to compare the capacity curve to the demand curve given by the design spectrum, the nonlinear pushover curves of the SDOF are approximated by elastic-perfectly plastic (or bilinear) curves. According to Annex B if the draft EC8 0 this transformation can be based on the equal energy principle. A target displacement is assumed, and equal energy is assumed between bilinear and nonlinear pushover curves. This simple procedure is illustrated in Window 3-4.

Window 3-4: Linearization of the capacity curve

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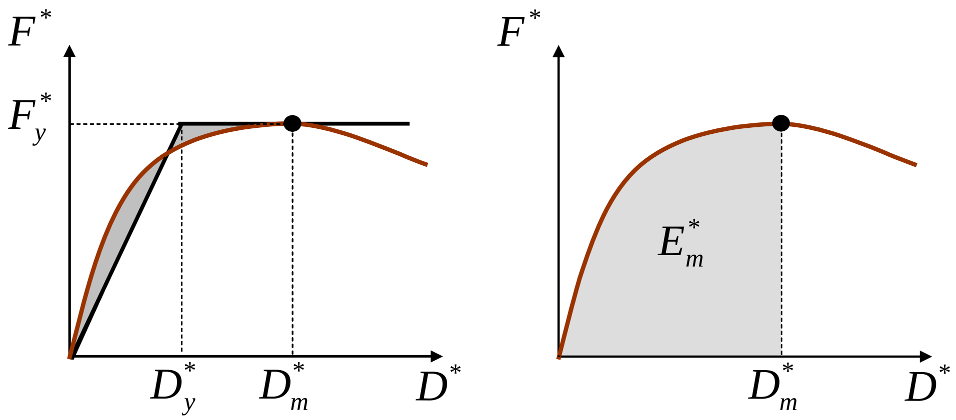

Bilinearization of the capacity curve of SDOF

Window 3-4

The bilinearization of Window 3-4 gives the yield force and the yield displacement

$$D_y^* = 2\left(D_m^* - \frac{E_m^*}{F_y^*}\right),$$

which allow the initial elastic period to be computed as:

$$T^* = 2\pi\sqrt{\frac{m^* D_y^*}{F_y^*}}.$$

Secondly, the capacity curve is transformed into capacity spectrum by normalizing the force with respect to the SDOF weight. The resulting capacity spectrum is shown in Window 3-5.

Window 3-5: Capacity spectrum

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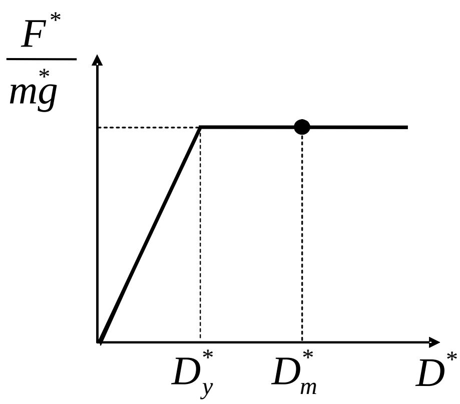

SDOF capacity spectrum

Window 3-5

3.2.2 Seismic demand

The demand on the building is given by the design spectrum provided by the design codes. In order to compare capacity and demand, the first step is to transform the format of the design spectrum from the classical Acceleration A vs Period T format to the ADRS format, i.e. Acceleration A vs. Displacement D. The procedure is rather simple, as Acceleration and Displacement are related by[2]

$$S_D = \left(\frac{T}{2\pi}\right)^2 S_A$$

The transformation to the ADRS spectrum is shown in Window 3-6. Lines from the origin represent constant periods.

Window 3-6: ADRS linear spectrum

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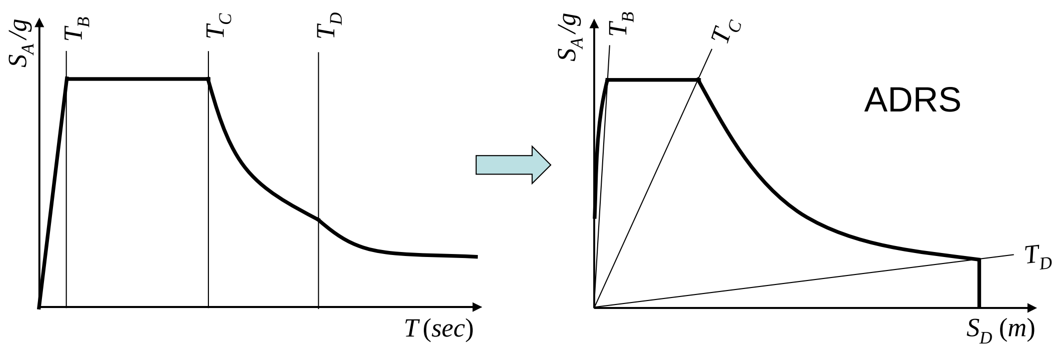

Transformation to ADRS linear spectrum

Window 3-6

The capacity spectrum of Window 3-5 is now compared to the ADRS demand spectrum of Window 3-6. The comparison is not immediate, because the capacity spectrum is nonlinear, while the ADRS spectrum given by the design codes is linear.

For a SDOF system with an bilinear plastic behavior, the acceleration spectrum S_A and the displacement spectrum S_D can be determined as:

$$S_A = \frac{S_{Ae}}{R_\mu}$$
$$S_D = \frac{\mu}{R_\mu} S_{De} = \frac{\mu}{R_\mu}\frac{T^2}{4\pi^2} S_{Ae} = \mu \frac{T^2}{4\pi^2} S_A$$

where subscript e indicates elastic, μ is the ductility factor $=$ maximum inelastic displacement/yield displacement, and R_μ is the reduction factor due to ductility. The reduction

[2]The design spectra are actually given in terms of pseudo-acceleration rather than acceleration, but for low damping the two are basically identical, thus the "pseudo-" is typically dropped. It is worth recalling that the pseudo-acceleration is defined as $A = \omega^2 D$

factor R_μ can be found in different ways, some analytical, other approximated. In the simple version of the N2 method, the following approximated expressions are given:

$$R_\mu = (\mu - 1)\frac{T}{T_C} + 1, \qquad T < T_C$$
$$R_\mu = \mu, \qquad T \geq T_C$$

where T_C is a characteristic period of the ground motion that depends on the soil type and is given by EC8. It typically corresponds to the transition from the constant acceleration range (short-period range) to the constant velocity range (medium-period range) in the response spectrum. The above equations suggest that in the short-period range the equal displacement principle is applied (elastic and inelastic SDOFs have the same maximum displacement), while in the medium- and long-period range the equal energy principle is applied. These principles are shown in Window 3-7.

Window 3-7: Transformation of elastic response into bilinear response

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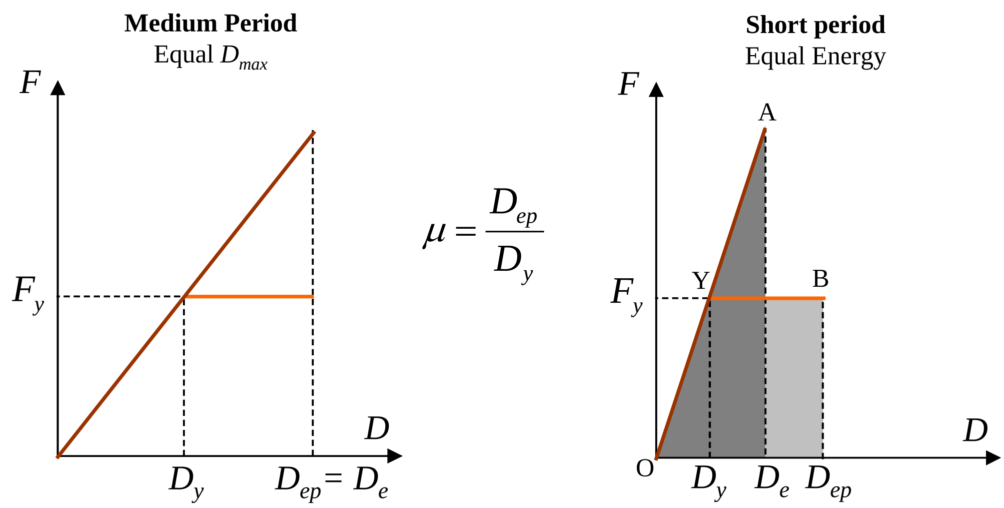

Transformation elastic response – bilinear response:

equal maximum displacement and equal energy assumptions

Window 3-7

Above equations are used to obtain inelastic demand spectra of constant ductility, as shown in Window 3-6. Note that this procedure is approximate, and that inelastic demand spectra can be determined by a rigorous (but more complex) procedure by using nonlinear dynamic analysis.

Using the procedures illustrated in the section, the seismic demand on the equivalent SDOF can be determined. The steps are schematically illustrated in Window3-10 for a bilinear oscillator with medium or long elastic period T^*. **Given the elastic demand spectrum and the bilinear capacity spectrum, from a theoretical point of view the target displacement D_t^* is determined by finding the inelastic demand spectrum of ductility $\bar{\mu}$ that intersects the capacity spectrum in a point corresponding to a capacity ductility $\bar{\mu}$. In other words, the design point is given by the point with equal**

demand and capacity ductility. In practice, this is done very easily using the approximate procedure described in Window 3-11.

Window 3-8: Capacity and Demand spectra for short period T^*

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(a). $T^* < T_C$ (Short periods)

(a1): $\dfrac{F_y^*}{m^*} \geq S_A(T^*) \Rightarrow$ the response remains linear elastic (case a) $\;D_t^* = D_{et}^*$

(a2): $\dfrac{F_y^*}{m^*} < S_A(T^*) \Rightarrow$ the response enters the nonlinear plateau (case b)

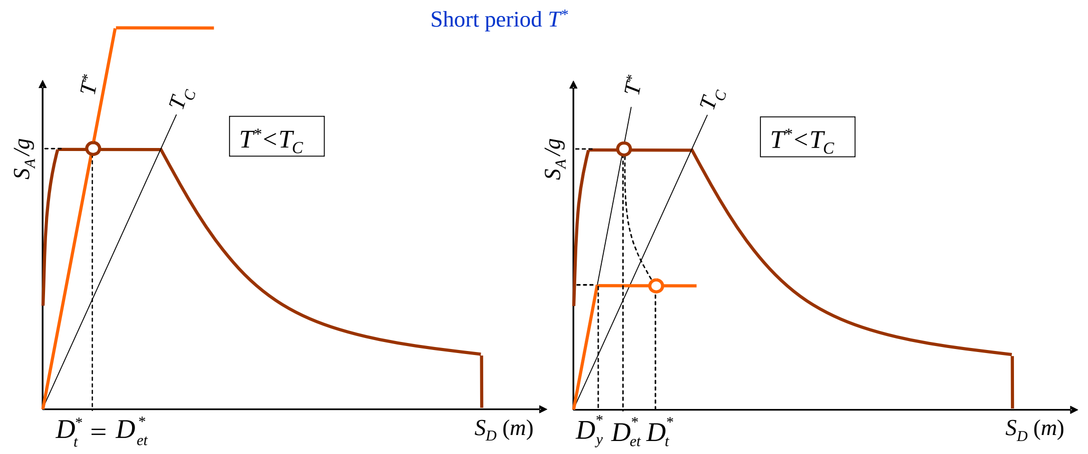
$$D_t^* = \frac{D_{et}^*}{q_\mu}\left(1 + (q_\mu - 1)\frac{T_C}{T^*}\right)$$

where $q_\mu = \dfrac{S_{Ae}(T^*)}{F_y^*/m^*}$ is the reduction factor

(b). $T^* \geq T_C$ (Medium and long periods) – Window 3-10

$$D_t^* = D_{et}^*$$

case a) case b)



Capacity and Demand spectra for short period T*:

NB: The above procedure is based on the assumption of a tentative target displacement D_m^* (from which the equal energy principle is used to obtain the bilinear capacity curve in Window 3-4). If the target displacement D_t^* is very different from the assumed value D_m^*, then the procedure must be repeated, setting for instance $D_m^* = D_t^*$. This is a simple iterative procedure that converges very rapidly.

Window 3-8

Window 3-9: Demand spectra for constant ductilities in AD format

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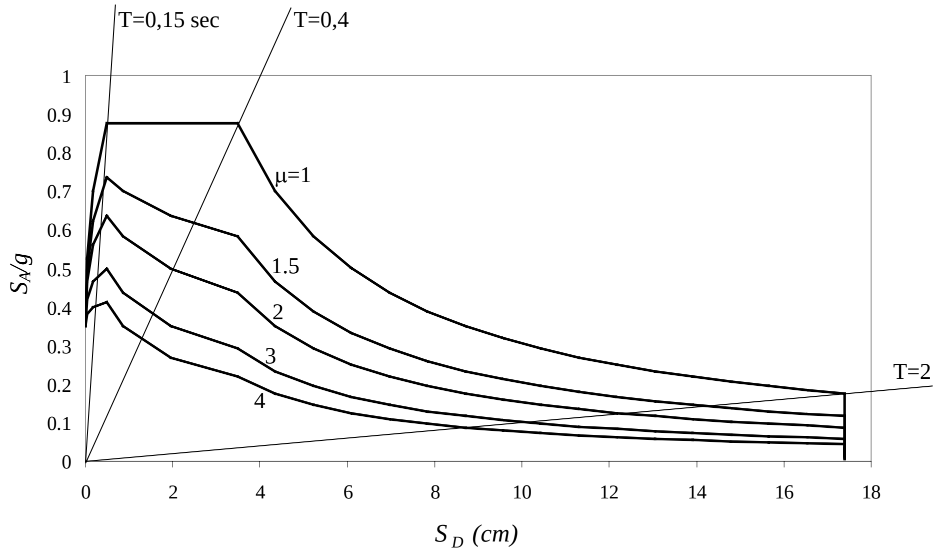

Demand spectra for constant ductilities in AD format (based on EC8 spectrum for Zone 1, Soil type A)

Window 3-9

Window 3-10: Capacity and Demand spectra for long and medium period T^*

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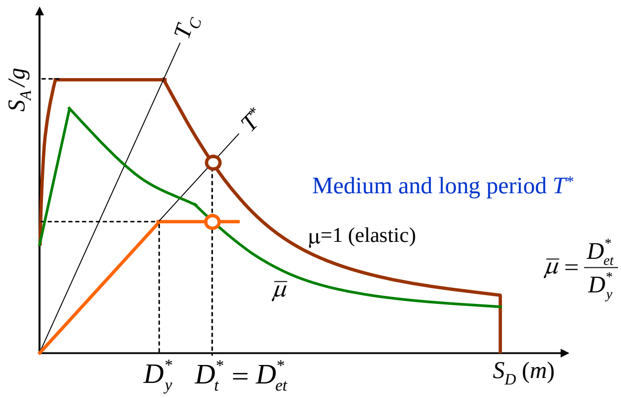

Capacity and demand spectra for long and medium period T*: determination of the target displacement.

Window 3-10

The target displacement at the top of the building is obtained by inverting the transformation in Window 3-7, i.e. $D_t = \Gamma D_t^*$.

One minor issue concerns the point at which the pushover curve is stopped. While it is not necessary to continue the pushover analysis to unreasonable values of the top floor displacement (this would mean long computational times and convergence problems at large displacement values), there is no general rule on when to stop the pushover curve. This means that it may happen that the pushover curve is stopped at a displacement level smaller that the computed target displacement. In this case, the pushover analysis must be repeated and it must be stopped at higher top displacement values. It is suggested to push the structure to top-displacements of the order of 2%-3% h, where h is the entire height of the building, for pushover analysis at the Ultimate and Collapse Limit States.

Finally, note that in general for each seismic input direction (x and y), four different pushover analysis should be performed. Besides considering two different load shapes (Window 3-1) it is in general necessary to consider the forces applied both with the positive and the negative sign, as the irregularity of the building may lead to different responses in the positive and negative directions.

3.3 Summary of nonlinear static pushover analysis in Eurocode 8

The steps necessary for nonlinear static pushover of a structure subject to the design ground motion in a single direction are as follows:

Window 3-11: Summary of nonlinear static pushover

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1. Create the nonlinear structural model
2. Apply constant gravity loads
3. For each load shape ($\mathbf{\Phi} = \mathbf{\Phi}_1$ and $\mathbf{\Phi} = \mathbf{R}$) eventually taken with the positive and negative signs:

A: Compute the capacity curve for MDOF

B: Compute the capacity curve for SDOF.

C: Get the bilinear capacity spectrum for SDOF based on assumed D^*_m (assumed target displacement)

D: Compute the target displacement D^*_t for SDOF. If D^*_t is very different from $D^*_m = D^*_t$, go back to (c) with $D^*_m = D^*_t$

E: Convert D^*_t to the target displacement D_t for MDOF building

F: Check structural performance corresponding to limit state under consideration

Window 3-11

3.4 Accidental torsional effects in Eurocode 8

According to EC8 (Part 1 §4.3.2), "...in order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated center of mass at each floor i shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:

$$e_{ai} = \pm 0.05 L_i$$

where e_{ai} is the accidental eccentricity of storey mass i from its nominal location, applied in the same direction at all floors and L_i is the floor dimension perpendicular to the seismic action direction...". The procedure is schematically shown in Window 3-12. The procedure is clearly cumbersome if applied to nonlinear analysis. It is routinely applied to linear analyses.

Window 3-12: Accidental eccentricity

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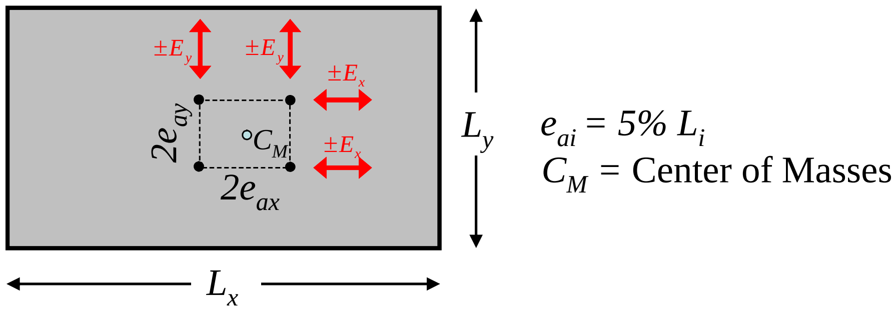

Application of loads to take into account accidental torsional effects

Window 3-12

An alternative approach is given by EC8 Part 1 §4.3.3.4.2.7 for nonlinear pushover analyses of structures which are torsionally flexible, where the first or second mode is torsional. It is suggested to consider torsional effects by multiplying the target displacements obtained from the pushover analyses by an amplification factor that results from an elastic modal analysis. No more information is provided by recent drafts of EC8. This approach refers to recent improvements by Fajfar to the original N2 method applied to asymmetrical building (Modified N2 method, (Fajfar, 2005)). The procedure proposed in (Fajfar, 2005) is schematically shown in Window 3-12 through Window 3-13. After performing the pushover analyses with applied loads in the x and y direction, two different pushover curves are obtained for the two analyses, and two target displacements at the top floor, D_{tx} and D_{ty}, are obtained (Window 3-12)

Window 3-13: Handling accidental eccentricity

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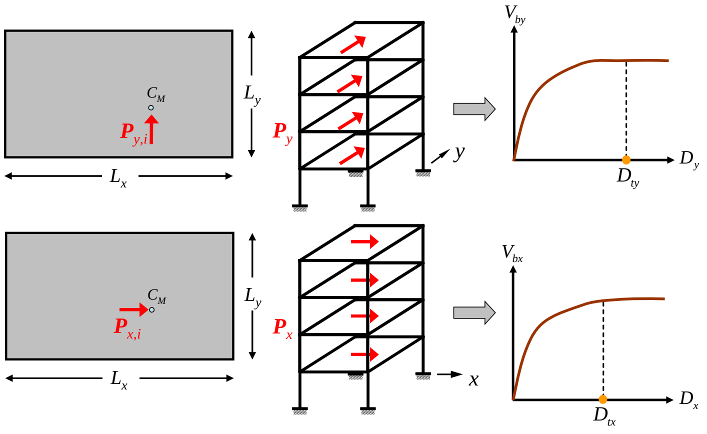
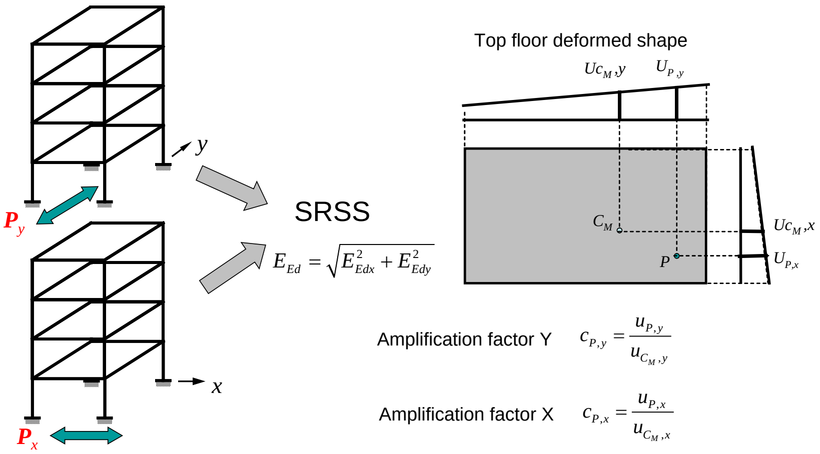
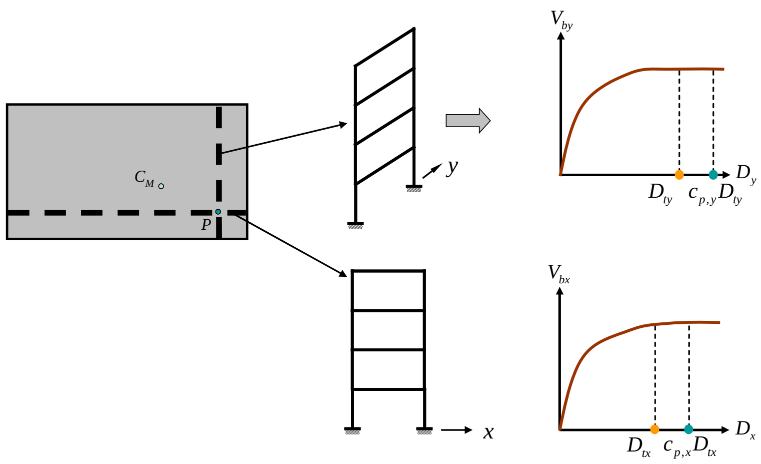


Pushover analyses in the x and y directions with loads applied at the center of mass



Top floor disp. from modal analysis and amplification factors at given point P



Amplification of the target displacements computed at the center of mass for a column with projection P on the top floor.

Window 3-13

The target displacements D_{tx} and D_{ty} are multiplied by and amplification factor obtained from the modal analysis of the building. This is schematically shown in Window3-13 Fig 2. Given for example a column passing through point P, amplification factors are computed for this point. The amplification factors $c_{P,x}$ and $c_{P,y}$, different for the x and y directions, are given by the ratios between the displacements of point P (($u_{P,x}$ and $u_{P,y}$ for the x and y directions, respectively) and the displacements of top floor center of mass ($u_{CM,x}$ and $u_{CM,y}$ for the x and y directions, respectively):

$$c_{P,x} = \frac{u_{P,x}}{u_{CM,x}} \qquad c_{P,y} = \frac{u_{P,y}}{u_{CM,y}}$$

The target displacements for which the column passing through point P is checked are then modified to the values $c_{P,x}\, D_{tx}$ and $c_{P,y}\, D_{ty}$. This is schematically shown in Window3-13 Fig 3. The actions (forces and/or deformations) used for the design checks of the column are those corresponding to the target displacements $c_{P,x}\, D_{tx}$ and $c_{P,y}\, D_{ty}$. There can be no de-amplification, thus $c_{P,x} \geq 1$ and $c_{P,y} \geq 1$. The actions in the x and y directions are then combined using the combination rules discussed in Section 3.5 of the present report.

If two planar models are used for the pushover analysis, i.e. the building has plan regularity, the torsional effects may be estimated using the following approach given in EC8 Part1 §4.3.3.2.4. The action effects (forces and/or deformations) in the individual load resisting elements resulting from the application of the pushover loads are multiplied by a factor δ.

$$\delta = 1 + 0.6\frac{x}{L}$$

where x is the distance of the planar frame under consideration from the center of mass of the building plan, measured perpendicularly to the seismic action direction considered, and Le is the distance between the two outmost lateral load resisting elements, measured perpendicularly to the seismic action direction considered.

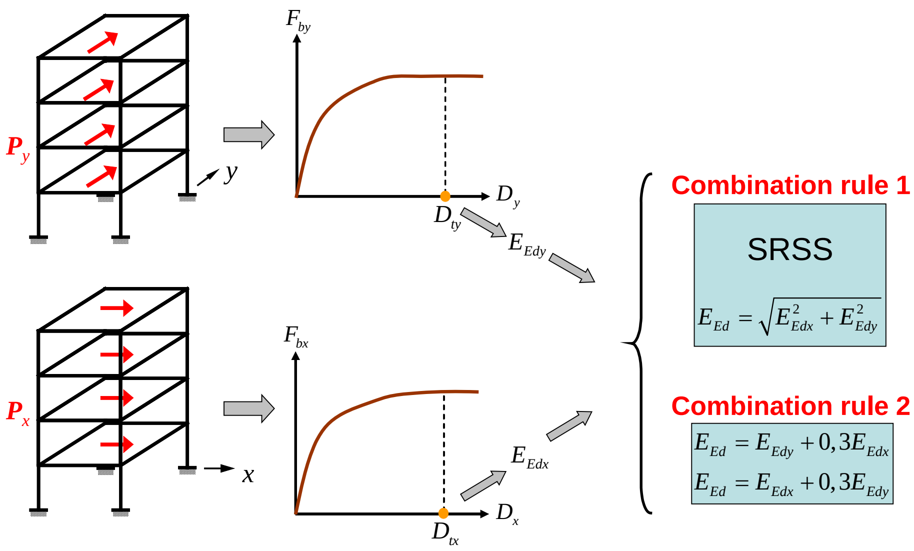
Recent EC8 drafts also indicate that for planar models, another procedure, outlined in EC8 Part 1 §4.3.3.3.3, can be applied, but this procedure, which implies the application of additional torsional moments to a spatial model of the structures, needs further studies before it can be used in regular practice.

3.5 Combination of the horizontal seismic action effects according to Eurocode 8

The pushover analysis described in Sections 3.1 and 3.2 refers to the application of the ground motion in a single direction. When the ground motion is applied in the two horizontal directions x and y, combination rules are given by EC8 (Part1, §4.3.3.5). EC8 gives two alternatives for the combination, as shown in Window 3-14.

Window 3-14: Combination rules

ZSoil®



EC8 Combination rules for horizontal seismic forces

Window 3-14

"... The structural response to each component shall be evaluated separately...". If E_{Edx} and E_{Edy} represent the action effects due to the application of the seismic action along the x and y directions, respectively, then the design value of each action may then be estimated by:

the SRSS rule:

$$E_{Ed} = \sqrt{E_{Edx}^2 + E_{Edy}^2}$$

the 100/30 rule:

$$E_{Ed} = E_{Edy}" + "0.3E_{Edx}$$
$$E_{Ed} = E_{Edx}" + "0.3E_{Edy}$$

In the second rule, " + " implies "to be combined with" according to EC8.

In nonlinear analysis, the action effects (as they are termed in EC8) may be either forces or deformations. For ductile mechanisms (such as flexure in beams) the action effects are the deformations (for flexure, plastic hinge rotations or curvatures), while for brittle mechanisms (such as shear) the action effects are forces (shear forces in the structural elements). EC8 gives some indications on how to check the seismic performance of a building based on nonlinear analysis results, but studies are still under way and a clear checking procedure is still missing.

Chapter 4

NONLINEAR MODELING IN EUROCODE 8

EC8 does not give any specific guidelines on how to consider material nonlinarities. Only general statements are made in EC8 Part 1 §4.3.3.4.1. The model shall include the strength of structural elements and their post-elastic behaviour. As a minimum, a bilinear force-deformation relationship should be used at the element level. For bilinear force-deformation relationship (typical of ductile mechanisms such as the bending moment – curvature in reinforced concrete beams) the elastic stiffness should be that of the cracked section and can be computed as the secant stiffness to the yield point. Zero post-yield stiffness may be assumed. Strength degradation may be included using more refined constitutive laws. These are minimum requirements for basic constitutive laws. For a fiber section model, for example, the section behaviour derives from the fibers' behaviour. Element properties should be based on mean values of the material properties. For new structures, mean values of the material properties may be estimated from the corresponding characteristic values on the basis of specific Eurocodes (for example EC2 for concrete).

EC8 Part 1 §4.4.2.2, which deals with resistance conditions at the Ultimate Limit State, provides guidelines on geometric nonlinearities. Second-order, or P-D effects, need not be taken into account if the following condition is satisfied:

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \leq 0.10$$

where θ is the interstorey drift sensitivity coefficient, P_{tot} is the total gravity load at and above the storey considered in the seismic design situation, d_r is the design interstorey drift, evaluated as the average lateral displacement d_s at the bottom of the storey under consideration and calculated in accordance with EC8 Part 1 §4.3.4, V_{tot} is the total seismic storey shear, h is the interstorey height. If $0.1 < \theta \leq 0.20$ the second order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1-\theta)$. Finally, the value of the interstorey drift sensitivity coefficient is limited by $\theta = 0.3$.

The above conditions relate to the ultimate limit state. It appears that these rules apply mainly to linear methods of analysis. For nonlinear analysis, no specifics are given on geometric nonlinearities and the published literature lacks studies on the importance of accounting for geometric nonlinearities in either pushover or time-history analyses. From limited published data and common experience it appears that for more flexible buildings in zones of high seismicity, geometric nonlinearities will have an important effect on the overall response at the ultimate limite state. For analyses at the collapse limit state, geometric nonlinearities should always be considered.

Chapter 5

Applications: Study of Bonefro buidling

The study of the seismic response of an existing reinforced concrete building which is analyzed using both nonlinear pushover and time history analyses results according to EC8 is presented.

An existing three-storey reinforced concrete building is studied using the nonlinear frame analysis capabilities of ZSoil®. The building is in Bonefro, Italy, and is a good example of residential buildings of the 70's and 80's in Italy, prior to the introduction of the seismic code in the early 80's.

5.1 Bonefro building modeling

Data file: BonefroPSH

Two models were created for the building, a general 3D model and a 2D model of a single frame(see Belgasmia &al., 2006). Details on materials, reinforcement, geometrical simplifications, and the nodal masses computed based on the loads given by for the 2D model are decribed in (Belgasmia &al., 2006):

Window 5-1: Bonefro building

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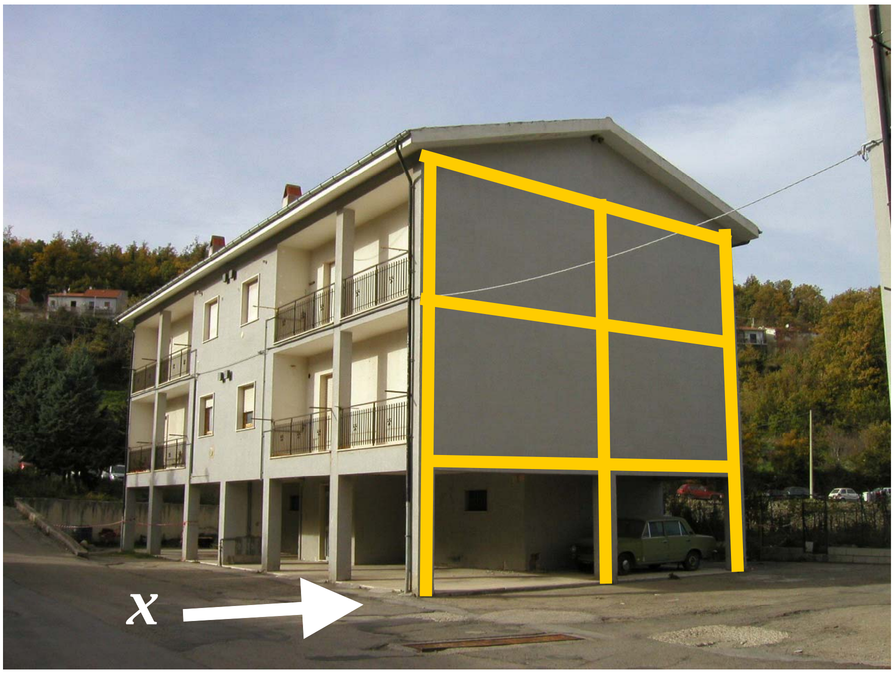
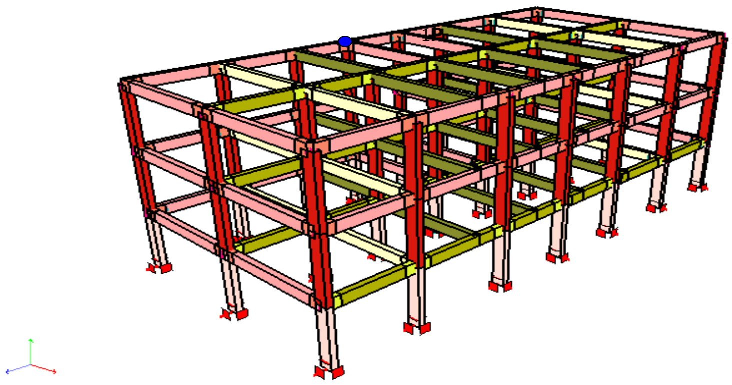


Bonefro reinforced concrete 3-storey building and corresponding 2D frame model



Bonefro building 3D model in ZSOIL

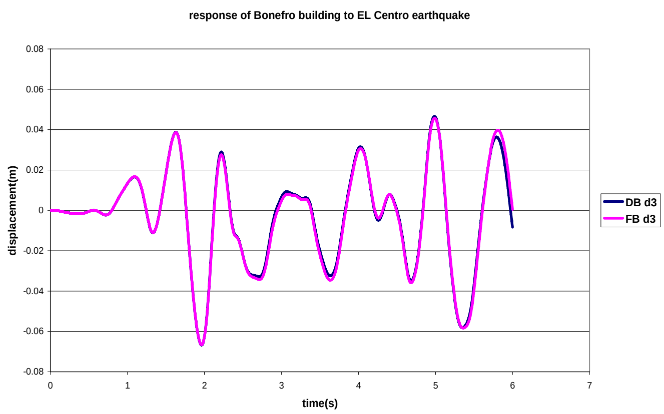
Window 5-1

5.2 Response of Bonefro building to ground acceleration, comparison of force and displacement based elements

The response of a single 2D frame is studied, only the side frame is considered. The response of the frame to the El Centro accelerogram is analyzed first, both using force-based and displacement-based elements. Six displacement-based elements in each beam and column were needed to converge to the solution obtained with force-based elements. One force-based element per column was used, while three elements were used throughout this report for the beams, because of the different reinforcement near the columns. Both types of elements show appropriate behavior.

Window 5-2: Bonefro building: Top floor response

ZSoil®



Top floor response to El Centro accelerogram

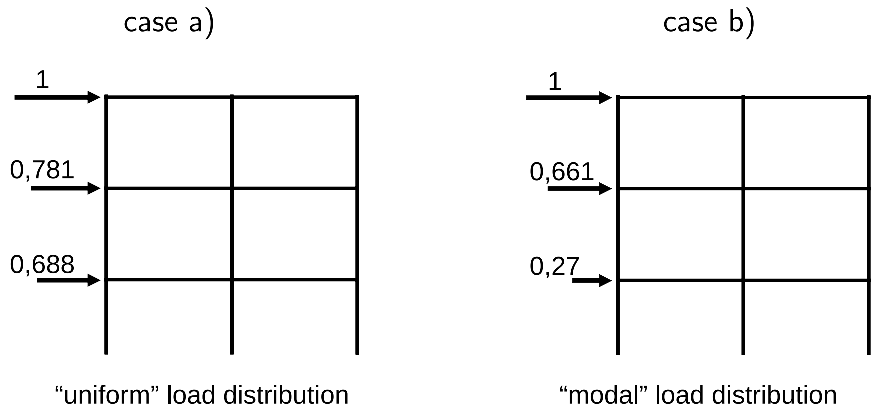
Window 5-2

5.3 Response of Bonefro building to 2D pushover analysis

The equivalent 2D frame is obtained by summing the modal masses, the steel reinforcement and the beam and column widths. The EC8 pushover procedure is applied to the 2D model, the frame is modeled with force-based elements. The load distributions are shown in Window 5-3. The "uniform" load distribution corresponds to case a), while the "modal" load distribution corresponds to case b). The symbols are those of defined earlier. The elastic response spectrum refers to a type 1 spectrum of EC8 Part 1 §3.2.2.2, ground type A ($S = 1$, $T_B = 0.15$ [sec], $T_C = 0.4$ [sec], $T_D = 2$ [sec]) 5% damping ($\eta = 1$) and $a_g = 0.15g$.

Window 5-3: Bonefro building: Load distributions

ZSoil®



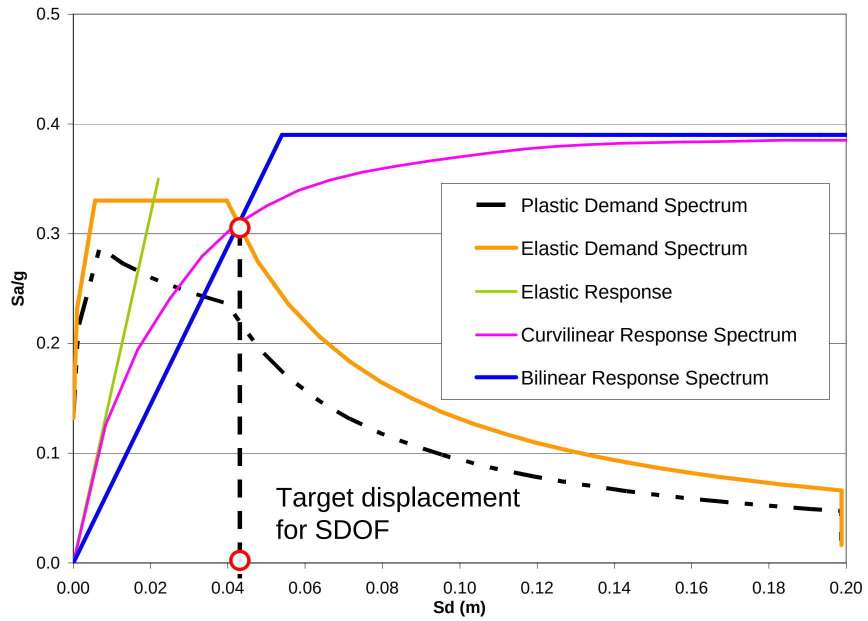
Load distributions used in the pushover analyses.

Window 5-3

Window 5-4 and Window 5-5 show the result of the pushover analyses for modal and uniform load distributions, respectively. In both cases the structure remains elastic.

Window 5-4: Bonefro building: Response and demand for modal load

ZSoil®

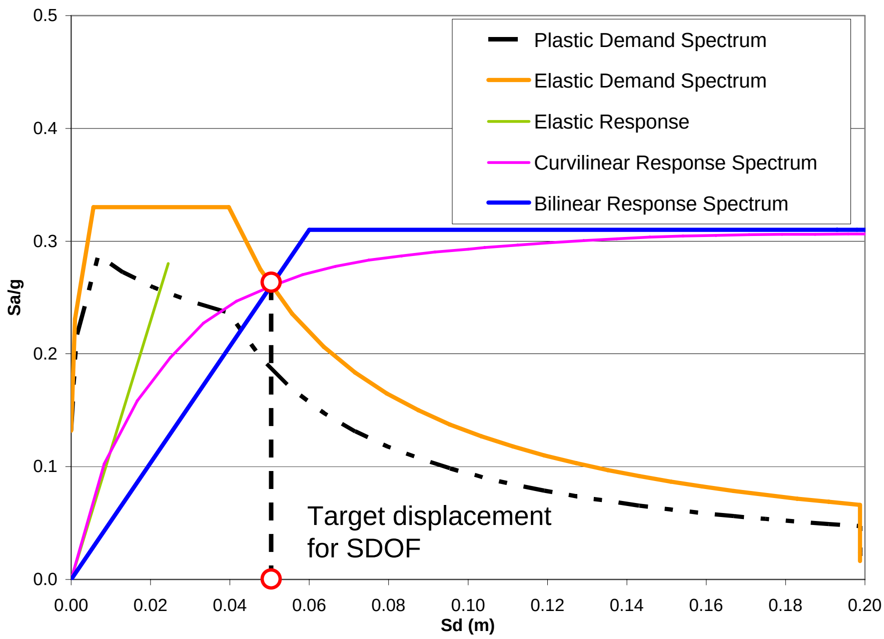


Determination of target displacement for modal load distribution. 2D case.

Window 5-4

Window 5-5: Bonefro building: Response and demand for uniform load

ZSoil®



Determination of target displacement for uniform load distribution. 2D case.

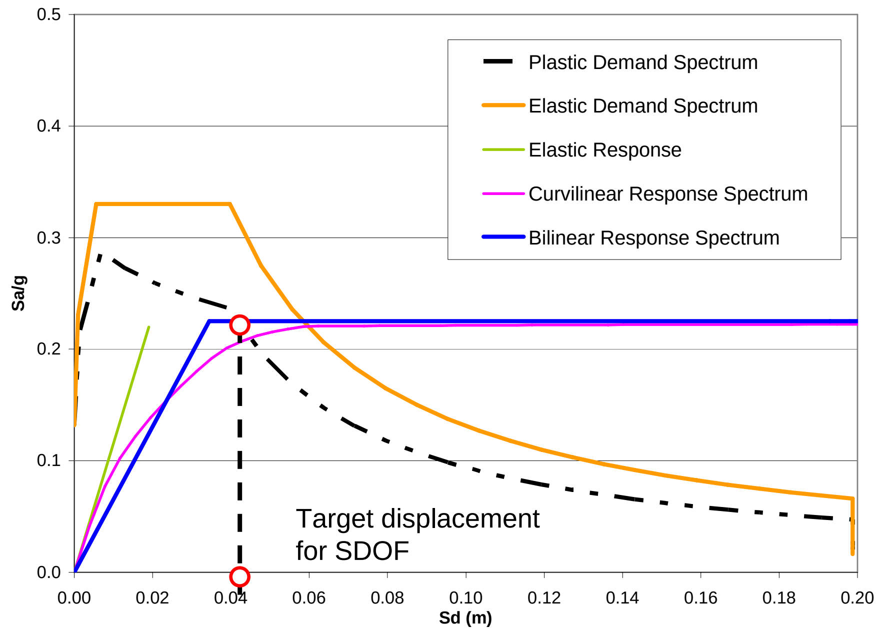
Window 5-5

5.4 Nonlinear pushover on 3D frame with ZSoil® (small displacements)

The target displacements for 2D & 3D model of Bonefro building in the case of the two loads distributions are summarized in a table in Window 5-9. These results are for a SDOF, in order to have the displacement of MDOF we must multiply the results of SDOF by Γ which is equal for 3D modal to 1.28 and for 2D modal to 1.2.

Window 5-6: Bonefro building (3D): Response and demand for modal load

ZSoil®

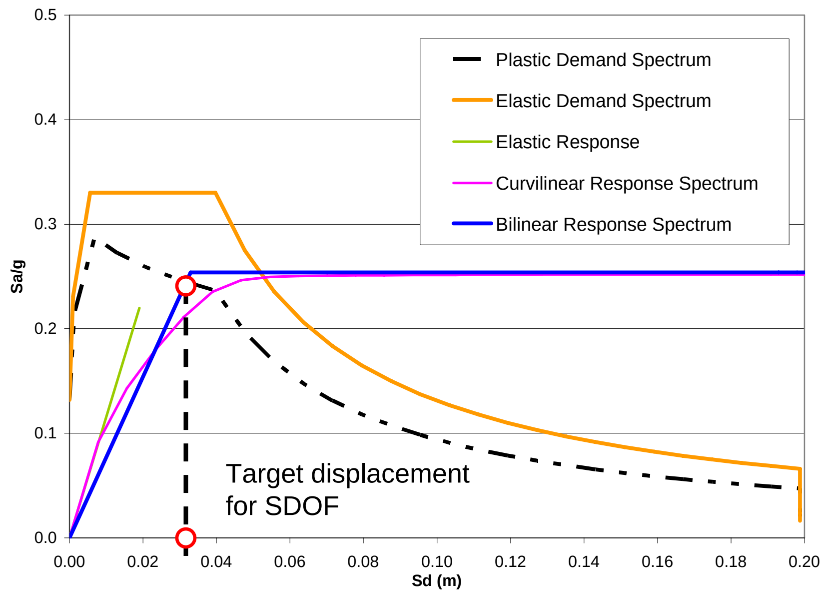


Determination of target displacement for modal loading in x direction.

Window 5-6

Window 5-7: Bonefro building (3D): Response and demand for uniform load)

ZSoil®

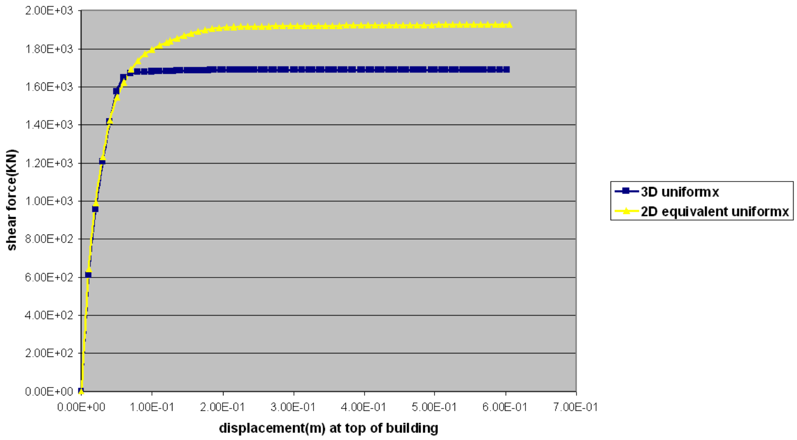


Determination of target displacement for uniform load distribution.

Window 5-7

Window 5-8: Bonefro building: Pushover curves (3D and 2D)

ZSoil®



Pushover curves of 3D model and 2D equivalent frame under uniform load distribution.

Window 5-8

Window 5-9: Bonefro building: Summary of pushover results

ZSoil®

Target displacements for:	2D	3D
Uniform load (SDOF)	0.051m	0.052m
Modal load (SDOF)	0.042m	0.06m
Uniform load (MDOF)	0.0612m	0.067m
Modal load (MDOF)	0.0504m	0.077m

Summary of pushover results for SDOF and MDOF system

Window 5-9

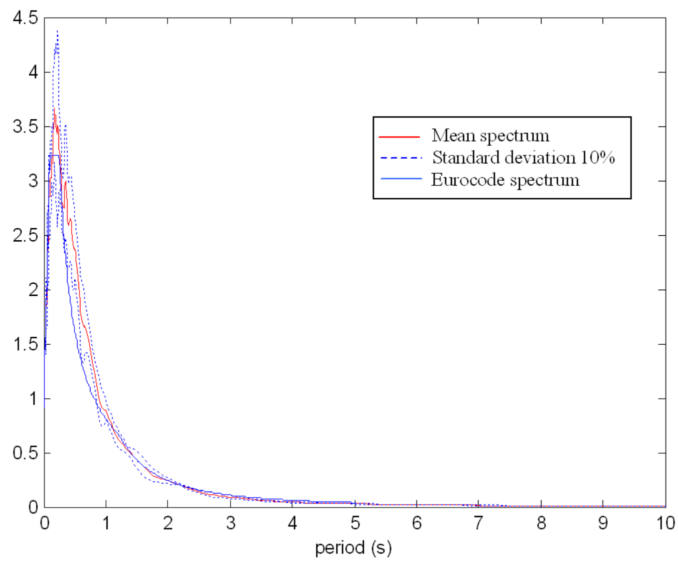
5.5 Simulated accelerograms

Three accelerograms compatible with the linear design spectrum are applied next. The three accelerograms (applied in one direction only – the x direction) - are simulated accelerograms generated with a computer program (Sabetta 0). The input parameters for this program are the epicentral distance, the earthquake magnitude and the type of soil. Sabetta program allows the generation of nonstationary artificial earthquakes according to an empirical method based on the regression of the relations of attenuation of a collection of earthquakes measured in Italy (95 accelerograms of 17 earthquakes magnitudes between 4.6 to 6.8). It generates a significant number of earthquakes and it takes the average of these earthquakes. The input parameters that yield an acceptable mean spectrum are: magnitude = 6.02, epicentral distance = 22.8 km, type of soil = shallow. The relevant mean spectrum is shown in

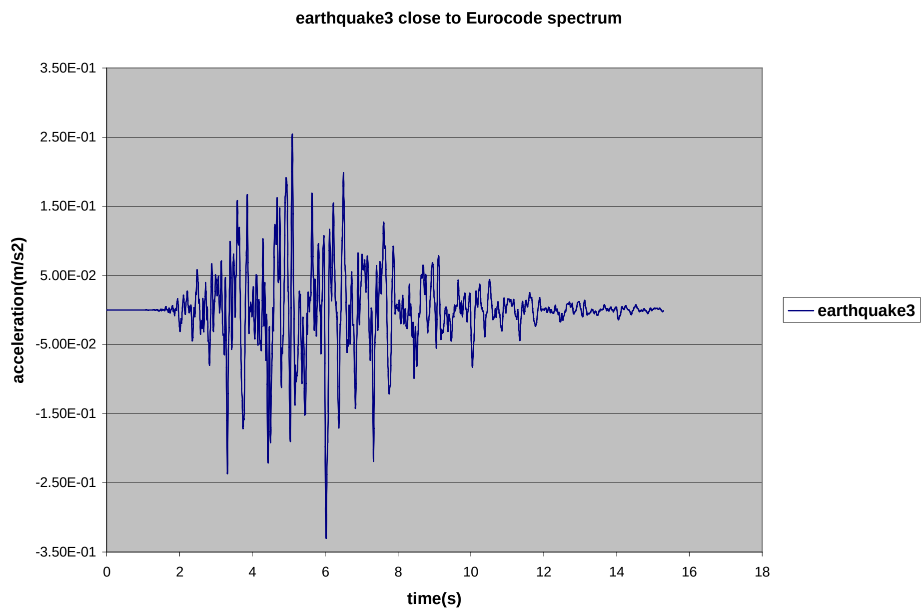
One of the 3 accelerograms is also shown in Window 5-10. The earthquake is scaled to 0,33g PGA.

Window 5-10: Simulated ground motions)

ZSoil®



Mean spectrum of 3 simulated ground motions



Generated accelerogram

Window 5-10

5.6 Nonlinear time history analysis of 3D model

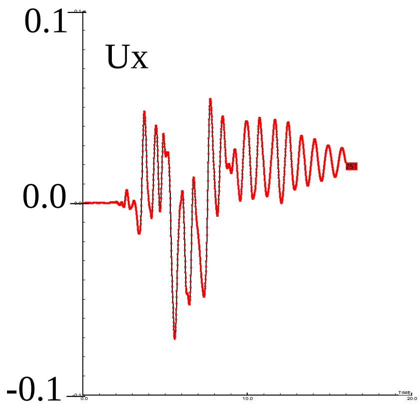
A time history analysis of the 3D model was performed (Belgasmia &al.) and selected results are presented next. The results for input ground motion applied in one direction only, the x direction, are compared for pushover and time-history analysis. Stiffness proportional

Rayleigh damping is prescribed, with 5% damping at 2 Hertz. The resulting values for Rayleigh damping are $\alpha = 0$ and $\beta = 0.008$. The top-floor response to the selected accelerogram is shown in Window 5-11, representing the maximum response out of 3 acceleration time-histories, as prescribed by EC8.

The maximum top floor displacement for the three earthquakes is 0.072 [m]. The target displacement for the pushover analysis is 0.077 [m].

Window 5-11: Response to simulated ground motions)

ZSoil®



Top floor response of 3D building to earthquake 3

Window 5-11

Pushover analysis provides the maximum base shear one can expect for a given maximum target displacement (corresponding to a given earthquake intensity). The time histories provide not only the maximum values, but the entire history. For the example at hand, the max displacement of the time history is $0.072m$, and the maximum base shear is approximately equal to 1500 kN, but during the time history this value of the base shear can be reached at several instances and for different values of the top displacement. On the pushover curve this translates into a single point that provides maximum displacement and maximum base shear that can be expected for that given earthquake.

5.7 Sensitivity to seismic parameters

The sensitivity of the time history analyses to the seismic parameters used in generating the accelerograms with the program Sabetta 0 is presented here. The three parameters: magnitude, epicenter distance and soil type were varied. The results are shown in the following tables. The reference values are magnitude MG= 6.02, epicenter distance ED = 22.8 km, shallow soil type. Following tables gives maximum displacement response under:

- modal load in x direction for static pushover analysis (PSH)
- earthquakes in x direction for Time history analysis (TH)

Window 5-12: Sensitivity to seismic parameters

ZSoil®

Variation of magnitude (MG), other parameters held at reference value:

case:	MG=5.72	MG=6.02	MG=6.32
PSH	.	$Max = 1.28 * 0.068m$ $Max = 0.077m$	.
TH	$Max = 0.06m$	$Max = 0.072m$	$Max = 0.106m$

Variation of epicentral distance (ED), other parameters held at reference value:

case:	ED=11.4km	ED=22.8km	ED=45.6km
PSH	.	$Max = 1.28 * 0.068m$ $Max = 0.077m$	.
TH	$Max = 0.065m$	$Max = 0.072m$	$Max = 0.095m$

Variation of soil type, other parameters held at reference value:

case:	shallow	deep
PSH	.	$Max = 1.28 * 0.068m$ $Max = 0.077m$
TH	$Max = 0.072m$	$Max = 0.065m$

Window 5-12

5.8 Sensitivity of response to large deformation

The pushover analyses were repeated using the large displacement analysis capabilities in ZSoil®. The pushover curve become strain softening after yielding, when geometric nonlinearities are accounted for. The increase in target displacement is approx. 3.3% for modal load distribution and and 11.5% for uniform load distribution.

5.9 Observations

After satisfactory comparison of flexibility and displacement based formulation, only the first one was used in this study.

- Comparison of 2D vs 3D results under imposed uniform horizontal displacement at the top of the building indicate that 2D and 3D models are equivalent in the elastic range, then a stiffer behavior of the 2D model, moderate in the early nonlinear behavior, significant (20%, Window 5-8) close to the maximum base shear. This seems to indicate a truly 3D resistance mode in the nonlinear range.

- 2D analysis indicates a larger target displacement for uniform loading vs modal loading (Windows 5-4 & 5-5) 0.051m against 0.042m.(reference point P structural displacements 0.061m vs 0.05m).

- 3D analysis, uniform loading vs modal loading, indicates the opposite trend, (Windows 5-6 & 5-7), 0.052m against 0.06m. (at P 0.067m vs 0.077m)
- Combining loading, 100% in x direction and 30% in z direction shows no influence on the pushover curve and hence on the target displacement.
- The comparison between pushover and dynamics gives a difference of 7% (dmax modal from pushover/ dmax dynamics) =0.077/0.072.
- The sensitivity to Magnitude, in dynamics a (+-5%) variation in EQ magnitude yields a variation in max displacement of (+47% and –17%).
- The sensitivity to epicenter distance, in dynamics (via Sabetta program) a (+100%, -50%) variation yields a variation in max displacement of (+31%, -10%).
- The sensitivity to soil, in dynamics (via Sabetta program): shallow/deep = 0.072/0.065=1.1 the difference is about 10%.
- The maximum base shear indicated by the pushover analysis is systematically reached for almost any maximum top displacement during dynamic, this is probably indicative of significant influence of 2nd 3rd etc modes.
- Large displacement induce global softening and increase target displacements in 3D pushover +11% for uniform loading 0.058/0.052, +3% for modal loading 0.062/0.06. No influence was noticed in dynamics.

Chapter 6

Data preparation for pushover analysis in ZSoil®

Data preparation for pushover analysis requires specification of the following data:

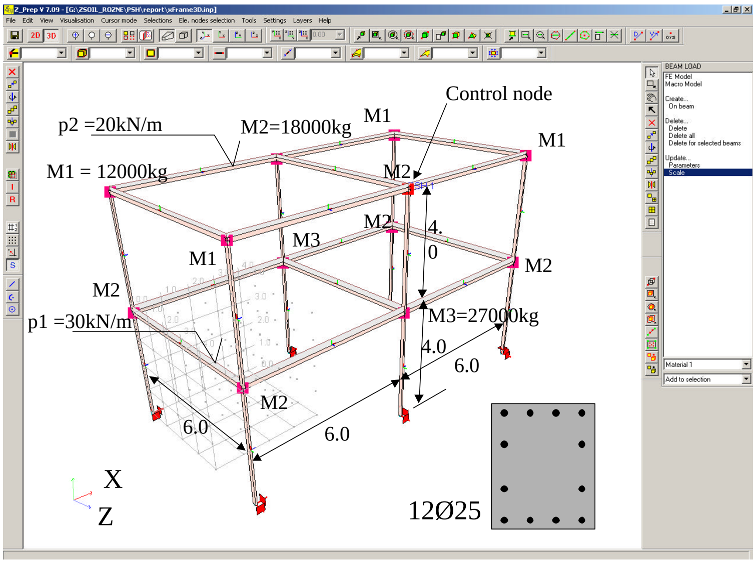
- Control phase:
  - ⋆ Activating driver(s)under *Menu/Control/Analysis and drivers*
  - ⋆ Setting control data for each driver under *Menu/Control/Pushover*
- Preprocessing phase:
  - ⋆ Introducing masses to the FE model of the structure. See Section 6.2
  - ⋆ Selecting and labeling control node i.e. node where target displacement is set. See Section 6.3

The example (file **xFrame3D.inp**) concerns a simple reinforced concrete 3D frame representing the skeleton of a two-storey building (with stiffness of the wall neglected).

The structure is shown in the Window 6-1

Window 6-1: Example of FE model)

ZSoil®



xFrame3D FE model. Masses, loads and control node

Window 6-1

For sake of presentation simplicity there is one reinforced concrete section assumed in all members with dimension $0.4 \times 0.3m$, made of concrete characterized by $f_c = 25MPa$, $f_t = 1.8MPa$, reinforced by symmetric reinforcement $10\varnothing25$ as shown.

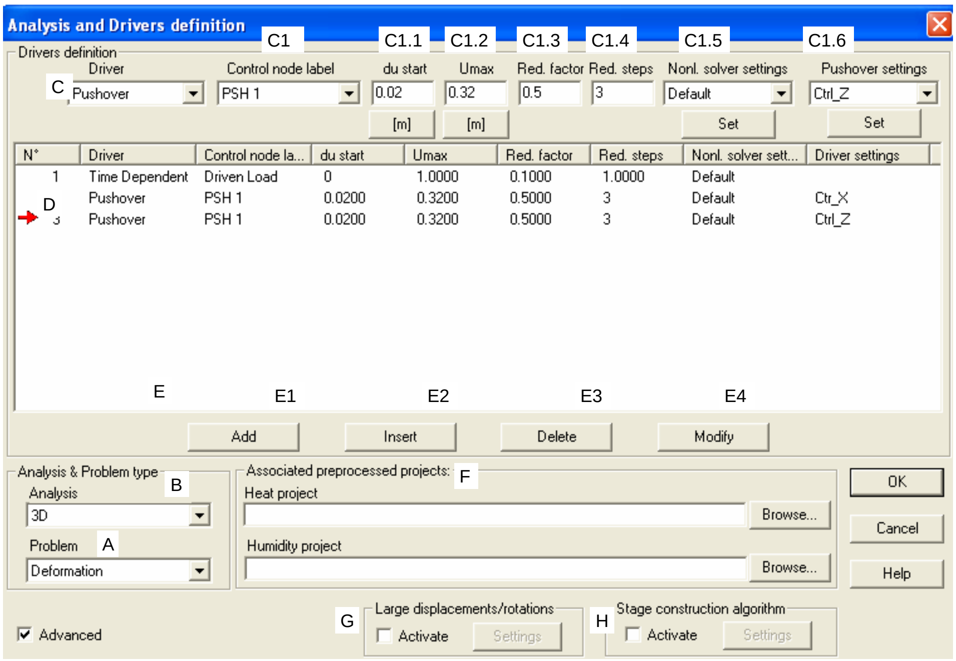
6.1 Control

Pushover analysis is a ZSoil® *driver* activated under `Control / Analysis & Drivers`. `Pushover` should be run after evaluation of the mechanical state of the structure corresponding to gravity load in a sequence of `Initial State` and/or `Time dependent`, `Driven Load Drivers`.

Window 6-2: Pushover

ZSoil®

A. `Problem type:` `Deformation`
B. `Analysis type:Plane Strain`, 3D (structures only)
C. `Driver:` `Pushover`
C1. `Control node label:` define new or use existing



Control data of the driver are:

Key	*Option*	*Comment*	*Default*	*Remarks*
C1				2,4,5
C1.1	`du start`	Initial value of displacement increment du	(0.2 [m])	3
C1.2	`Umax`	Final value of du	(1.0[m])	3
C1.3	`Red. factor`	du reduction multiplier	(0.5)	8
C1.4	`Red. steps`	Nr of du reduction steps	(3)	9
C1.5	`Nonl. Solver`	Control data for nonlinear statics		
C1.6	`Dyn. Solver`	Control data for pushover direction		

Window 6-2

Remarks:

1. Model must have nonzero mass (masses are defined as element and/or nodal)
2. `Driver type: Control node` **must** be set during `preprocessing`.
3. Recommended value of `Umax` is 4% of structure height roughly. To make pushover curve sufficiently smooth use `du start` $<$ `Umax`/10
4. A sequence of `Pushover Drivers` can be defined, with different `Control` and `Pushover Settings` specification.
5. `Pushover` can be run within a sequence of `Time dependent`, `Driven Load Drivers`

6. The state of the structure at start of a `pushover` driver takes into account results (e.g. plastic status, stresses, deformation)) of previous `Time dependent` driver(s).
7. Changes of the structure state (stresses, deformation, plastic status) induced by `pushover` driver are **disregarded** during subsequent drivers of any type
8. Step reduction procedure is automatically performed in case when applying initially set du causes divergence.
9. When convergence is not reached after applying displacement increment reduced `Red. steps` times from its initial value du, execution of the static procedure is terminated before reaching final value `Umax`.
10. Large deformation analysis may be activated.
11. After running `pushover` driver use `Postprocessing/Pushover Result` to perform seismic demand assessment automatically.

Other data to be defined are:

- Nonlinear solver settings (use `Control` / `Control`)
  The only data taken into account during pushover analysis are:
  - ⋆ *Tolerance for solid phase RHS*
  - ⋆ *Absolute max. nr of iterations* -if reached, the step length reduction is performed
- Pushover settings: (use `Control` / `Pushover`)
  - ⋆ *Label* can be given to ease identification of the given set in case of multiple use in one job,
  - ⋆ *Mass filtering* (if on) will be performed in specified direction
  - ⋆ *Direction* - the vector introduced will be used to set: pushover force direction, control displacement direction, and mass filtering (if activated),
  - ⋆ *Force pattern*- selection between *Modal*, *Uniform* or *Triangular*, see Window 3-1

6.2 Modeling of masses

ZSoil® offers different possibilities to model masses present in the structure. They include:

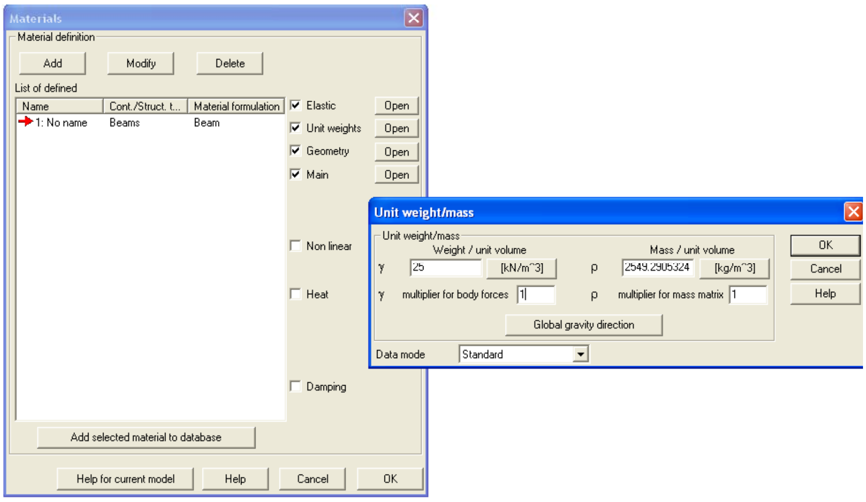
- Element masses
  They are related to the material density and geometry of the structural element itself. In each element type they have to be defined as a part of material model attached to the element. Their existence at a given instant of the analysis is controlled by the element itself.
- Added masses
  They can be added independently from element description, their activity (existence) can be controlled by existence and load functions. They can be attached to:
  - ⋆ a node (by specifying total nodal mass),
  - ⋆ an edge or existing beam and truss (by specifying linear mass density),
  - ⋆ a face of a shell (by specifying surface mass density).

In ZSoil® 2011 lumped (diagonal) or consistent mass matrix is created in each of the above cases.

In the example **xFrame3D** there are element masses originating from the element dead weight set during material (i.e. cross-section properties) definition, see the figure as well as nodal masses (due to mass of walls and floors other than mass of frame member itself) attached to each node of the 1-st and 2-nf floor set during Prepro set under *FE model / Added masses / Nodal mass.*

Window 6-3: Element masses

ZSoil®



Setting element masses

Window 6-3

6.3 Selection of a control node

If *pushover* analysis is to be run, a control node must be selected (in interactive graphical manner) during pre-processing phase. This must be done prior to activation of *Pushover Drivers* from the *Menu/Control /Analysis & Drivers.* The node should be labeled.

In the example **xFrame3D** there is one control node set under *Prepro* option *Domain/Pushover control node,* labeled as PSH_1, see the figure.

6.4 Results

Section describe actions specific to pushover result handling. These include:

- Automated seismic demand assessment under *Menu/Results/Pushover results*, see Section 6.5
- Tracing structural performance under graphical postprocessor under *Menu/Results/Postprocessing*, see Section 6.6.

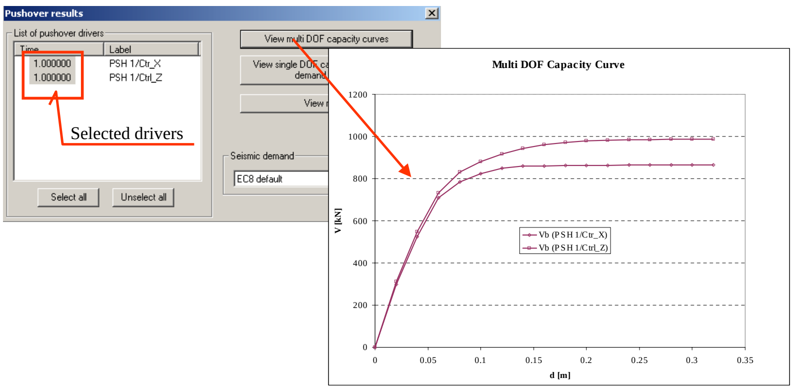
6.5 Pushover results

After running computational part of pushover analysis, seismic demand assessment can be done automatically (see Appemdix B) under *Menu/Results/Pushover results* option. Each of the available actions will invoke EXCEL spreadsheet containing numerical results together with their graphical presentation:

- *View MDOF capacity curve* (for the single driver or multiple selection of drivers, irrelevant to active demand spectra), see the Window 6-4.
- *View SDOF capacity curve and demand spectra* (for the single driver or multiple selection of drivers and for the active demand spectra). Examples together with some comments are given in the Figs. 6-6,.6-7, 6-8
- *View Report*(for the single driver or multiple selection of drivers and for the active demand spectra), EXCEL table containing results as shown at the right of the Figs. 6-6, 6-7, 6-8 will be created.
- Seismic demand activation and edition.

Window 6-4: MDOF capacity curve)

ZSoil®



Invoking MDOF capacity curve

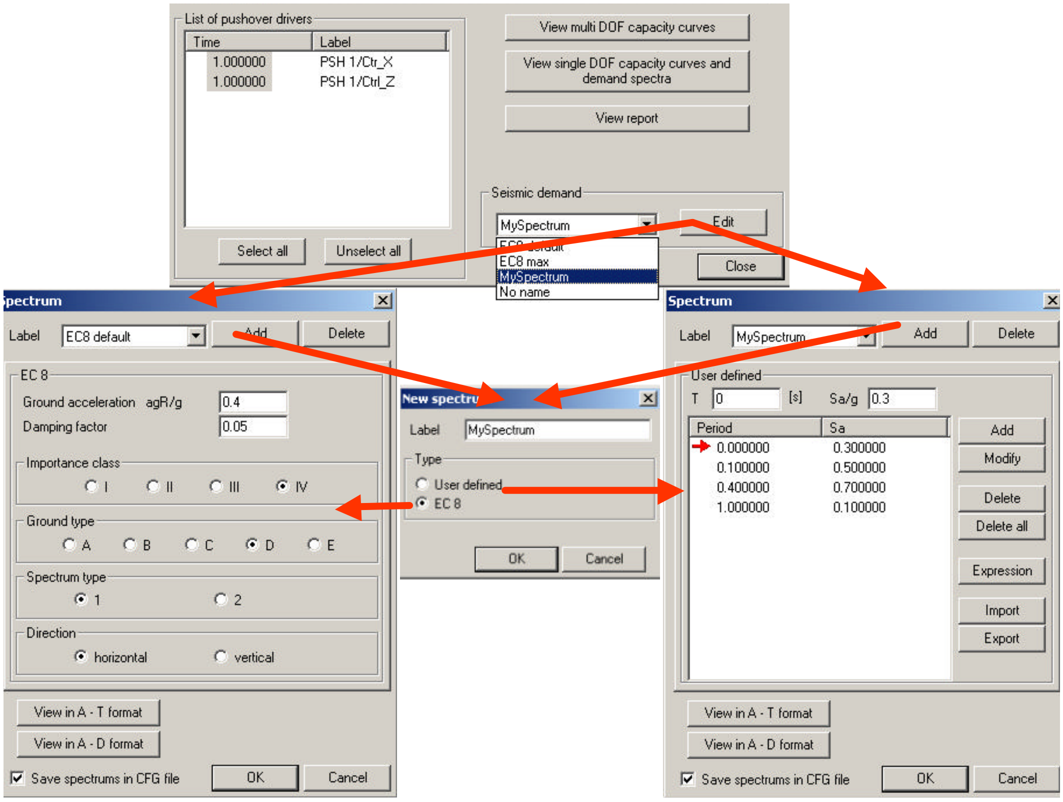
Window 6-4

In ZSoil® demand spectra can be created as conforming strictly to EC8 regulation and specified data items, or can be entered in a custom mode as pairs $(T_i, A_i/g)$. Once created spectra are stored (in a Pushover.CFG file) for multiple use. Button *Edit* opens selected spectrum for edition and modification. Button *Add* should be used in order to open new spectrum. Prior to entering *Spectrum* edition dialog mode *EC8* or *User* has to be set in appearing *New Spectrum* dialog.

Elastic demand spectra can be viewed both in *A-T*(pseudo-acceleration w.r.t period) or *A-D* (pseudo-acceleration w.r.t relative displacement) format.

Window 6-5: SDOF Demand spectra)

ZSoil®



Handling of demand spectra under *Menu/Results/Pushover results*

Window 6-5

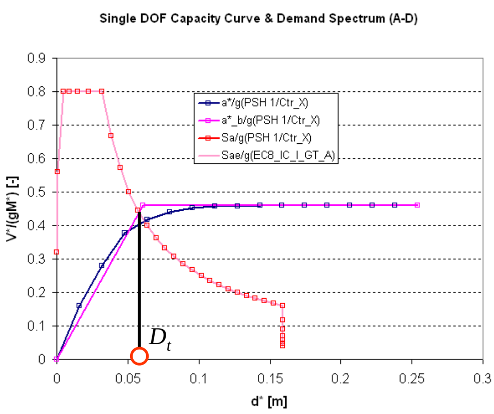
Having set the elastic demand spectrum, inelastic demand spectrum related to given capacity spectrum can be set leading to evaluation of displacement demand (or target displacement). Typical relations between demand spectrum and capacity spectrum are shown in the Windows 6-6, 6-7, 6-8. All cases concern results of pushover analysis labeled as CTRL_X driver in xFrame.inp, but here they are treated in conjunction with different EC8 demand spectra, as specified in the table:

Fig.:	Importance class	Ground type
6-6	I	A
6-7	IV	A
6-8	IV	D

Remaining EC8 demand spectrum data are the same, including *ground acceleration* $a_g = 0.4$, damping factor $\eta = 0.05$, *spectrum type 1,* and *direction: horizontal.*

Window 6-6: Case of: importance class I, ground type A)

ZSoil®



		Pushover analysis report
Item	Unit	PSH 1/Ctr_X
MDOF Free vibr. period........T	[s]	0.5714
SDOF Free vibr. period.......T*	[s]	0.7281
SDOF equivalent mass.........M*	[kg]	151997
Mass participation factor Gamma	-	1.2593
Bilinear yield force value..Fy*	[kN]	687.07
Bilinear displ. at yield....Dy*	[m]	0.0607
Target displacement.........Dm*	[m]	0.2541
SDOF displacement demand....Dt	[m]	0.0607
Energy......................Em*	[kN*m]	153.74
Reduction factor............qu	-	1
Demand ductility factor......mi	-	1
Capacity ductility factor...miC	-	4.1861
MDOF displacement demand.....D	[m]	0.0764

Purely elastic response in the range of seismic demand. No reduction of demand spectrum.

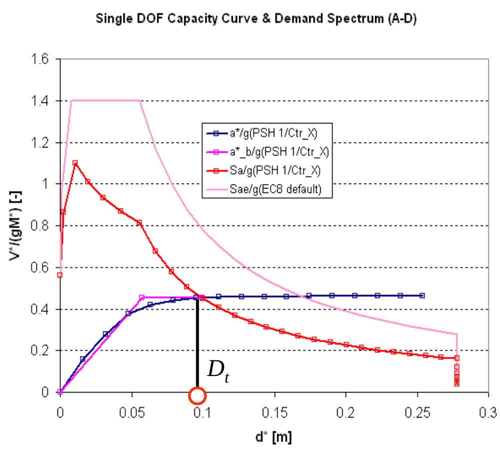
Window 6-6

Both cases Window 6-6 and 6-7 are successful, in the sense that target displacement was found within the initially assumed range of analysis $D_t < U_{\max}$. If so, evaluation of the structural performance at the state corresponding to the evaluated value D_t can be performed.

In the last case (Window 6-8), maximum value of control displacement *Umax* was to small. In this case a warning: "CAPACITY CURVE IS TOO SHORT" is issued both in the graph description field and in the report. If it was only due to initial user assumption of *Umax* it is enough to re-run the analysis with bigger *Umax*. If it was due to limited possibility of the structure to sustain bigger values of *Umax* appropriate design conclusions should be drawn.

Window 6-7: Case of: importance class IV, ground type A)

ZSoil®



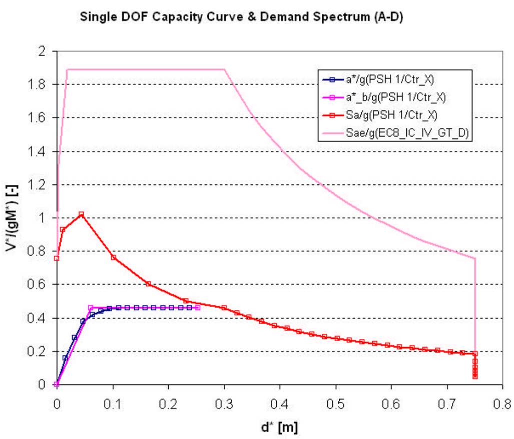
		Pushover analysis report
Item	Unit	PSH 1/Ctr_X
MDOF Free vibr. period........T	[s]	0.5714
SDOF Free vibr. period.......T*	[s]	0.7148
SDOF equivalent mass.........M*	[kg]	151997
Mass participation factor Gamma	-	1.2593
Bilinear yield force value..Fy*	[kN]	676.96
Bilinear displ. at yield....Dy*	[m]	0.0576
Target displacement.........Dm*	[m]	0.0995
SDOF displacement demand....Dt	[m]	0.0995
Energy......................Em*	[kN*m]	47.854
Reduction factor............qu	-	1.7266
Demand ductility factor......mi	-	1.7266
Capacity ductility factor...miC	-	1.7265
MDOF displacement demand.....D	[m]	0.1253

Plastic response with ductilty reduced demand spectrum. Target displacement found within the range of capacity spectrum. $(D_t < D_m)$

Window 6-7

Window 6-8: Case of: importance class IV, ground type D)

ZSoil®



	Pushover analysis report	
Item	Unit	PSH 1/Ctr_X
MDOF Free vibr. period........T	[s]	0.5714
SDOF Free vibr. period.......T*	[s]	0.7281
SDOF equivalent mass.........M*	[kg]	151997
Mass participation factor Gamma	-	1.2593
Bilinear yield force value..Fy*	[kN]	687.07
Bilinear displ. at yield....Dy*	[m]	0.0607
Target displacement.........Dm*	[m]	0.2541
SDOF displacement demand....Dt*	[m]	0.2812
Energy......................Em*	[kN*m]	153.74
Reduction factor.............qu	-	4.1003
Demand ductility factor......mi	-	4.4063
Capacity ductility factor...miC	-	4.1861
MDOF displacement demand.....Dt	[m]	0.3542

Plastic response with ductility reduced demand spectrum. Target displacement not found within the range of capacity spectrum ($D_m < D_t$). The warning: "CAPACITY CURVE IS TOO SHORT" is issued. Analysis should be re run with greater values of *Umax*

Window 6-8

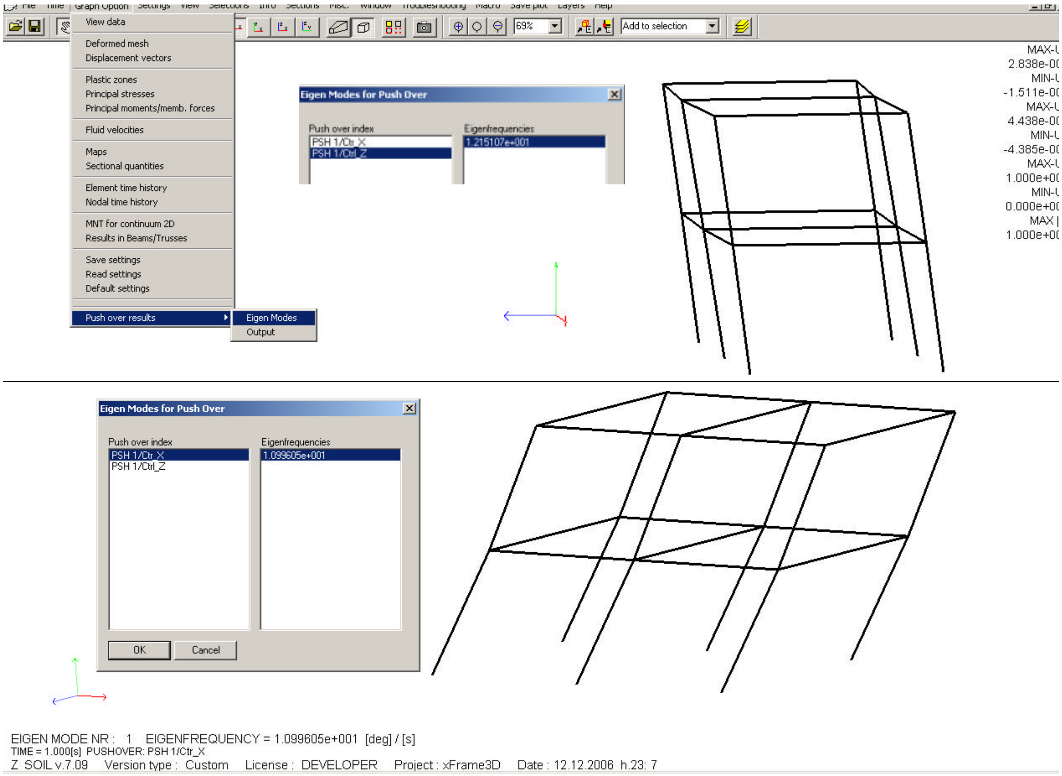
6.6 Postprocessing

Postprocessing for Pushover analysis, beside of all standard ZSoil® capabilities displays the modal deformation and eigen-frequency. This can be done under: *Graph option/Pushover results/Eigen modes*.

After evaluation of target displacement during seismic demand assessment automatic procedure (performed under *Menu/Results/Pushover results*), all static analysis results (including deformation, internal forces), giving hints on performance of the structure under given seismic event, can be traced at the demanded level of control displacement, closest to the particular value of D_t. For example, for the case from Fig.6-7, target displacement is equal to $D_t = 0.1256m$. The closest value of control node displacement i.e. *Pushover U-ctrl* (in that case $= 0.120m$) has to be set in the dialog appearing under *Time/Select Current Time Step*.

Window 6-9: Modal results

ZSoil®



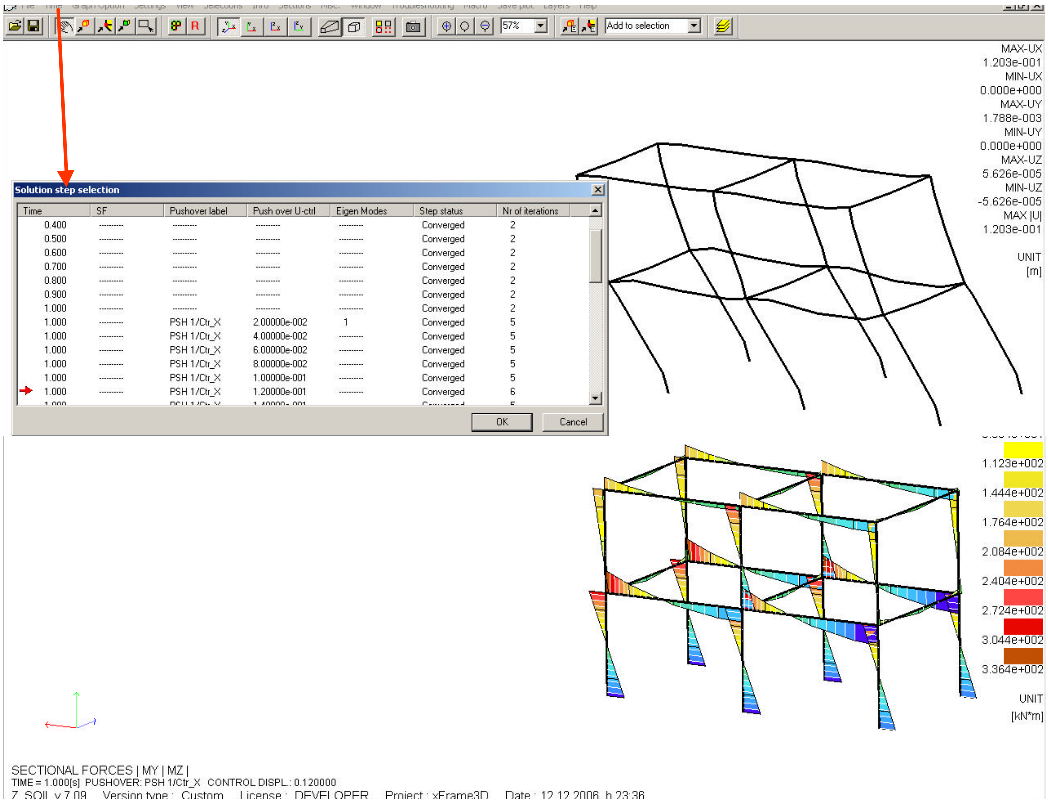
xFrame.inp. Modal results including eigen-modes and eigen-frequencies for X and Z pushover directions

Window 6-9

Note, the difference between the deformation for flexibility beam elements in modal and standard case. Modal deformation display is built on nodal displacement exclusively, while standard deformation include deflection alongside elements.

Window 6-10: Static results

ZSoil®



xFrame.inp. Static results including deformation and bending moments for X-direction ad $U_{ctrl} = 0.12$ [m]

Window 6-10

References

1. S. Antoniou and R. Pinho, "Advantages and Limitations of Adaptive and Non-adaptive Force-Based Pushover Procedures", Journal of Earthquake Engineering, Vol. 8, No. 4 (2004) 497-522.

2. M.N. Aydinoglu, "An Incremental Response Spectrum Analysis Procedure Based on Inelastic Spectral Deformation for Multi-Mode Seismic Performance Evaluation", Bulletin of Earthquake Engineering, 1 (2003), 3-36.

3. M.N. Aydinoglu, "An Improved Pushover Procedure for Engineering Practice: Incremental Response Spectrum Analysis (IRSA)" in Performance-based seismic design. Concepts and implementation, Krawinkler and Fajfar eds., Bled, Slovenia, June 28 – July 1, PEER report 2004/05, 345-356.

4. M.Belgasmia, E. Spacone, A.Urbanski, Th. Zimmermann. "Seismic Evaluation of Constructions: Static Pushover procedure and time history analysis for Nonlinear Frames", LSC Internal Report, August 2006.

5. J.M. Biggs, "Introduction in structural dynamics."1964.

6. A.K. Chopra, "Dynamics of Structures: Theory and Applications to Earthquake Engineering" 2nd Ed., Prentice Hall, Upper Saddle River, NJ, 2001.

7. A.K. Chopra, R.K. Goel, "A modal pushover analysis procedure for estimating seismic demands for buildings." Earthquake Engineering and Structural Dynamics 2002, 31:561-582.

8. A.S. Elnashai "Do we really need Inelastic Dynamic Analysis?" Journal of Earthquake Engineering, Vol. 6, Special Issue 1 (2002) 123-130.

9. Eurocode 8: Design of Structures for Earthquake Resistance, European Committee for Standardization, 2003.

10. P. Fajfar, "Capacity Spectrum Method Based on Inelastic Demand Spectra." Earthquake Engineering and Structural Dynamics, 28. 979-993, 1999.

11. P. Fajfar, "Structural Analysis in Earthquake Engineering – A Breakthrough of Simplified Non-Linear Methods." Proc. 12th European Conference on Earthquake Engineering, Paper 843, 2002.

12. P. Fajfar, V. Kilar, D. Marusic, I. Perus, "The extension of the N2 method to asymmetric buildings." Proceedings of the 4th European Workshop on the Seismic Behaviour of Irregular and Complex Structures, Paper No. 41, Thessaloniki, Greece, 2005.

13. B. Gupta, S.K. Kunnath, "Adaptive Spectra-Based Pushover Procedure for Seismic Evaluation of Structures", Earthquake Spectra, 16(2), 367-391.

14. F. Sabetta, A. Pugliese, Estimation of Response Spectra and Simulation of Nonstationary Earthquake Ground Motions, Bulletin of the Seismological Society o America, 86(2) 337-352, 1996.

15. E. Spacone, F.C. Filippou, F.F. Taucer, "Fiber Beam-Column Model for Nonlinear Analysis of R/C Frames. I: Formulation, II: Applications." Earthquake Engineering and Structural Dynamics, 25(7), 711-742, 1996.

16. T. Vidic, P. Fajfar, M. Fischinger, "Consistent Inelastic Design Spectra: Strength and Displacement." Earthquake Engineering and Structural Dynamics, 1994, 23: 502-521.

17. Th. Zimmermann, A. Truty, A. Urbanski, K. Podles. Z-Soil user manual, Zace Services Ltd, 1985-2006.

Appendix

A1. Seismic demand assesment algorithm

A. Given D_m^*, F_y^* from the bilinearized capacity spectrum:

B. Set:

$$D_y^* = 2\left(D_m^* - \frac{E_m^*}{F_y^*}\right); \quad T^* = 2\pi\sqrt{\frac{m^* D_y^*}{F_y^*}}; \qquad q_\mu = \frac{S_{Ae}(T^*)}{\frac{F_y^*}{m^*}}$$

C. Reduce spectrum:

if $(T^* < T_C)$

$q_\mu \leq 1 \Rightarrow D_t^* = S_{De}(T^*)$ response remains elastic

$$q_\mu > 1 \Rightarrow D_t^* = \frac{S_{De}(T^*)}{q_\mu}\mu; \quad \text{with } \mu = 1 + (q_\mu - 1)\frac{T_C}{T^*}$$

else $(T^* \geq T_C)$

$q_\mu \leq 1 \Rightarrow D_t^* = S_{De}(T^*)$ response remains elastic

$$q_\mu > 1 \Rightarrow D_t^* = \frac{S_{De}(T^*)}{q_\mu}\mu; \quad \text{with } \mu = q$$

D. Check:

if $|D_m^* - D_t^*| < tol$ stop

else

set : $D_m^* = D_t^*$,

go to A

A2. Nonlinear static algorithm with displacement control

Problem to be solved at each step n:

Given:

$\boldsymbol{\sigma}_{n-1}$ (previous stress state),

$\mathbf{f}_{n-1}$ (previous ext. forces),

$\mathbf{f}_0$ (pushover load pattern),

ΔD^{Ctrl} (control displacement increment).

Find: $\Delta\mathbf{u}$, $\Delta\lambda$ such that:

$$\begin{gathered}\mathbf{N}(\boldsymbol{\sigma}_{n-1}, \Delta\mathbf{u}) = \mathbf{f}_{n-1} + \Delta\lambda\mathbf{f}_0 \\ \mathbf{a}^T\Delta\mathbf{u} = \Delta D^{Ctrl}\end{gathered}$$

Initialize: $k = 0$, $\Delta\mathbf{u}^k = \mathbf{0}$; $\Delta\lambda^k = 0$.

Loop For $k = 1$; $k < MAXITER$, do:

solve for $\delta\mathbf{u}$, $\delta\lambda$:

$$\begin{gathered}\mathbf{K}_T\delta\mathbf{u} - \mathbf{f}_0\delta\lambda = \mathbf{f}_{n-1} + \Delta\lambda^{k-1}\mathbf{f}_0 - \mathbf{N}(\boldsymbol{\sigma}_{n-1}, \Delta\mathbf{u}^{k-1}) \equiv \boldsymbol{\psi}^{k-1} \\ \mathbf{a}^T\delta\mathbf{u} = \Delta D^{Ctrl} - \mathbf{a}^T\Delta\mathbf{u}^{k-1} \equiv p^{k-1}\end{gathered}$$

thus:

$$\begin{aligned}\delta\lambda &= \frac{p^{k-1} - \mathbf{a}^T\delta\widehat{\mathbf{u}}}{\mathbf{a}^T\widehat{\mathbf{u}}_0} \\ \delta\mathbf{u} &= \delta\widehat{\mathbf{u}} + \delta\lambda\widehat{\mathbf{u}}_0\end{aligned}$$

where:

$$\begin{aligned}\widehat{\mathbf{u}}_0 &= \mathbf{K}_T^{-1}\mathbf{f}_0 \\ \delta\widehat{\mathbf{u}} &= \mathbf{K}_T^{-1}\boldsymbol{\psi}^{k-1}\end{aligned}$$

update:

$$\begin{aligned}\Delta\mathbf{u}^k &= \Delta\mathbf{u}^{k-1} + \delta\mathbf{u}, \\ \Delta\lambda^k &= \Delta\lambda^{k-1} + \delta\lambda.\end{aligned}$$

if $\left\|\boldsymbol{\psi}^k\right\| < TOL:\ \ \Delta\lambda = \Delta\lambda^k;\ \Delta\mathbf{u} = \Delta\mathbf{u}^k$ stop

else $k = k + 1$

end do

A3. Step length adjustment algorithm

Step length adjustment algorithm for step n

$\Delta D^{Ctrl} = dD_0^{Ctrl}$

for $ired = 1$; $ired < Nred$, do
given: $\Delta D^{Ctrl}, : \mathbf{f}_{n-1}, : \mathbf{f}_0$.
Solve for: $\Delta\lambda, : \Delta\mathbf{u}$, see Appendix A2.
if (converged before MAXITER):
::: $\mathbf{f}_n = \mathbf{f}_{n-1} + \Delta\lambda\mathbf{f}_0$, take next step:: $n := n + 1$
else (diverged after MAXITER):
::: $\Delta D^{Ctrl} = \mu\Delta D^{Ctrl}$
if($ired \leq Nred :: ired := ired + 1$)else Stop
end do