

Evaluation of the ductility demand in partial strength steel structures in seismic areas using non-linear static analysis

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ABSTRACT: The performance of a structural system can be evaluated resorting to non-linear static analysis, also commonly referred to as Pushover Analysis, because of the nature of application of lateral loads while defining the capacity of the structure. This analysis involves the estimation of the structural strength and deformation demands and the comparison with the available capacities at desired performance levels. This paper aims at evaluating the seismic response of three steel structures using non-linear static analyses, based on the N2 method, adopted by Eurocode 8, the European seismic code. The results are compared with the ones evaluated by means of non-linear dynamic analyses. As a conclusion it can be ascertained that the non-linear static procedures are an alternative tool in seismic assessment and design, over just a force-based design. For structures that vibrate mainly in the fundamental mode, the pushover analysis applied in this work will provide good estimates of global, as well as local inelastic deformation demands.

Keywords: Steel Structures, Non-linear Static Analysis, Pushover analysis, Non-linear Dynamic Analysis, Seismic Behaviour

1 INTRODUCTION

Non-linear dynamic analyses (time history analyses) should be the preferred method to estimate the seismic demands, if such analyses are implemented prudently. Thus, they frequently are used as a verification tool of the non-linear static analyses at this developmental stage. Nevertheless, there are still some reservations to adopt non-linear dynamic analysis, which are mainly related to its complexity and suitability for practical design applications. Moreover, this type of analysis is very much sensitive to the characteristics of the input motions and thus selection of representative accelerograms is fundamental. Besides, the hysteretic behaviour of all the critical sections should be carefully defined. Finally, the efforts in computation and assimilation of results, contribute to avoid practical design utilization.

The pushover analysis is becoming a popular tool for the seismic performance evaluation of a structural system, by estimating its strength and deformation demands induced during a seismic event, by means of a static non-linear analysis. The demands are then compared to available capacities at the performance levels of interest. The evaluation is based on assessment of important performance parameters

such as global displacements, interstory drift and inelastic element deformations (either absolute or normalized with respect to a yield values). This type of analysis can be viewed as a methodology for predicting seismic force and deformation demands, which can account for, in an approximate manner, the redistribution of internal forces occurring when the structure behave non-linearly. The main advantages of pushover analysis over the linear methods (Linear Static and Linear Dynamic analysis) are: i) the design is achieved by controlling the deformations in the structure; ii) the consideration of the non-linear behaviour, which avoids the use of behaviour coefficients (reduction factors), that can not be rigorously assessed; iii) the possibility to trace the sequence of yielding and failure on the member and the structure levels; iv) the existence of different design levels to verify the performance targets.

Generally, a non-linear static analysis has three main steps: 1) the definition of the capacity of the structure based on a suitable lateral load pattern and on a adequate mathematical model with the non-linear force deformation relationships for the various components/elements of the structure; 2) the definition of the seismic demand in the form of an elastic response spectrum and 3) the definition of the seismic performance of the building.

In this work the N2 method (Fajfar 2000) is presented and applied to three steel structures, two plane frames and a three dimensional (3D) building. The results obtained are then compared with the ones from non linear dynamic analyses. In this paper the results are presented in terms of horizontal displacements, inter-storey drifts and of rotations at the connections.

2 DESCRIPTION OF THE N2 METHOD

In this section, the steps of the simple version of the N2 method (Fajfar 2000) adopted by Eurocode 8 (EC8 2003) are described.

2.1 Data

A mathematical model of the structure has to be defined with the non-linear force deformation monotonic relationships for the various structural elements of the structure, as illustrated in Fig. 1. The most common element model is the beam model with concentrated plasticity at both ends.

For the three dimensional structures the floor diaphragms have to be assumed rigid in the horizontal plane.

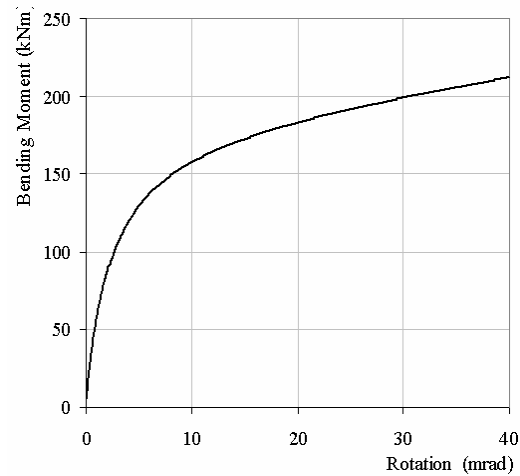


Figure 1. Monotonic curve for a semi rigid connection.

The seismic action is traditionally defined in the form of an elastic response spectrum for a certain damping coefficient and a peak ground acceleration value.

2.2 Seismic Demand in ADRS Format

In this method the spectrum is graphically represented in ADRS format (Acceleration Displacement Response Spectrum), where the acceleration spectral values are defined as a function of the spectral displacement values (Fig. 2).

For an elastic (single-degree-of-freedom) SDOF system with a period of vibration T the relationship between the elastic acceleration S_{ae} and the elastic

displacement S_{de} is defined according to the equation 1.

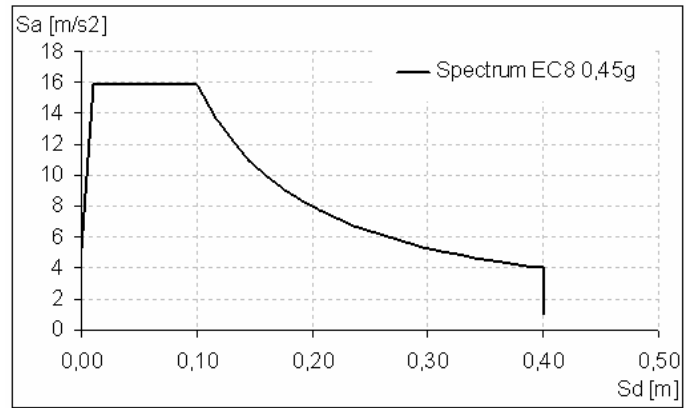


Figure 2. Response Spectrum in ADRS format, $\xi = 2\%$.

$$S_{ae} = \frac{T^2}{4\pi^2} S_{de} \quad (1)$$

The inelastic response spectrum (S_a and S_d), for constant values of ductility μ , is obtained according to equation 2, where q_u – (equation 3) – represents the factor of reduction due to ductility, *i.e.* due to the hysteretic energy dissipation in ductile structures. In equation 3, T_c represents the characteristic period of the ground motion (EC8 2003).

$$S_a = \frac{S_{ae}}{q_u} \quad S_d = \frac{\mu}{q_u} S_{de} \quad (2)$$

$$q_u = \begin{cases} (\mu-1) \frac{T}{T_c} + 1 & T < T_c \\ \mu & T \geq T_c \end{cases} \quad (3)$$

2.3 Definition of the capacity curve

The capacity curve is defined by subjecting a structure to a monotonically increasing pattern of lateral forces, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. It is usually defined in terms of base shear and roof (top) displacement, as can be seen in the Fig. 3.

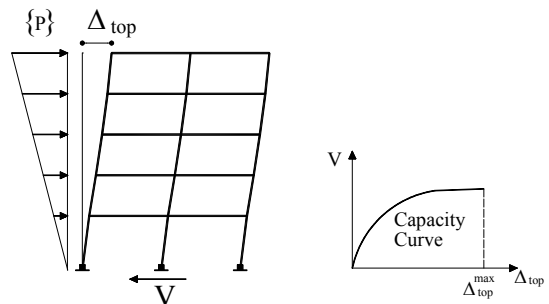


Figure 3. Definition of the capacity curve of the structure.

The selection of an appropriate lateral load distribution is an important step within the pushover

analysis. A unique solution does not exist. One practical possibility is to use two different load patterns, and to envelope the results. According to Eurocode 8 (EC8 2003) a uniform pattern, based on lateral forces that are proportional to mass regardless of elevation, and a modal pattern, consistent with the lateral forces distribution determined in the elastic analysis, are required. The latter vector of the lateral loads can be defined according to equation 4.

$$P_i = p m_i \phi_i \quad (4)$$

The magnitude of the lateral loads is controlled by the factor p . The loads are related to the normalized displacements ϕ_i and to the mass m_i of the floor i .

2.4 Equivalent SDOF Model

In the N2 method, seismic demand is determined by using a response spectrum. Therefore, the structure should, in principle, be defined as an equivalent SDOF system. So, the multi degree of freedom (MDOF) structure has to be modified to an equivalent SDOF by means of a transformation factor (constant) defined in equation 5.

$$\Gamma = \frac{\sum_i m_i \phi_i}{\sum_i m_i \phi_i^2} = \frac{m^*}{\sum_i m_i \phi_i^2} \quad (5)$$

The force and displacement of the equivalent SDOF system F^* and d^* are then:

$$F^* = \frac{V}{\Gamma} \quad d^* = \frac{\Delta_{top}}{\Gamma} \quad (6), (7)$$

As the same constant Γ applies for the transformation of both force and displacements, the force-displacement relationship defined for the structure applies also to the equivalent SDOF system (F^* - d^* diagram) - Fig. 4.

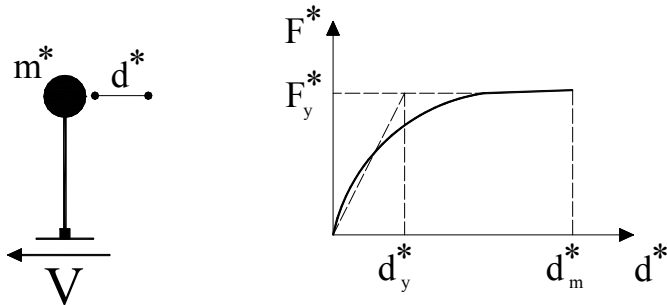


Figure 4. Equivalent single-degree-of-freedom system and capacity curve

The bilinear representation (elastic – perfectly plastic) force – displacement for the SDOF system, has to be defined to identify the value of the elastic period of the equivalent system (T^*). In the N2 method, the idealized bilinear response is defined with post-yield stiffness equal to zero and based on

the equal energy principle. Then, the strength capacity of the equivalent SDOF system is determined (F_y^*), as well as the yield displacement (d_y^*). The elastic period of the idealized bilinear system T^* can be defined according the following equation:

$$T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}} \quad (8)$$

2.5 Seismic demand for the Equivalent SDOF System

The seismic demand for the equivalent SDOF system can be determined using the graphical procedure illustrated in figures 5 and 6, for low periods of structures and for medium and long periods, respectively.

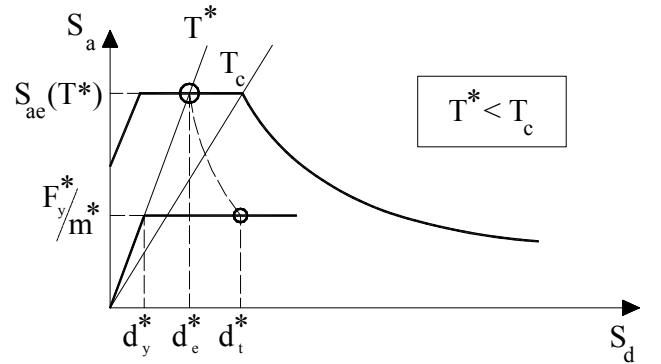


Figure 5. Target displacement for low periods SDOF systems.

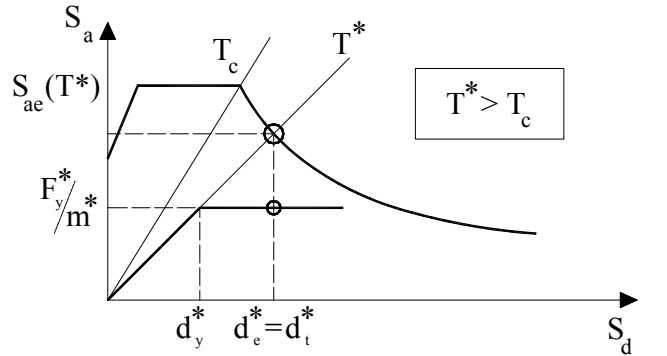


Figure 6. Target displacement for medium or long periods SDOF systems.

Both the demand spectrum and the capacity diagram are plotted in the same graph. The intersection of the radial line corresponding to the elastic period of the idealized bilinear system T^* with the elastic demand spectrum S_{ae} defines the acceleration demand (strength) required for elastic behaviour and the corresponding elastic displacement demand (d_e^*) - equation 9.

$$d_e^* = S_{ae}(T^*) \left[\frac{T^*}{2\pi} \right]^2 \quad (9)$$

The determination of the target displacement d_t^* depends on the dynamic characteristics of the system:

a) For lower periods $T^* < T$ (Fig. 5)

If the structure presents an elastic behaviour ($F_y^*/m^* > S_{ae}(T^*)$)

$$d_t^* = d_e^* \quad (10)$$

If the structure is non-linear ($F_y^*/m^* < S_{ae}(T^*)$)

$$d_t^* = \frac{d_e^*}{q_u} \left(1 + (q_u - 1) \frac{T_c}{T^*} \right) \geq d_e^* \quad (11)$$

where q_u is defined according to equation 12 (see equation 2).

$$q_u = \frac{S_{ae}}{S_a} = \frac{S_{ae}}{F_y^*/m^*} = \frac{S_{ae} m^*}{F_y^*} \quad (12)$$

b) For medium or long periods $T^* > T$ (Fig. 6)

$$d_t^* = d_e^* \quad (13)$$

2.6 Performance evaluation (damage analysis)

The displacement demand for the SDOF system is transformed into the maximum top displacement of the structure by using equation 14.

$$\Delta_{top} = \Gamma d_t^* \quad (14)$$

The local seismic demand of the MDOF model is obtained pushing the structure to Δ_{top} , under monotonically increasing lateral loads with a fixed pattern (step 2.3). It is assumed that the distribution of deformations throughout the height of the structure in the pushover analysis (non-linear static analysis) approximately corresponds to the results obtained in the non-linear dynamic analysis. Eurocode (EC8 2003) recommends to increase Δ_{top} until a minimum equal to 150% of the displacement obtained in equation 14.

Then the seismic performance of the structure can be assessed by comparing the seismic demands, previously determined, with the capacities for the performance level considered. For instance, the demands and capacities can be compared in terms of rotations in the connections and inter-storey drifts.

3 APPLICATION OF THE NON-LINEAR ANALYSIS

The N2 method is applied to three steel structures, two plane frames and a three dimensional structure. The first structure is a low rise office building with four spans and two floors, the second

structure has two spans and five floors, and finally the third is a 3D structure, with four by five spans and eight floors.

For the three structures non-linear static and dynamic analyses were performed. The mathematical model consists of beam elements, for the beams and columns, and of joint elements, to model the non-linear behaviour of the connections.

The numerical studies were performed with the SeismoStruct program (Seismosoft 2004), which has a specific joint element to model the connections. For this element several parameters have to be defined to characterize all non-linear hysteretic behaviour (Nogueiro *et al.* 2005).

For the non-linear static analyses the capacity curve for each structure are determined for two distributions of lateral loads, a modal and a uniform distribution. The results obtained with pushover analyses are compared with the ones of non-linear dynamic analysis.

For the 3D structure the floor diaphragms are assumed to be rigid in the horizontal plane. For this structure the study was performed only for the strong axis direction.

For all structures a 2% damping coefficient was considered.

3.1 Description of the analyzed structures

The first 2-D structure, herein called E-2x4-2D is a low rise office building, with four spans and two floors, as can be seen in the figure 7.

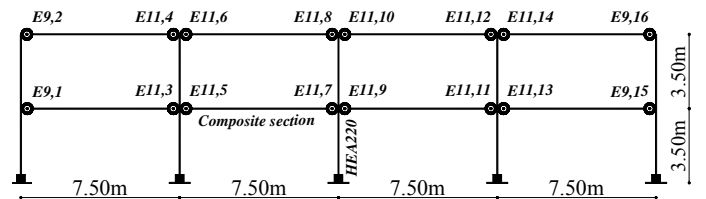


Figure 7. Geometry of the structure E-2x4-2D.

As referred, the structural elements are in elastic range, considering that the energy dissipation occurs in the connections. The connections considered to this structure were tested in the laboratory of Coimbra University (Simões *et al.* 2001) according to the ECCS procedure. The hysteretic behaviour obtained for this cyclic loading is graphically represented in figures 8 and 9, respectively for external E9 and internal E11 connections.

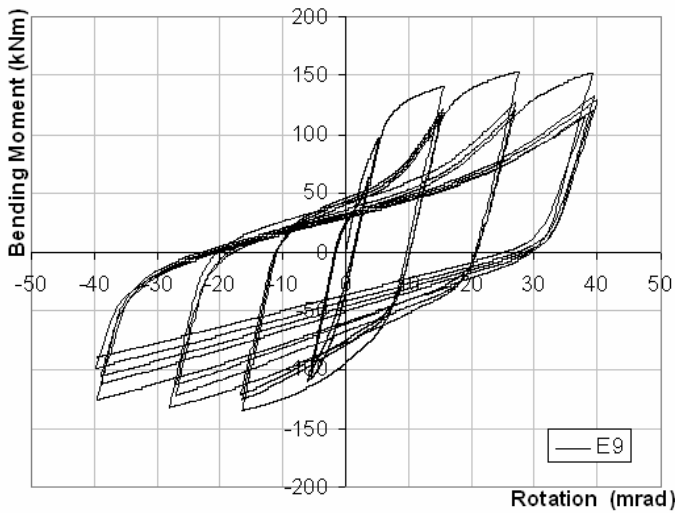


Figure 8. Hysteretic curve for external E9 connection.

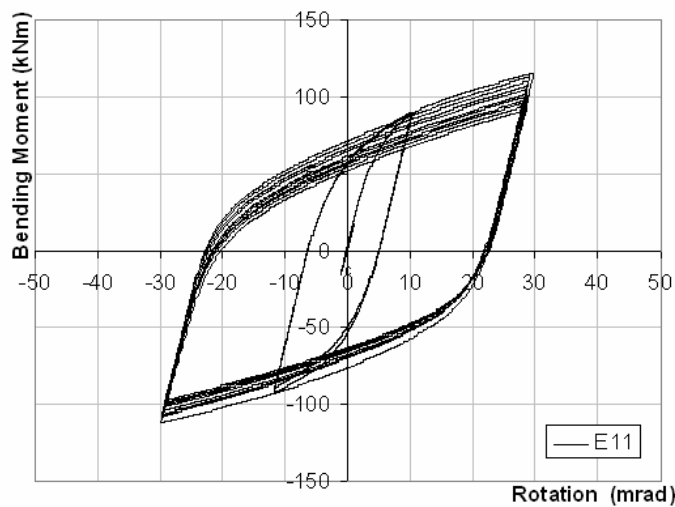


Figure 9. Hysteretic curve for internal E11 connection

The second structure studied, E-5x2-2D, (Della Corte *et al.* 2000), is also a 2D structure, with two spans and five floors, as can be seen in figure 10. The behaviour of the connections was idealised, according to the JB1-3A connection (Bursi *et al.* 2002) and the corresponding parameters are presented in (Nogueiro *et al.* 2005). These connections were double extended end-plate, with their initial strength and stiffness given by the component method (Eurocode 3 2004). Their behaviour was conditioned by the end plate thickness. Figure 11 represents the hysteretic behaviour of J4 connection, when subjected to cyclic loading defined according to the ECCS procedure. For the other connections, the hysteretic behaviour is the same, and their strength and stiffness decrease with the decreasing of the adjacent structural member strength.

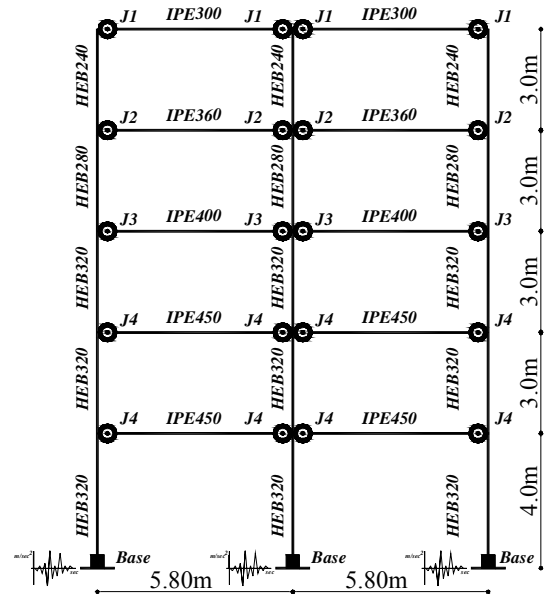


Figure 10. Geometry of the structure E-5x2-2D

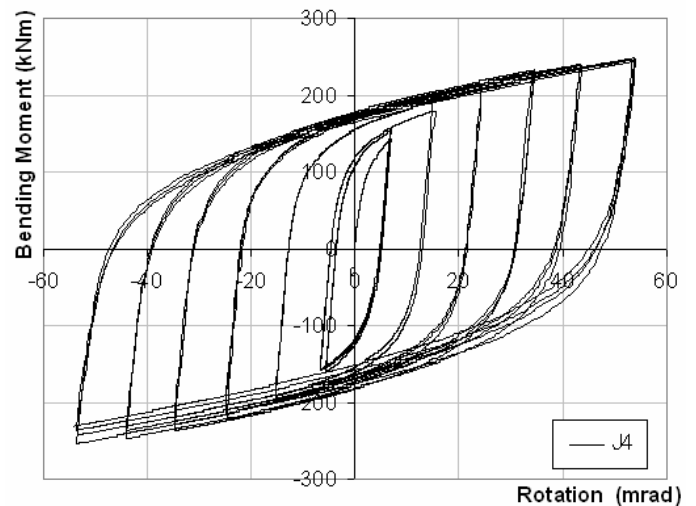


Figure 11. Hysteretic curve for internal E11 connection

The third structure, E-4x5x8-3D, is three dimensional with four by five spans and eight floors, as represented in figures 12 and 13. It corresponds to a real structure existing in Cardington, England, and has been used for several studies in fire research. In reality, the real structure is a no-sway braced frame with a central lift shaft and two end staircases providing the necessary resistance against lateral loads, and their connections designed as flexible end-plates in both directions. For this study, the bracing in the X-direction were removed and replaced by semi-rigid connections with partial strength. A two classes of connections were considered named J-X610 and J-X356, according to the adjacent structural beam, and their cyclic behaviour is showed in figures 14 and 15. The characteristics of these two connections based on the JB1-3A connection (Bursi *et al.* 2002), which has a good seismic performance.

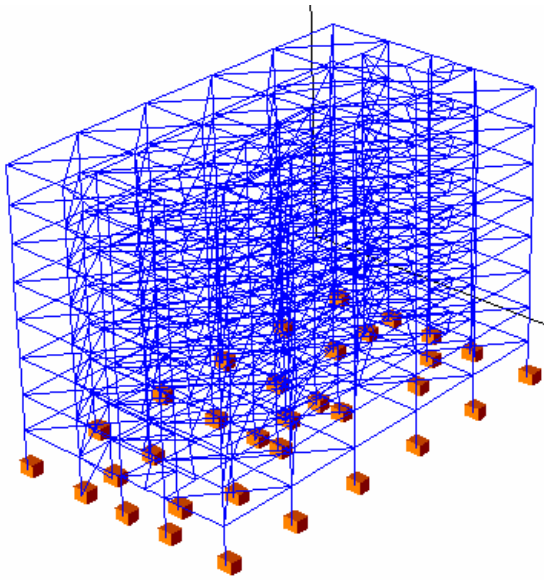


Figure 12. Tri dimensional view of E-4x5x8-3D.

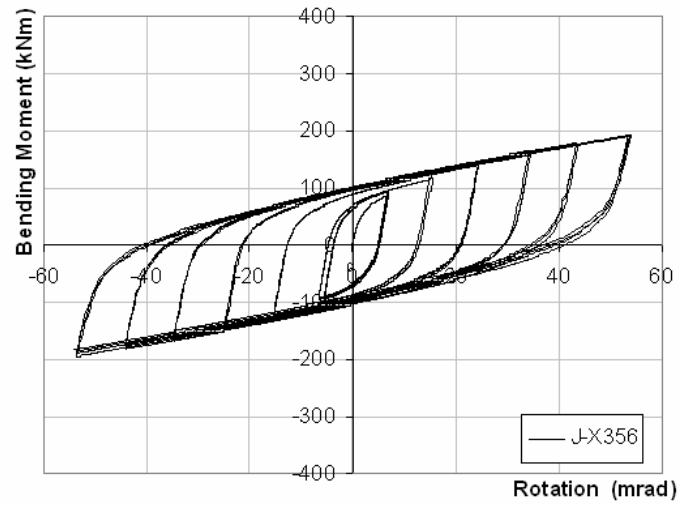


Figure 15. Hysteretic curve for J-X356 connection.

Table 1 summarizes the initial mechanical properties for the various connections used, where M_i is the strength of the connection, K_i the initial stiffness and the K_{pi} the post yield stiffness. With these mechanical properties and after a modal analysis the dynamic characteristics of the structures were defined. The structures have the following fundamental periods of vibration: 1.07, 0.88 and 1.92 seconds, respectively. As expected, the two plane frames represent flexible structures.

Table 1. Initial mechanical properties for the connections.

Structure	Connection	M_i (KNm)	K_i (KNm/rad)	K_{pi} (KNm/rad)
E-2x4-2D	E9	130	24570	600
	E11	85	16830	825
	J1	182	78631	1000
E-5x2-2D	J2	162	63991	1000
	J3	147	48511	1000
	J4	124	32879	1000
	E-4x5x8-3D	X356	100	50000
X610		200	100000	3000

3.2 Definition of the action in the dynamic analysis

The structures were designed according to Eurocode 8 for a soil type B (medium soil) response spectrum, for a given design q factor (reduction factor) and assuming a given ductility class. Linear dynamic analyses of the structures were carried out, and the design action effects were calculated adding the gravity and the seismic effects divided by the assumed q factor.

In the design procedure dead and live loads were considered and the seismic action was represented by the acceleration response spectrum. The gravity and the seismic loads were evaluated and combined according to EN 1990 and EN 1991 adding the values of the permanent action with the design value of the seismic action and the quasi permanent value of variable action (a 0.3 value was used for the corresponding combination coefficient).

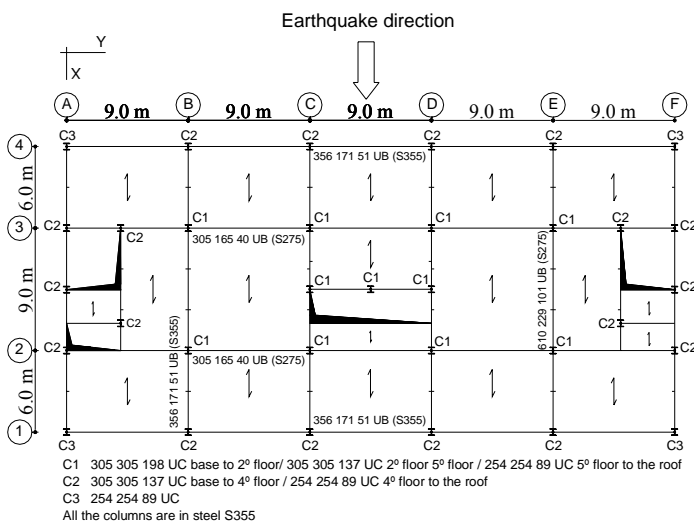


Figure 13. Plan of one inter storey.

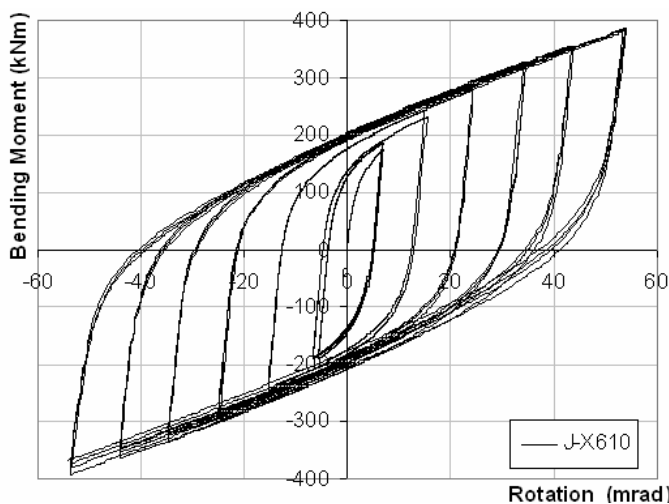


Figure 14. Hysteretic curve for J-X610 connection.

Table 2 presents the permanent and variable loads considered for the three structures studied, according to their occupancies.

Table 2. Definition of the gravity loads.

Structure	E-2x4-2D	E-5x2-2D	E-4x5x8-3D
Permanent (floor)	3 KN/m ²	3 KN/m ²	2.5 KN/m ²
Variable (floor)	2 KN/m ²	2 KN/m ²	2.5 KN/m ²
Permanent (roof)	3 KN/m ²	3 KN/m ²	2.5 KN/m ²
Variable (roof)	2 KN/m ²	2 KN/m ²	4 KN/m ²

For the non-linear dynamic analyses the designed structures have to be subjected to, at least, three accelerograms (#1, #2 and #3), compatible with soil type B response spectrum (Eurocode 8 2003). A large number of accelerograms compatible with the target response spectrum were generated. The three having the best fit to the target response spectrum were chosen. These accelerograms are different from real earthquake records, given that their target response spectrum is a smooth one. Anyway, they are in accordance with the seismic action that was assumed for design purposes and which is the one considered as most likely to occur, mainly in the range of periods between $0.2 T_1$ and T_1 as it is specified in the Eurocode 8 (T_1 represents the fundamental period). Fig. 16 presents the three response spectra corresponding to the three chosen accelerograms together with the target (Eurocode 8) response spectrum.

Fig. 17 illustrates one of the accelerograms chosen.

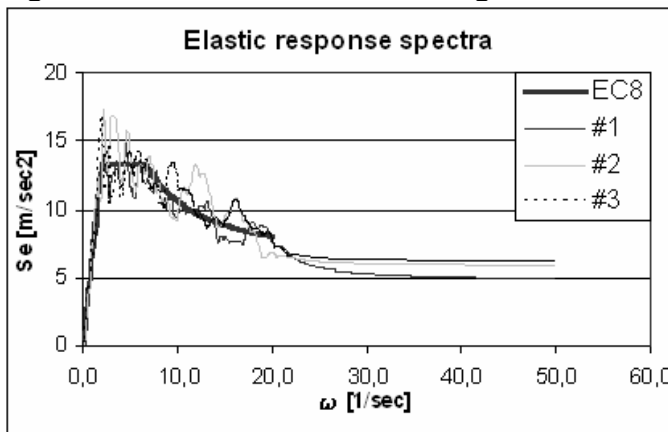


Figure 16. Elastic response spectra, $\xi = 2\%$.

For the non-linear dynamic analyses, three combinations of loads were considered, one for each accelerogram. The 3D structure was subjected only to seismic action acting in X-direction, as can be seen in Fig. 13.

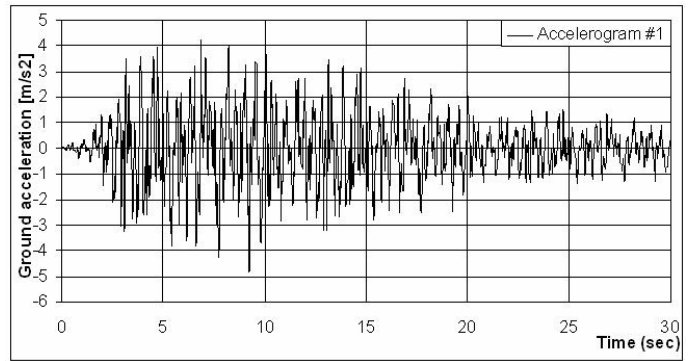


Figure 17. Artificial accelerogram.

3.3 Results

All the three structures described, were firstly analysed through the N2 method (Fajfar 2000). Then, all of the structures were studied by means of non linear dynamic analyses. For both type of analysis the software SeismoStruct (Seismosoft 2004) was used. Finally the results obtained with non-linear static and dynamic analyses are compared, in terms of horizontal displacements, interstorey drifts and rotation at the most stressed connection.

Firstly all the results obtained with the structure E-2x4-2D by applying the N2 method are presented in detail.

The results of pushover analysis are shown in Fig. 18. It represents the capacity curve, base shear vs. control node displacement (top displacement), using two different lateral load patterns (modal and uniform distribution). This curve shows important properties of the structure, such as the initial stiffness, the maximum strength and yield global displacement. From Fig. 18 it can be seen that the uniform load produced larger shear forces for the same displacement values.

Figures 19 and 20 show graphically the determination of the target displacement for uniform and modal loading, respectively.

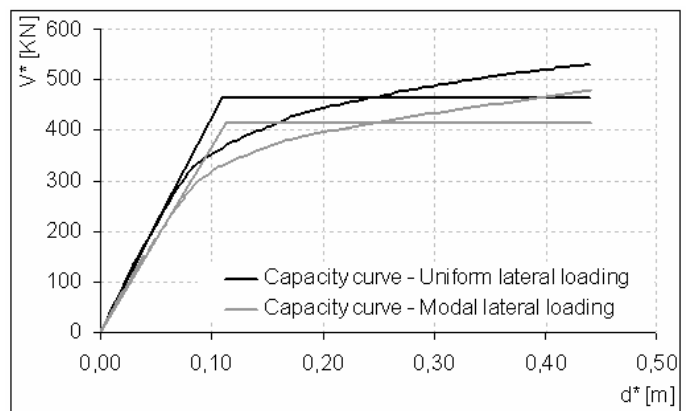


Figure 18. Capacity curves for uniform and triangular modal lateral loading - structure E-2x4-2D.

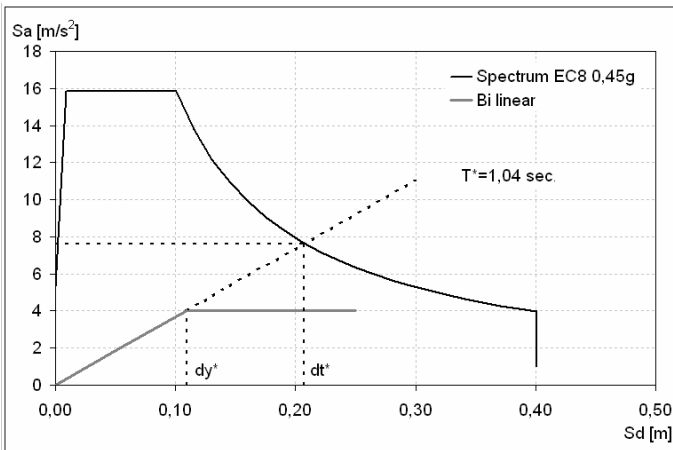


Figure 19. Target displacement for the uniform lateral loading - structure E-2x4-2D.

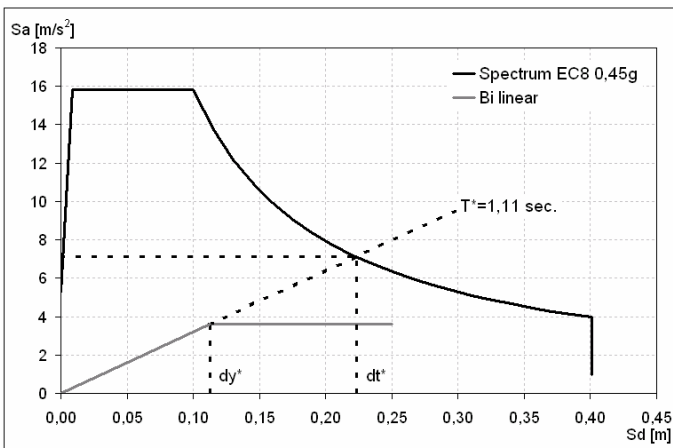


Figure 20. Target displacement for modal lateral loading - structure E-2x4-2D.

Based on the results in the previous three figures, it is observed that the response of the structure is sensitive to the shape of the lateral load distribution. As expected, the target displacement obtained considering the uniform (rectangular) loading is smaller than the value found based on modal loading distribution. Thus, the uniform loading distribution corresponds to a more conservative way of seismic assessment and design.

As described, to determine the target displacement corresponding to a real structure, the target displacement of the equivalent SDOF system has to be multiplied by the Γ factor (the values 1.204, 1.35 and 1.27 were evaluated for the first, second and third structures).

Table 3 shows the horizontal top displacements for all the methodologies performed (non linear dynamic analyses (average values), and N2 method for uniform and modal lateral loading) and for all three structures studied. The last column presents the reference values, representing the ultimate strength (these values were defined considered a 2.5% inter storey drift). Based on the results presented in Table 3, it seems that the methodologies lead to similar maximum top displacements for a 0.45g seismic action. These results also indicate the seismic demands of the first structure are significantly above its ca-

pacities, indicating that their connections should be redesigned or retrofitted. On the other hand, for the other two structures the maximum top displacement and the inter-storey values are smaller than the reference values, which means that their connections exhibit overstrength.

Table 3. Horizontal top displacements.

Structure	δ_{topo} (mm) dynamic	δ_{topo} (mm) N2 (uniform)	δ_{topo} (mm) N2 (modal)	δ_{topo} (mm) 0.025xh
E-2x4-2D	234	249	268	175
E-5x2-2D	195	228	263	400
E-2x4-2D	345	452	508	925

In the following figures the results are presented in terms of horizontal displacements and inter-storey drifts for the three structures. The detailed distribution of storey displacements, and inter-storey drifts are presented for the first structure in Figs. 21-22, respectively. It includes the results obtained for the three dynamic analyses (one for each accelerogram), the corresponding average, and the results from the N2 method, considering the uniform and modal lateral loading distribution. As expected, the maximum inter-storey drifts occur in the second storey. These results show a good agreement between the various methods. Moreover, the differences observed are mainly in magnitude, however maintaining similar profiles in all analysis cases.

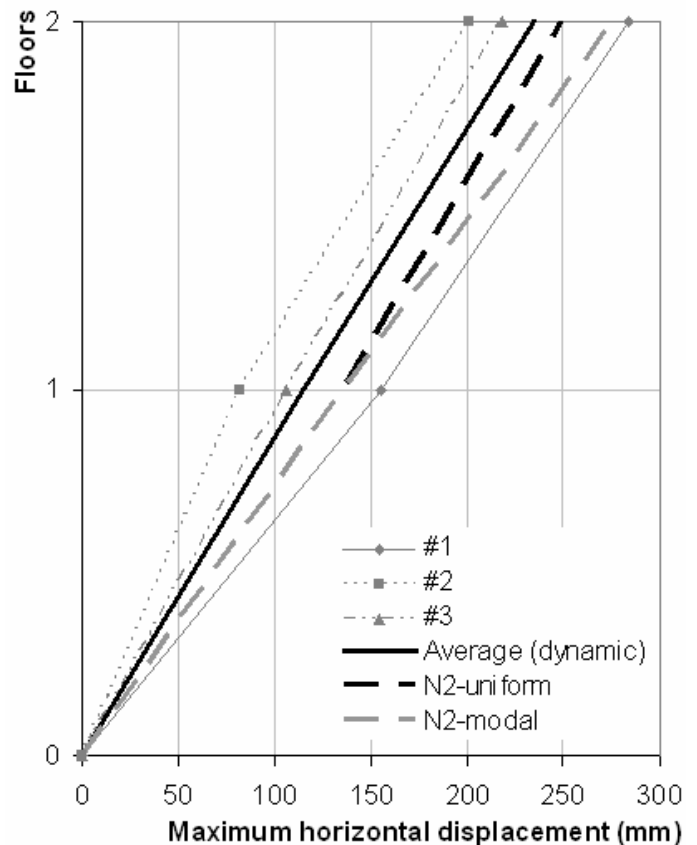


Figure 21. Maximum horizontal displacements for the structure E-2x4-2D.

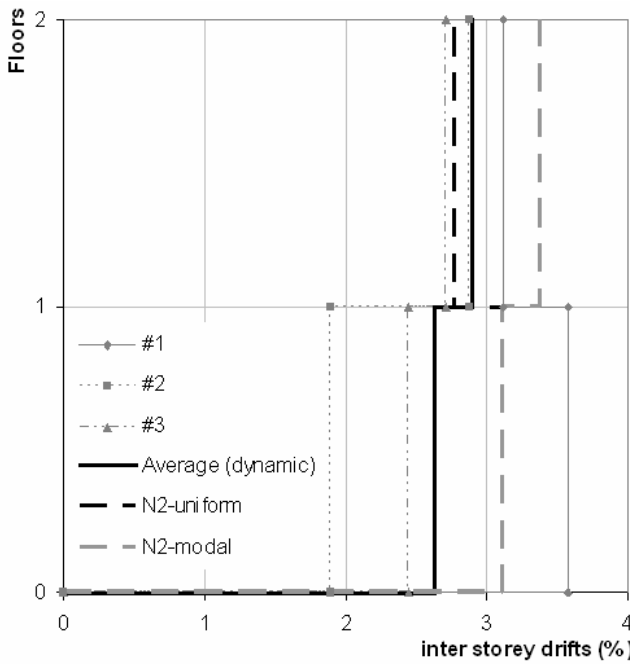


Figure 22. Inter storey drifts for the structure E-2x4-2D.

Figures 23 and 24 present the same results for the second structure. The good agreement between the several methods still is observed. The maximum top displacements are lower than the reference value (given in last column of table 3) and regarding the inter-storey drift normally the maximum value is observed in the third storey. The exception occurs for the N2 method corresponding to uniform lateral loading case, where the maximum inter-storey drift is observed in the second storey.

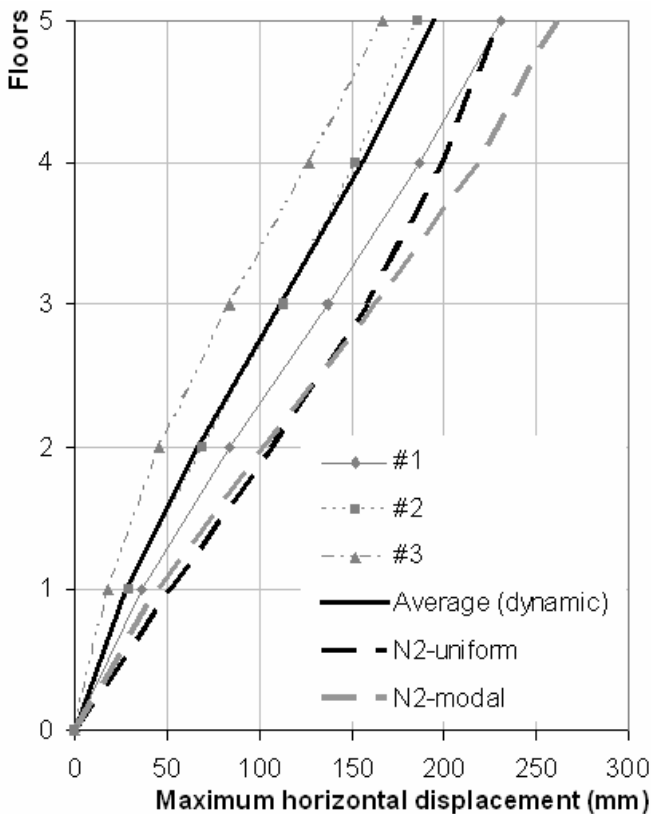


Figure 23. Maximum horizontal displacements for the structure E-2x4-2D.

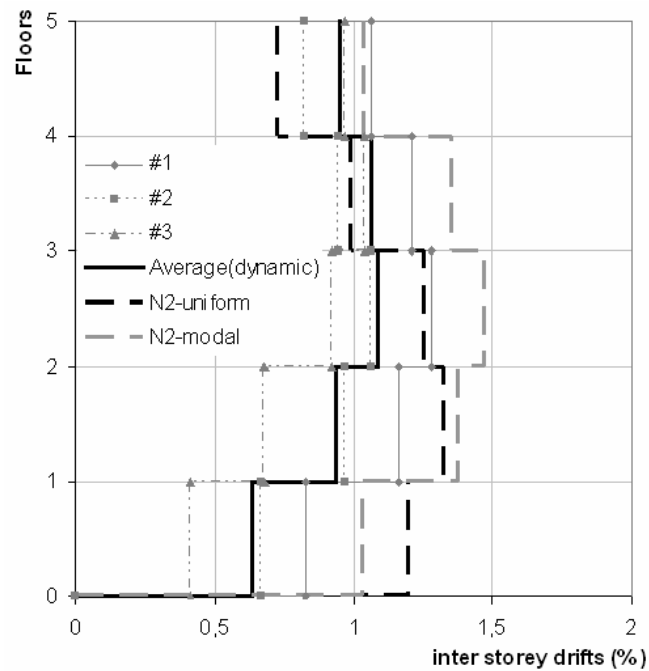


Figure 24. Inter storey drifts for the structure E-2x4-2D.

Finally, figures 25 and 26 present similar results for the 3D structure.

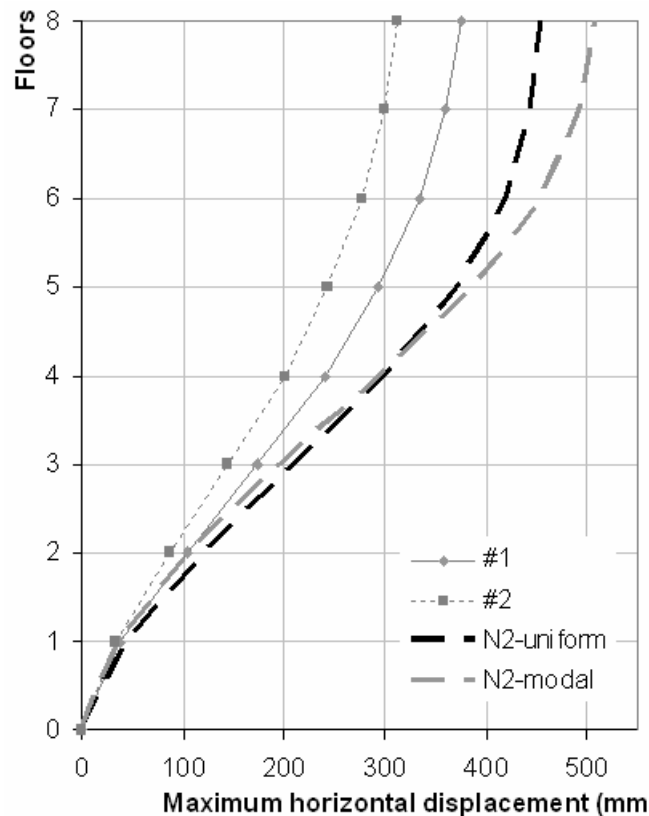


Figure 25. Maximum horizontal displacements for the structure E-2x4-2D.

In the pushover analysis, the loads were applied at the centre of the mass of the different storeys, and only the x-direction was considered. Due to lack of time, only two accelerograms were considered for the non-linear dynamic analysis. Also for this structure it can be observed an acceptable agreement between the results obtained with non-linear dynamic and static analysis. Higher values of inter-storey

drifts occur for the third storey. Only for the N2 method uniform modal loading distribution the maximum inter-storey drift is observed in the fourth storey

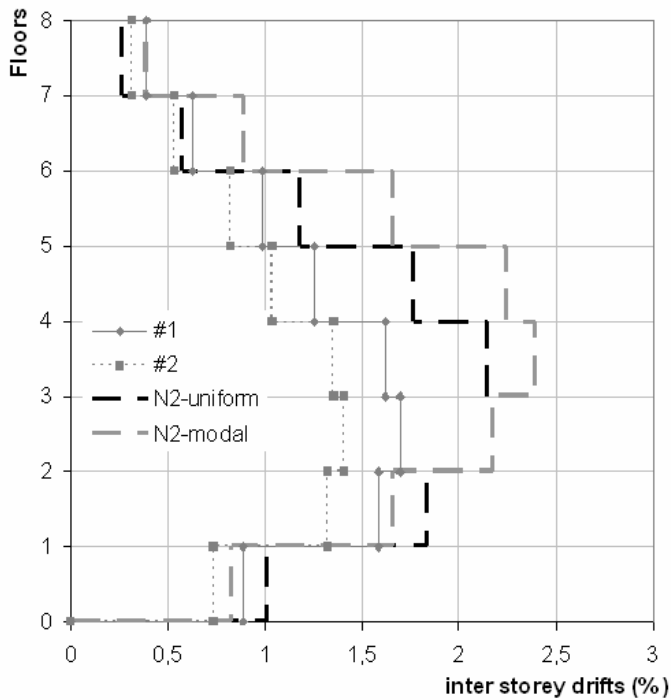


Figure 26. Inter storey drifts for the structure E-2x4-2D.

Other important results for the seismic assessment of the structures are the values of the rotations reached in the connections, mainly in the ones more stressed.

In figures 27 and 28 are presented the hysteretic curve Moment-Rotation for the first structure and for the connections type E9 and E11, respectively. These relationships were obtained with non-linear dynamic analysis using the accelerogram #1. As expected, the values of the maximum rotations are above the ultimate values, respectively 48 and 46 mrad. The connection E9 is quite influenced by the pinching, which approximately accounts for 20% of strength degradation (Nogueiro *et al.* 2005). Both connections are in floor 2.

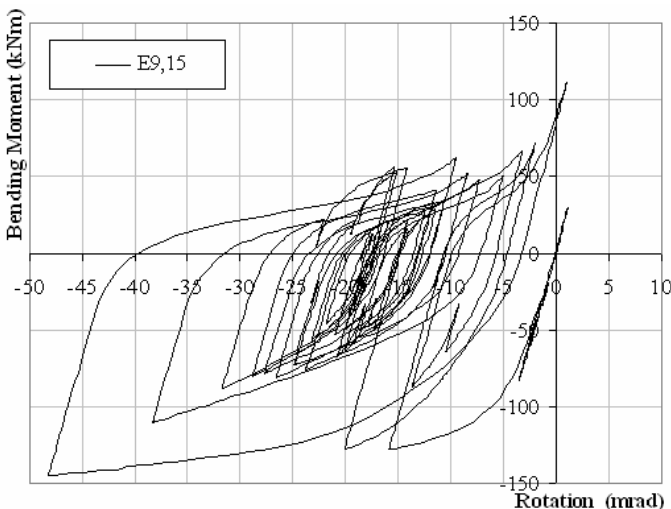


Figure 27. Hysteretic curve for E9 connection in the structure E-2x4-2D.

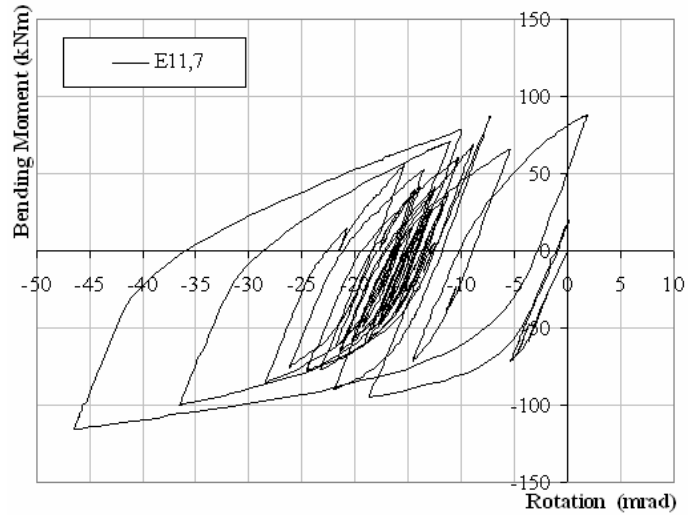


Figure 28. Hysteretic curve for E11 connection in the structure E-2x4-2D.

Figure 29 represents the hysteretic curve for one J4 type connection from the structure E-5x2-2D. The shape of the curve is like expected, with “fat” branches, as it was idealised. The maximum value of rotation reached 15.7 mrad, showing that it is quite far from the strength limit value, as it was observed with the horizontal displacements. This connection is in floor 2 (Fig. 10).

Figure 30 presents the hysteretic curve for one J-X610 type connection. With the design seismic action considered the maximum values reached by the connection (about 10 mrad for a connection in floor 4) are significantly far from the ultimate limit values.

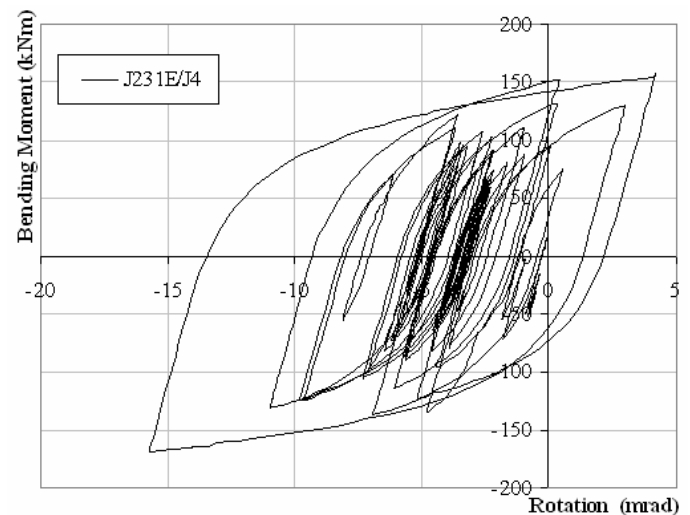


Figure 29. Hysteretic curve for J4 connection in the structure E-5x2-2D.

Table 4 presents the maximum rotation in the connections analysed, for the several methods. For structure E-2x4-2D, where the horizontal top displacements are almost the same, the rotations are larger in the dynamic analysis. The monotonic method does not include the effect of pinching, and damage of strength and stiffness. In the second structure, where the horizontal top displacements, in

the N2 method are larger than in the dynamic analysis, the rotation in the J231E/J4 connection is almost the same, and finally, in the third structure, where the horizontal top displacement in monotonic method is significantly larger than in dynamic analysis, it is showed that in J53EF/J-X610 connection the rotation is smaller for the dynamic analysis, as expected. It can be concluded that, for study of the behaviour of the connections, the monotonic methods have some limitations, because they give insufficient hysteretic information.

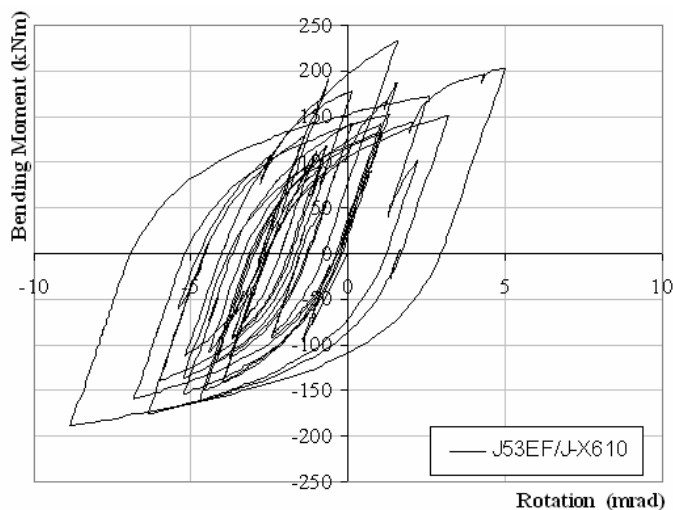


Figure 30. Hysteretic curve for J-X610 connection in the structure E-4x5x8-3D.

Table 4. Maximum rotation in the analysed connections.

Connections	rot. (mrad)	rot. (mrad)	rot. (mrad)
	dynamic	N2 (uniform)	N2 (modal)
E 9,15	48.0	27.6	30.2
E11,7	46.3	30.4	33.3
J231E/J4	15.7	13.9	15.8
J53EF/J-X610	8.8	16.9	20.3

4 CONCLUSIONS

The seismic assessment of three steel structures is performed by means of non-linear static analyses (the N2 method (Fajfar, 2000) was adopted) and the results obtained are compared with the ones obtained with non-linear dynamic analysis. The SeismoStruct program (Seismosoft, 2004) was used for all the numerical studies. Some conclusions could be reached regarding the non-linear static analysis, the structures and the results obtained:

- It is observed a good agreement between the nonlinear static and dynamic analyses, in particular in terms of the horizontal displacements and interstorey drifts results;

- The first structure studied (E-2x4-2D) needs to be redesigned or retrofitted; as the seismic demands values are above the correspondent capacity values, however, it is noted that the reference seismic event exhibit an extreme value of peak ground acceleration

(0.45g), 50% in excess of the usual reference earthquake for Portugal.

- The other two structures, E-5x2-2D and E-4x5x8-3D, exhibit overstrength, considering the horizontal displacements values and the maximum rotation values at the connections;

- The model to simulate the hysteretic connection behaviour for the non-linear dynamic analysis presents good results;

- The N2 method seems to be a conservative design procedure, when compared with the dynamic analysis.

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