Seismic Behaviour and Design of Frame Buildings with Discontinued Columns

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Abstract

Structural irregularities in which a vertical element is interrupted and indirectly supported on beams are associated with poor seismic behaviour and are often avoided by designers. The main objectives of this work are the assessment of the consequences of this irregularity and the identification of the structural layouts in which it proves to be more detrimental to the seismic behaviour of reinforced concrete frame structures.

In the first part a parametric study based on linear 3D frame models was carried out. In that study, the values of the internal forces in the structures corresponding to the gravity loads and the seismic loads were compared with the objective of determining the load combination which led to the most unfavourable condition.

In the ensuing part of the study, both static and dynamic non-linear time-history analyses were performed, comparing the most detrimental layout identified in the parametric study with the corresponding regular model. This study was carried out to evaluate the influence of the irregularity on both the non-linear behaviour and on the dynamic effects.

The possibility of taking into account this irregularity at the design stage through a reduction of the behaviour factor was also considered. It was concluded that a careful design and detailing may be the key to ensure that structures with this irregularity have a suitable behaviour under seismic actions. Failing to properly approach this kind of irregularity, may lead to severe structural damage and eventually to collapse, as seen in past occurrences.

Key words: Discontinued columns; Structural irregularities; RC structures; Seismic behaviour; Non-linear analysis

1. Introduction

In current days there is an ever-growing architectural need for increased spans, e.g., entrance hall of hotels, movie theatres, auditoriums, parking lots etc. In some of these cases the increased span is only required in the lower levels, therefore, the resource to the same span typology in upper floors would lead to an uneconomical waste of space. One of the ways to conciliate these architectural design constraints is to adopt a transfer structure (which may have a more global or local effect in the structure). Entire transfer floors - which may be of four types: girder, truss, plate and box, being the first two the more common - can then be used, whereas, for more local situations a single beam may suffice.

The main objective set for this work is to study the effects of discontinued columns in RC structures. Different layouts are taken into consideration, aiming to identify the one that proves to be more unfavourable. For that layout it is ascertained whether the effect of the irregularity is local or global and indicated suitable ways to account for it at the design stage.

A parametric study was carried out to assess the effect of a discontinued column in different situations, regarding distinct structural layouts as well as the location of the irregularity.

The implications of the discontinued column in the non-linear global and local behaviour were studied in a tridimensional FEM model. The linear and non-linear response of the model with and without the discontinued columns were compared in terms of structural displacements, total base shear, axial force and chord rotation.

2. Literature Review

The most significant results obtained regarding the studies developed by other authors are presented, of which only the most significant will be addressed in detail.

In the work of Poonam *et al*. [15] various types of structural irregularities were studied and their effect on horizontal displacement, storey drift and storey shear were compared. Based on their results the irregularity that leads to the higher horizontal displacement and storey drift is the case where the discontinued columns are considered.

In the research of Singla and Rahman [19] the effect of discontinued columns was studied in three different 3D models. The compared parameters were the fundamental time period, the spectral acceleration, the base shear and the storey displacement. The most significant result was that the irregular models have a fundamental period higher than that of the regular model which indicates that the irregularity leads to a softer structure when no cross-section re-dimensioning is considered.

Harugoppa and Muranal [8] compared the effect of discontinued columns in the internal forces of the beams (M and V), whereas Mundada and Sawdatkar [11] compared the bending moments and shear force in columns of three 3D building models. Both studies concluded that the existence of the irregularity leads to an aggravation of the effect of the seismic loading leading to increased internal forces.

In the work of Nanabala *et al.* [13] a very consistent study of the effect of discontinued columns is presented. The authors considered 3 different 3D models: the first was a regular 6 floor building with plan of five 5m spans in each direction; the second model was similar to the first although with all the peripheral columns discontinued below the first floor; the third model was based on the second model but the cross-section of the beams and columns were different. The study was performed first by an equivalent static analysis method and then the results were compared with the results attained by a linear time-history analysis. The results obtained in that work are that the consideration of the irregularity increases the displacement of the structure but by considering different cross-sections the displacements reduced significantly, being smaller than in the regular model. The irregularity considered was shown to lead to a structure with serious problems of soft storey, contrarily to what happened in the regular model. The results also indicated that the consideration of the mentioned irregularity led to an increase of approximately 40% in materials (both concrete and rebars).

Rohilla *et al.* [16] thoroughly studied various possible positions for the irregularity in different types of structures arriving to conclusions similar to the ones presented before.

In the work of Nautiyal *et al.* [14] the effect of "floating" columns under seismic action was studied and a magnification factor (factor that compares the regular and irregular models) was proposed. Two 2D frame models were considered in that study, one of four floors and other of six floors. For each model different positions for the discontinued column were considered. The more relevant conclusion is that, regarding base shear, the irregular models, by being more flexible, are also more favourable than the regular ones. Regarding the bending moments in columns and beams the results were more variable; in some cases the irregular models were more conditioning whereas in others the contrary is observed. In the case of the beams this was justified by the increase in span although for the columns the reason was not so clear. A possible explanation is that the reduction in base shear does not compensate for the removal of one of the columns.

Considering the occurrences of collapse in which the existence of discontinued columns led to structural failure, it is important to mention the 2001 Bhuj earthquake in India and the bombing of the Alfred P. Murrah Federal Building in Oklahoma City.

In case of the Bhuj earthquake, in order to bypass the floor surface index, designers considered this structural layout (discontinued columns) without properly defining a load path for the horizontal loads. The implications and effects of this are addressed in detail in the works of Murty *et al.* [12], Agarwal *et al.* [1] and the EERI [5].

Corley [3] studied the partial collapse of the Murrah Federal Building and the possibility of applying earthquake detailing to reduce the losses caused by similar actions.

3. Methodologies

3.1. Study 1: Parametric Linear Elastic Analysis

For the parametric study, the linear elastic properties of the materials were considered and the models were developed with the FEM program *SAP2000*. A set of case studies were considered with the intention of determining the situations in which the irregularity leads to the most unfavourable situation.

For this, the following parameters were considered:

- Height of the building;
- Span between vertical elements;
- Presence of shear walls;
- Position of the discontinued columns.

3.1.1. Definition of the Case Study

The floor plan of the case study is defined by a grid of 4 by 3 spans ***L***[m] for the ***x*** and ***y*** directions respectively (Figure 1).

For this plan two situations were considered: Plan A with a span of 7m and Plan B of 4m.

The other plan considered (Plan C) is similar to Plan A, but has shear wall in all four corner, minimizing the torsional effects on the building.

The elevations considered in this study were defined according to the number of floors ***n***, with a regular height of 3.0m. The number of floors considered were 2, 6 and 10.

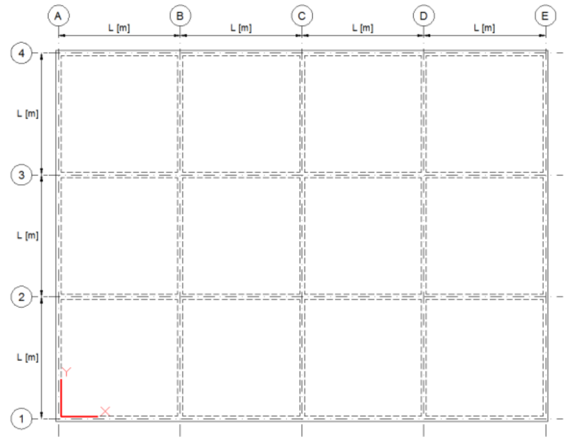

Figure 1 – Floor plan grid and axes

Based on the plans and elevations previously mentioned, the case studies considered are defined in Table 1.

Table 1 – Definition of the adopted cases

Case Reference	Span [L]	Shear Walls	*n* floors
A.I	7 m	No	6
A.II	7 m	No	2
A.III	7 m	No	10
B.I	7 m	Yes	6
C.I	4 m	No	6

3.1.2. Position of the Discontinued Columns

For each of the models defined, different positions for the discontinued column were considered. The direction of the transfer beam (TB) in which the discontinued columns rests varied accordingly. The positions of the discontinued column considered in this study are those presented in Table 2.

Table 2 – Definition of the adopted irregular situation

Situation Reference	Direction of the TB	Alignment of the TB	Discontinued Column
R	No discontinued columns		
X-b1	X	1	B1
X-c1	X	1	C1
X-bd1	X	1	B1+D1
X-b2	X	2	B2
X-c2	X	2	C2
X-bd2	X	2	B2+D2
Y-a2	Y	A	A2
Y-b2	Y	B	B2
Y-c2	Y	C	C2

The models considered in this study combine the identification of the case reference with the situation reference. For example, **A.II_X-c1** refers to the irregular model with plan A and elevation II where the discontinued column is B1 and the transfer beam is oriented in the x direction.

3.1.3. Definition of the Analysis Parameters

The analysis of the results for the parametric study was carried out through the definition of the adimensional parameters $\boldsymbol{\alpha_b}$ and $\boldsymbol{\alpha_c}$, respectively for the beam and column structural elements. These parameters reflect to what extent the Seismic Load Combination (SLC) leads to increased seismic action effects when compared to the Fundamental Load Combination (FLC).

The parameter $\boldsymbol{\alpha_b}$ is defined in equation (1) and $\boldsymbol{\alpha_c}$ is defined in equation (2).

$$\alpha_b = \frac{E_{SLC}}{E_{FLC}} \quad (1)$$

Where E_{SLC} and E_{FLC} are the maximum value per alignment of the bending moment for the SLC and for the FLC, respectively.

$$\alpha_c = \frac{U_{SLC}}{U_{FLC}} = \frac{\left(\sqrt{\nu^2 + \mu_x^2 + \mu_y^2}\right)_{SLC}}{\left(\sqrt{\nu^2 + \mu_x^2 + \mu_y^2}\right)_{FLC}} \tag{2}$$

Where $\boldsymbol{\nu}$ is the normalized axial force, $\boldsymbol{\mu_x}$ and $\boldsymbol{\mu_y}$ are the normalized bending moment in the $\boldsymbol{x}$ and $\boldsymbol{y}$ direction, respectively. Also, $\boldsymbol{U_{SLC}}$ and $\boldsymbol{U_{FLC}}$ refer to the maximum value of $\sqrt{\nu^2 + \mu_x^2 + \mu_y^2}$ for a given alignment for the SLC and for the FLC, respectively.

3.1.4. Definition of the Structural Elements

The definition of the cross-sections of the beams and columns, as well as the thickness of the floor slabs were performed adopting pre-design criteria that normally ensure a good behaviour for both the Ultimate and Serviceability Limit States.

The materials considered were a concrete, steel and pre-stress steel conforming to material classes C30/37, A500NR and A1860/1670, respectively.

A thickness of 0.20m was adopted for all slabs for all the cases with the exception of **C.I** for which a slab thickness of 0.15m was adopted.

The cross section of the transfer beams is summarized in Table 3 (DC meaning “discontinued column”).

Table 3 – Transfer beams assigned for each model

Element Reference	Models assigned with the transfer beam
TB1_1.6x0.6	A.I and B.I with DC in the internal alignment
TB2_1.4x0.5	A.I and B.I with DC in the peripheral alignment
TB3_1.3x0.5	A.II with DC in the internal alignment
TB4_1.1x0.4	A.II with DC in the peripheral alignment
TB5_1.8x0.6	A.III with DC in the internal alignment
TB6_1.6x0.5	A.III with DC in the peripheral alignment
TB7_1.0x0.4	C.I with DC in the internal alignment
TB8_0.8x0.4	C.I with DC in the peripheral alignment

The regular beams have a cross-section of 0.6X0.3m (height x width) for all cases but the C.I, which had 0.4X0.30m beams.

Regarding the columns, their cross-section varies along the height of the buildings, having been defined based on the pre-design criteria $\nu_{ELU} \leq 0.6$ which normally ensures a ductile behaviour under seismic loads (with values of ν between 0.3 and 0.4). All columns followed the mentioned criteria as well as the following rules:

- Preferentially all columns should have a squared cross-section with sides of $[h \times b]$, with $h = b$. Although exceptions can be made to verify the following criterion.
- The dimension of the beam width should be equal to or smaller than the perpendicular side of the column supporting it.

Based on these principles the cross-section of the columns varied from 0.3m to 1.2m with increments of 0.1m, for the squared cross-sections.

For this parametric analysis given the large number of considered cases and that the determined values will be compared to each other, the effect of the pre-stress in the transfer beam was not taken into consideration.

3.1.5. Definition of the Loading Patterns

The loading patterns considered were the dead loads (structural self-weight and other permanent loads with a value of $3\ kN/m^2$ in all floors), live loads ($2\ kN/m^2$ in all floors) and the seismic loads defined according with the EN1998-1 [6] for a building with a importance class II ($\gamma_I = 1.0$), located in Lisbon (zones 1.3 and 2.3, respectively for the distant and close earthquake scenarios) and a soil type B. For the horizontal components the behaviour factor (q) considered were of $q = 3.0$ for the regular models and of $q = 2.4$ for the irregular models (20% reduction to account for the irregularity effects). The nationally determined parameters were those of EN1998-1 [6].

Regarding the vertical component of the seismic action, considered in this study, the response spectrum was defined in accordance with the Portuguese national annex with $q = 1.5$.

3.2. Study 2: Non-Linear Inelastic Analysis

The non-linear analyses were conducted using the FEM program SeismoStruct [18] that considers non-linear inelastic properties of the materials as well as the geometrical non-linearities.

The case study considered for the development of the non-linear analysis was the **A.II**, which, in study 1 was shown to be the case in where the irregularity led to a more significant aggravation. Furthermore the situation considered was the **X-b1**.

The cross-sections considered for this model were those presented in Table 4.

Table 4 – Dimensions of the cross-sections adopted for the non-linear analysis

Element	Position	h [m]	b [m]
RB_0.6x0.3	Regular beams	0.60	0.30
TB_1.1x0.4	Transfer beam	1.10	0.40
C_0.4x0.4	Peripheral columns	0.40	0.40
C_0.5x0.5	Internal columns	0.50	0.50

Figure 2 presents the irregular and the regular models developed for **Study 2**.

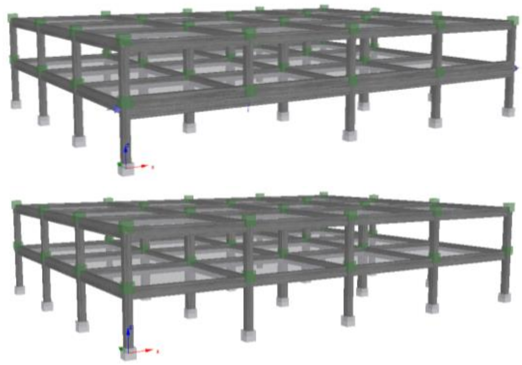

Figure 2 – Irregular model (above) and regular model (below) considered in the non-linear study

3.2.1. Design of the Pre-stress and Steel Reinforcement

The pre-stress tendon of the irregular model was defined considering the pre-design criterion that the vertical displacement in the zone of the discontinued column should be approximately 0 for the Quasi-Permanent Load Combination (QPLC). The solution adopted was *1 cable 6/19 Normal* (with a pushing force at infinite time of 2850 kN) and displacements for the QPLC and for the effect of the pre-stress were, respectively, -0.0291m and +0.0291m. Figure 3 presents the loads equivalent to the effect of the pre-stress for a pushing force of 1000 kN.

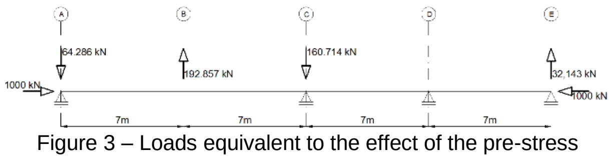

Figure 3 – Loads equivalent to the effect of the pre-stress

Considering current design rules that ensure good ductility and adequate behaviour for serviceability and ultimate limit states, as well as the rules of the capacity design, the following steel reinforcements were computed for both the regular and irregular models:

- For the regular beams (RB_0.6x0.3), longitudinal reinforcement $4\phi20$ and $4\phi25$ on the top and bottom face, respectively, and stirrups $\phi10//0.15m$ with 2 legs;
- For the transfer beam (TB_1.1x0.4), longitudinal reinforcement $4\phi25$ on both the top and bottom face and stirrups $\phi10//0.15m$ with 2 legs;
- For the peripheral columns (C_0.4x0.4), longitudinal reinforcement $3\phi25$ in each face and $\phi10//0.15m$ with 4 legs (perimeter of 2.0m);
- For the internal columns (C_0.5x0.5), longitudinal reinforcement $4\phi25$ in each face and $\phi10//0.15m$ with 4 legs (perimeter of 2.3m);

3.2.2. Definition of the Constitutive Relationships

For modelling the concrete the chosen constitutive relationship was that proposed by Mander *et al.* [9] (non-linear concrete model). The values adopted for the different parameters, entered in the program, are presented in Table 5.

Table 5 – Properties of the concrete

Mean compressive strength (f_{cm}) [MPa]	38.0
Mean tensile strength (f_{ctm}) [MPa]	3.8
Modulus of elasticity (E_c) [GPa]	28.97
Strain at peak stress [m/m]	0.002
Specific weight (γ_c) [kN/m^3]	24

The chosen constitutive relationship for modelling the steel reinforcement was that proposed by Menegotto and Pinto [10]. Based on the definition of this model, the parameters entered in the program that fully defines it are present in Table 6.

Table 6 – Properties of the steel rebar

Modulus of elasticity (E_s) [GPa]	200
Yield strength (f_y) [MPa]	500
Strain hardening parameter (b) [-]	0.005
Transition curve initial shape parameter (R_0) [-]	20
Transition curve shape calibrating coefficient (a_1) [-]	18.5
Transition curve shape calibrating coefficient (a_2) [-]	0.15
Isotropic hardening calibrating coefficient (a_3) [-]	0
Isotropic hardening calibrating coefficient (a_4) [-]	1
Fracture/buckling strain [-]	0.100
Specific weight (γ_s) [kN/m^3]	77.5

3.2.3. Definition of the Element Class

Regarding the elements, in all cases the force-based the integration method was adopted. The advantages (and disadvantages) of force-based elements over the displacement-based are comprehensibly explained in the study by Correia *et al.* [4]

For the beams, the adopted element class was the **inelastic force-based frame element**. The element was divided into 5 integration sections and each section was divided into 150 and 200 fibres for the regular beams and transfer beam, respectively.

For the columns, the chosen element class was the **inelastic plastic-hinge force-based frame element** with 150 fibres per section and a plastic-hinge length equal to the height of the section (around 15% of the element length in each end).

3.2.4. Definition of the Constraints

Regarding the rigid diaphragm this was modelled using the methodology of the penalty functions integration method (as defined in the user guide of the software). For the definition of the rigid diaphragm weight, the value adopted was 10E+7. Choosing an exceedingly large value for the penalty function was found to be inadequate, since above the given value the beams are not allowed to elongate when subjected to cyclic loadings. The proposed value was derived as a compromise between the rigid diaphragm conditions and those that allowed for realistic nonlinear behaviour of the beam elements.

3.2.5. Definition of the Damping

For the definition of the Rayleigh damping, two points must be entered in the program. These points are pairs [period; damping ratio] used to compute the proportionality constants (to the mass and stiffness matrixes) for defining the damping matrix, according to Clough and Penzien [2]. The adopted points are presented in Table 7, and regard the first vibration modes and the sixth vibration mode (last horizontal vibration mode).

Table 7 – Values for defining the Rayleigh damping

		Period [s]	Damping Ratio [%]
Regular Model	Mode 1	0.322	2
	Mode 2	0.089	2
Irregular Model	Mode 1	0.340	2
	Mode 2	0.100	2

The damping ratio has a value lower than the usual 5% normally adopted for RC structures because the majority of the overall structural damping is due to the nonlinearity of the materials which is already taken in consideration in this type of analysis.

Is also important to refer that the damping is proportional to the tangent stiffness.

3.2.6. Definition of the Loading

Apart from the gravity loading considered in **Study 1**, the seismic loading was considered by a time-history record computed using the software SeismoArtif [17] that generates a set of accelerograms compatible with the PGA of the introduced response spectrum. The seismic action type 1 was considered predominant over the seismic action type 2.

Also, a trapezoidal envelope was considered with a total duration of 40s and an intermediate stationary part of 30s.

Three accelerograms were generated for both the horizontal and vertical direction and 3 cases of combinations of these accelerograms were taken in consideration in this study. The values of the PGA(g) of the obtained accelerograms were 0.249, 0.274 and 0.248 for the horizontal ones and 0.176, 0.216 and 0.156 for the vertical accelerograms.

It is also important to mention that according to the EN1998-1 [6] and considering that only three accelerograms were generated, the maximum of the responses should be considered. Although this was concluded to be excessively unfavourable and therefore the average of the responses was considered.

As a final note regarding the **Study 2**, apart from the dynamic time-history analysis also a static time-history analysis was carried out. Although the results obtained were not as conclusive and therefore not included in this work.

4. Presentation and Discussion of Results

4.1. Study 1: Parametric Linear Elastic Analysis

4.1.1. Analysis of the Transfer Beam for the Seismic Loading

Analysing the bending moments (M) in the transfer beam due to the seismic action (SA) the following was observed:

- The absolute value of the bending moment increases in the x direction from the regular to the irregular situations, as was expected;
- Due to the significant increase in the oscillating mass and in the span length between contiguous columns, the vertical component of the SA becomes more relevant in the irregular cases;
- Despite not being the most common situation, the SA component in the direction perpendicular to the beam axis gains relevance, along with the other components. In the case of the transfer beam this can be justified based on the resisting systems. For any acting seismic action there are two resisting mechanisms. There is a main load transfer mechanism brought by the mobilization of the frame effect and there is a load transfer mechanism based on the tension of some columns and compression of others. In the case of a discontinued column, the tensile and compressive forces lead to bending in the beam which supports the column. This is the reason for the seismic action in the perpendicular to the transfer beam to originate bending moments with the same level of relevance as those originated by the SA acting on the same direction as the TB.

Table 8 presents the maximum bending moment in the transfer beam for the situation of the case **A.I** (**R**, **X-b1** and **X-c1**) acted by the seismic loads in the three directions. This proves the observations presented previously.

Table 8 – Bending moment [kNm] on the transfer beams do to the seismic action

Model A.I	R	X-b1	X-c1
SA in the x direction	452.7	1134.7	1057.9
SA in the y direction	59.8	1091.7	1027.7
SA in the z direction	0.0	513.3	477.9

4.1.2. Results of the Parametric Analysis

Due to the number of situations considered it is not manageable to present the full extent of the obtained results. Therefore, these results are presented in the following through a set of tables containing a synthesis of the results obtained.

The values presented regard to the variation, relative to the regular model (RV), of the maxima, per alignment, of the parameters $\boldsymbol{\alpha_b}$ and $\boldsymbol{\alpha_c}$ for each of the cases considered. This way, it is possible to assess the conditioning situation that leads to the highest aggravation. This relative variation is given by:

$$RV = \frac{Irr - Reg}{Reg} = \frac{\max[\alpha_{IRR}]}{\max[\alpha_{REG}]} - 1$$

Where α_{IRR} and α_{REG} are, respectively, the parameters $\boldsymbol{\alpha_b}$ and $\boldsymbol{\alpha_c}$ for the irregular situations and for the regular situation.

Table 9 and Table 10 present the RV of the parameters $\boldsymbol{\alpha_b}$ for the regular beams and $\boldsymbol{\alpha_c}$. To provide for a better interpretation of the results the values in bold relate to the maximum value per alignment of the five case studies and the highlighted values to the maximum value of each parameter.

Table 9 – RV of the parameters α_b for the regular beams

		A.I	A.II	A.III	B.I	C.I
Alignment	X1	0.28	**0.69**	0.21	0.35	0.40
	X2	0.06	0.13	0.03	0.09	**0.20**
	X3	0.17	0.30	0.15	0.09	**0.41**
	X4	0.24	**0.45**	0.23	0.46	0.44
	YA	0.15	**0.70**	0.16	0.32	0.41
	YB	0.14	**0.33**	0.13	0.11	0.32
	YC	0.20	**0.49**	0.10	0.12	0.29
	YD	0.17	**0.33**	0.16	0.11	0.32
	YE	0.25	**0.61**	0.26	0.32	0.41

Table 10 – RV of the parameters α_c

		A.I	A.II	A.III	B.I	C.I
Alignment	X1	0.40	**0.97**	0.17	0.37	0.39
	X2	0.26	**0.83**	0.12	0.16	0.26
	X3	0.13	**0.14**	0.07	0.08	0.09
	X4	0.41	**1.19**	0.17	0.23	0.39
	YA	0.41	**1.19**	0.17	0.37	0.39
	YB	0.33	**0.98**	0.14	0.22	0.31
	YC	0.38	**1.03**	0.17	0.17	0.34
	YD	0.13	**0.47**	0.07	0.37	0.17
	YE	0.40	**0.97**	0.17	0.37	0.39

Analysing the overall results is concluded that in all cases there is a situation in which the presence of this irregularity leads to the aggravation of the internal forces in the regular beams and columns.

It is also possible to conclude that, regarding the regular beams, the case study in which the variation between the regular and irregular models is higher is mainly the **A.II**. This is also true for the columns, as could be expected given that this case is the one with higher frequency of vibration, being located in the zone of the spectrum with higher accelerations.

The major conclusions for the regular beams and for the columns, taken from this parametrical analysis, are the following:

- The regular beams in the direction perpendicular to the TB are more affected by the irregularity than the ones parallel to the TB.
- In structures with lower number of floors and therefore stiffer, the effect of discontinuing a column is more severe than in structures with more floors (more flexible).
- In situation with lower spans the seismic loads gain relevance when compared with the gravity loads.
- The presence of shear walls reduces the negative effect of the discontinued column, by controlling the overall behaviour of the structure under horizontal loading.

Table 11 summarizes the values of the relative variation of the parameters $\boldsymbol{\alpha_b}$ for the transfer beams. In addition to the values in bold (maximum per alignment) there are values in red that indicate the minimum values per alignment.

Table 11 – RV of the parameters α_b for the transfer beams

		A.I	A.II	A.III	B.I	C.I
Alignment	X1	-0.59	**-0.31**	-0.56	-0.34	-0.70
	X2	-0.47	-0.31	-0.49	**-0.16**	-0.64
	YA	-0.60	**-0.33**	-0.57	-0.50	-0.71
	YB	-0.48	-0.34	-0.51	**-0.28**	-0.66
	YC	-0.49	-0.34	-0.48	**-0.28**	-0.65

The case that presents higher values of α_b for the peripheral alignments is the **A.II** and for the interior ones is the **B.I**, as expected given that these models are stiffer and therefore the seismic action is more conditioning. The case **B.I** might not be the conditioning case to the peripheral columns due to the fact that those alignments contain the shear walls which concentrate most of the horizontal loads transmitted to that alignment, therefore taking lesser advantage of the frame effect.

Other important conclusion is that **C.I** is the case in which the reduction is higher. The reason for this is connected mostly with the seismic action, given that in this model the irregular span (longer span of the TB) is shorter than in the other models, leading to a reduction in the oscillating mass. Thus, the overall internal forces due to the seismic loading are lower in this case, leading also to a lower value of the α_b parameters.

The major conclusions for the transfer beams, taken from this parametrical analysis, are the following:

- For the transfer beams the Fundamental Load Combination is always more conditioning than the Seismic Load Combination. This was concluded by the analysis of the absolute values of $\boldsymbol{\alpha_b}$, which were not presented.
- For stiffer structures the Seismic Load Combination gains more relevance. For the case with shear walls (**B.I**) there is a situation in which the SLC is almost equal to the FLC in terms of bending moments, but smaller nonetheless, that is, the absolute value of $\boldsymbol{\alpha_b}$ is nearly unitary.
- As a rough approximation it is possible to conclude for the TB:

$$\frac{SLC}{FLC} \approx 0.80 \; to \; 0.90$$

These values are meant to be regarded as an indication and not as an absolute rule because there are many factors that influence these.

4.1.3. Displacements

By comparing the displacements at each floor in the horizontal directions the following conclusions were drawn:

- For all the cases and for both directions there is always an irregular situation that is more conditioning than the regular model. The larger differences is registered in the y direction and for irregular cases in which the transfer beam is oriented in the y direction.
- The largest relative variation between regular and irregular models is of approximately of 14% and occurs for the case **A.II**. But the highest absolute difference is of 0.018m and occurs in model **A.III**. This suggests that, as seen for the internal forces, the model with fewer floors is more affected by this type of irregularity.
- In model **B.I** the presence of the irregularity does not affect the structure in what regards the horizontal displacements. In that model the lateral displacements are controlled by the shear walls and hence present a cantilever-type displacement pattern along height. This could lead to the conclusion that in structures with predominant wall behaviour the effect of discontinued columns is not as severe in terms of displacements as in other cases. Although, the considered position of the shear walls is a very specific and favourable case.
- There are situations in which the presence of studied irregularity tends to decrease the displacements, this is not due to the irregularity itself, but instead due to the modifications introduced in the structure that are result of the irregularity, for example, larger columns and beams.

4.2. Study 2: Non-Linear Inelastic Analysis

In order to assess if the presence of the discontinued column leads to an accumulation of inelasticity (and therefore damage) immediately above the irregularity, the chord rotation, structural displacement and base shear were computed and compared. The structural displacements and base shear are presented in order to compare the global behaviour of the structure. The chord rotations are determined to allow for a better understanding of the local behaviour of some parts of the structure.

4.2.1. Displacements

The results obtained for the displacements are presented in Table 12. The linear displacements (d_L) were obtained from a RSA (Response Spectrum Analysis) and the non-linear displacements (d_{NL}) presented are the average of the maxima of the three considered cases. Also, q_d is the displacement behaviour factor defined as the ratio between d_{NL} and d_L.

Table 12 – Displacements for the models of Study 2

Direction		x direction		y direction	
Floor		1st	2nd	1st	2nd
Regular	d_L [m]	0.017	0.032	0.017	0.033
	d_{NL} [m]	0.031	0.052	0.029	0.049
	q_d [-]	1.80	1.61	1.69	1.47
Irregular	d_L [m]	0.017	0.032	0.018	0.034
	d_{NL} [m]	0.031	0.051	0.030	0.049
	q_d [-]	1.84	1.60	1.65	1.43

The overall values obtained for q_d are higher than anticipated. The common assumption that the displacements behaviour factor is unitary (implying that the linear displacements are of the same magnitude as the non-linear) is not verified in this case.

According to the EN1998-1 [6], structures with fundamental periods lower than T_c have higher values of the displacements behaviour factor. This might be the reason for the obtained results. Other factors that might influence the obtained results, although with lesser relevance, are as follows:

- The reduction in the Young's Modulus of the concrete that was assumed (roughly) of 50%, as indicated in the EN1998-1 [6]. In most of the cases this is far from the real reduction in the flexural and shear stiffness due to cracking.
- The computing of the flexural and shear stiffness in the software which considers the contribution of the steel rebars to the cross-section inertia.

Apart from this the results do not show a significant aggravation from the regular to the irregular model.

4.2.2. Total Base Shear

Similarly to the displacements, Table 13 presents the results for the total base shear for the linear analysis (F_B^{lin}) and the average of the base shear of the three non-linear cases (F_B^{Nlin}). This table also presents the behaviour factor (q_F) which is the ratio between F_B^{lin} and F_B^{Nlin}.

Table 13 – Total base shear for the models of Study 2

Direction		X	Y	Z
Regular	F_B^{lin} [kN]	6448.7	6448.7	16249.9
	F_B^{Nlin} [kN]	5148.8	4959.4	14048.5
	q_F [-]	1.25	1.30	1.16
Irregular	F_B^{lin} [kN]	6950.5	6884.1	17561.6
	F_B^{Nlin} [kN]	5024.4	4880.1	17430.4
	q_F [-]	1.38	1.41	1.01

The values obtained are far from what could initially be expected for both the regular and irregular models. Furthermore, the values are not coherent with the assumption that the presence of the irregularity would mean a reduction in the behaviour factor. Although unexpected, these results might be explained by the following reasons:

Assuming that q_{EC8} is the behaviour factor defined according with the EN1998-1 [6], the design value of the total base shear ($F_{B,ED}$) is given by equation (3).

$$F_{B,ED} = \frac{F_B^{lin}}{q_{EC8}} \tag{3}$$

- Considering F_B^{Nlin} is equal to the resisting base shear ($F_{B,RD}$), q_F is given by the equation (4).

$$q_F = \frac{F_B^{lin}}{F_B^{Nlin}} \cong \frac{F_B^{lin}}{F_{B,RD}} \tag{4}$$

- Based on the equations (3) and (4) is possible to establish the relations between q_F and q_{EC8}, as shown in equation (5).

$$\begin{cases} if\ F_{B,RD} < F_{B,ED} \leftrightarrow \dfrac{F_B^{lin}}{q_F} < \dfrac{F_B^{lin}}{q_{EC8}} \leftrightarrow q_F > q_{EC8} \\ if\ F_{B,RD} = F_{B,ED} \leftrightarrow \dfrac{F_B^{lin}}{q_F} = \dfrac{F_B^{lin}}{q_{EC8}} \leftrightarrow q_F = q_{EC8} \\ if\ F_{B,RD} > F_{B,ED} \leftrightarrow \dfrac{F_B^{lin}}{q_F} > \dfrac{F_B^{lin}}{q_{EC8}} \leftrightarrow q_F < q_{EC8} \end{cases} \quad (5)$$

- In the design of the steel reinforcement it was considered that all elements with equal cross-section would have the same reinforcement and that only the conditioning section was considered for its determination. Also, by adopting a certain reinforcement defined by standard rebar diameter the adopted reinforcement is larger than the needed from the design. These assumptions led to a significant increase of $F_{B,RD}$ and therefore to a reduction in the value of q_F.

  This affects both models similarly and justifies the overall low values presented in Table 13 but does not account for the fact that the regular model present lower values than the irregular one.

  To justify that, it is important to recall that in the linear elastic analysis, used in the design the steel reinforcements, the behaviour factor adopted for the irregular model was reduced relatively to the used for the regular model in 20%, leading to higher design values for the internal forces. Although in the definition of the reinforcement, it was considered the same for both models with the intention of better comparing the results obtained. This assumption led to reinforcement areas in excess comparing to those strictly needed in the regular model and fewer than needed reinforcement in the irregular model. This leads to a reduction in $F_{B,RD}$ on the irregular model, hence an increase in q_F and an increase in $F_{B,RD}$ on the regular model leading to lower values of q_F.

The order of magnitude of the overdesign for both models can be attested by the fact that the seismic loading considered in the non-linear models should be multiplied by a scale factor of 4.09 or 4.64, respectively for the regular and irregular models, for the total collapse to be reached.

4.2.3. <u>Chord Rotation</u>

The results presented so far regard the overall behaviour of the structures, not showing a significant difference between the regular and the irregular models, as a likely consequence of the choice of adopting equal reinforcements in both model.

In order to study the local behaviour of these structures, the chord rotation is measured for the end-sections of the frames of alignment alignments 1, 2, B and C accordingly with Figure 1. Given that the steel reinforcement is equal in both models it is possible to compare the results of the regular and irregular models with the assurance that they are indeed comparable. For the chord rotation the results of the linear analysis are not compared. Figure 4 to Figure 7 present the ratio between the results for the non-linear analysis for the irregular and regular model (δ_θ).

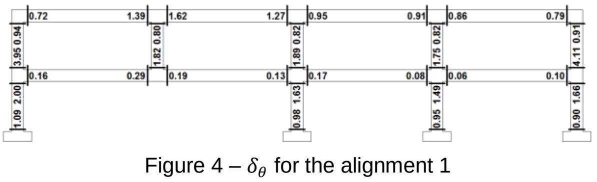

Figure 4 – δ_θ for the alignment 1

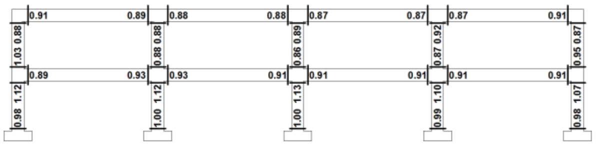

Figure 5 – δ_θ for the alignment 2

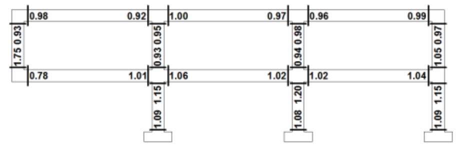

Figure 6 – δ_θ for the alignment B

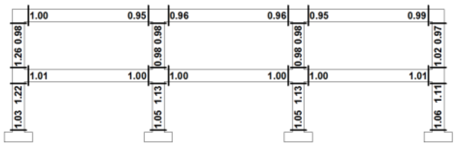

Figure 7 – δ_θ for the alignment C

Based on the values obtained for the chord rotations (CR) the following can be concluded for the beams:

- Considering all the alignments but the one with the irregularity it is possible to conclude that in general the CR are of the same magnitude in the two models. This leads to the conclusion that the presence of the irregularity does not significantly affect the regular beams chord rotations.
- Considering now the alignment 1 (Figure 4), it is possible to conclude that in the transfer beam the chord rotations are particularly low comparing to the same beam in the regular model. Although the geometry of both beams is different, making the comparison between these arguable, it is possible to assume that the results are expected given that the transfer beam is much stiffer.
- In the beams right above the transfer beam and next to the discontinued column there is a significant increase in the CR likely due to the local effect of the vertical component of the seismic action.
- The regular beam perpendicular to the transfer beam which is indirectly supported in the TB (alignment B) has a smaller chord rotation in the irregular model. This has to do with the fact that by removing the column the beam becomes more flexible leading to a lesser concentration of internal forces, hence reducing the CR. This is in accordance with the conclusions taken in the parametric study.

Considering now the results for the columns, the following conclusions may be taken:

- Considering the sections at the connection with the foundations it was concluded that there were no significant variations from the regular to the irregular models. Although, it is important to refer that there is a reduction in CR on the alignment 1, probably due to the increase that occurs in the other end of those columns, imposed by the transfer beam.
- In the columns sections connected to the beams on the first floor there was an overall increase in the chord rotation (of about 10%) in the sections bellow the mentioned beams, especially in the connection to the TB, where the increase was of 70% in average, which is related to the higher stiffness of the TB.

In the sections of the columns above the 1st floor the results were different; with the exception of the columns in the alignment 1, there was an overall reduction in chord rotation. In the columns above the transfer beam, there was a significant increase in the CR for both directions. This leads to an important conclusion that can be taken from the presented results as it is an indirect indicator that there is an accumulation of damage and inelasticity in the zones above the irregularity which could be related to the 20% reduction of the behaviour factor indicated in the EN1998-1 [6].

4.2.4. Axial Force

The following results pertain to the column of the 2nd floor, right above the discontinued column, to determine if there is a significant variation due to the effect of the seismic action. Figure 8 presents the evolution of the axial force along the time for the irregular model for the three considered cases.

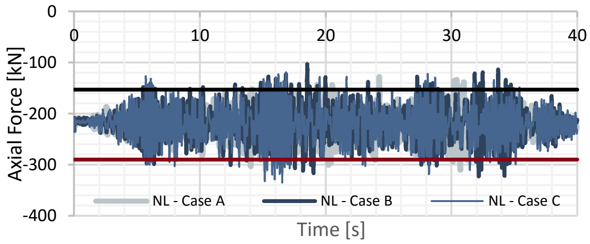

Figure 8 – Axial force on the discontinued column

The major conclusion taken from these results is that, in certain points, the non-linear values are higher than the linear ones. This is not a normal occurrence given that it implies a behaviour factor lower than unity.

4.2.5. Near Collapse Situation

Although the results of the comparison of chord rotations for the regular and the irregular models are very relevant, the absolute values are not very significate, in part due to the overdesign of the structures as mentioned previously.

In order to study the response of the structures in a situation near to collapse, a factor was defined by which the base accelerations should be multiplied to take in consideration the Limit State of Near Collapse (LSNC) as defined in the EN1998-3 [7].

The Limit State of Significant Damage (LSSD) defined in the EN1998-1 [6] used to compute the accelerograms considered in this study considers a return period of 475 years, on the other hand, the LSNC considers a return period of 2475 years. To take into consideration the different return period, the importance factor (γ_I), that was considered unitary in the definition of the elastic and design response spectrum, should now be calculated according with equation.

$$\gamma_I = \left(\frac{T_{LSSD}}{T_{LSNC}}\right)^{-1/k} = 3.00 \tag{6}$$

Being T_{LSSD} the return period for the LSSD (T_{LSSD}=457y), T_{LSNC} the return period for the LSNC (T_{LSNC}=2457y) and exponent k dependant of seismic action (for seismic action type 1 k=1.5).

The structures were analysed again for the new scale factor focusing on displacements, base shear and chord rotation results.

Regarding the displacements and the total base shear, Table 14 and Table 15 present the results for the regular and irregular models regarding the linear and non-linear responses as well as the behaviour factors, q_d and q_F.

Table 14 – Displacements for the LSNC

Direction		x direction		y direction	
Floor		1st	2nd	1st	2nd
Regular	d_L [m]	0.050	0.095	0.051	0.098
	d_{NL} [m]	0.123	0.149	0.122	0.152
	q_d [-]	2.46	1.57	2.39	1.55
Irregular	d_L [m]	0.051	0.097	0.053	0.101
	d_{NL} [m]	0.127	0.150	0.130	0.156
	q_d [-]	2.49	1.55	2.45	1.54

From the presented results the following conclusions may be drawn:

- The displacements in the LSNC obtained for the linear analysis were exactly three times higher than in the LSSD, as was supposed. For the non-linear analysis the displacements on the second floor were also 3 times higher in the LSNC, although for the first floor they were 4 times higher. This is an evidence that the linear analysis underestimates the displacements.
- The comparison of the displacements for the 1st and 2nd floors shows that the displacements were 1.2 and 1.9 times higher on the second floor for the non-linear and linear analysis, respectively. This is a strong indication that the collapse occurs by formation of soft-storey mechanism, furthermore, using a linear RSA the formation of this mechanism is undetectable.
- Comparing the results for the q_d of the LSNC with the ones for the LSSD leads to the conclusions that the results for the second floor are identical and for the first floor there was an increase of approximately 40% which was associated with the formation of the collapse mechanism.

Table 15 – Total base shear for the LSNC

Direction		X	Y	Z
Regular	F_B^{lin} [kN]	20847.3	20600.3	25184.1
	F_B^{Nlin} [kN]	5353.0	5254.6	25330.1
	q_F [-]	3.89	3.92	0.99
Irregular	F_B^{lin} [kN]	19345.8	19345.8	22458.9
	F_B^{Nlin} [kN]	5429.8	5360.0	15039.8
	q_F [-]	3.56	3.61	1.49

Based on the results presented the following conclusions may be drawn in what respects the base shear:

- There is an inevitable increase in the values of q_F due to the fact that the growth in base shear for the linear model is directly proportional to the action (which was 3 times higher) but the increase in the non-linear analysis has a upper limit that is the maximum resisting force of the structure; i.e. the base shear in the linear model increases 3 times when in the non-linear the increase is very low (approx. 1.1 times),
- The fact that values of q_F are higher confirms the reason given for the overall low values obtained for the LSSD due to the overdesign of the structure. Although, the values for the irregular model are still higher than the ones for the regular model, also likely due to the same reason as the indicated presented in the LSSD.

Regarding the chord rotation for the LSNC the values of δ_θ are not presented, preferring to instead present the average ratio between the LSNC and the LSSD CR values (Table 16).

Table 16 – Ratio between the LSNC and the LSSD for the CR

Element	Floor	Irregular Model	Regular Model
Columns	1st	4.54	5.00
	2nd	1.45	1.63
Beams	1st	1.90	1.27
	2nd	1.46	1.22

It was concluded that the increase in chord rotation is more significant in the regular model for the columns and more significant in the irregular model for the beams. It was also observed that the increase in the CR on the columns of the first floor is much higher due to the formation of the collapse mechanism.

In order to assess the suitability of the structure to accommodate the predicted chord rotation, the ultimate chord rotation capacity (θ_{um}) is determined according with the paragraph 3.2.2 of Annex A of the EN1998-3 [7], given by equation (7).

$$\theta_{um} = \frac{1}{\gamma_{cl}} 0.016 \cdot (0.3^{\nu}) \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225} \cdot \\ \cdot \left(\min\left(9;\frac{L_V}{h}\right)\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.25^{100\rho_d}) \tag{7}$$

All the former parameters are defined in the mentioned standard. Given that all the columns in alignment 1 have the same geometry (square cross-section of 0.4 by 0.4m) it is possible to derive most of the values defined for equations (7). Table 17 presents the geometrical properties according with the definitions in the standard and Table 18 presents the internal forces in the sections A, B and C in both models but only for one of the three considered accelerograms (the one with higher values of CR). Sections A and B are the both the end-sections of the column of the first floor of alignment **A1**, the first at the level of the foundations and the other at the connection with the TB. Section C is the end-section of the column of the second floor, right above the irregularity.

Table 17 – Geometrical properties of the columns cross-section

$h = b$ [m]	$h_0 = b_0$ [m]	b_i [m]	A_{sx} [cm^2]	s_h [m]
0.400	0.330	0.153	7.85	0.150

Table 18 – Internal forces for the instant *t* with higher CR

Model	Irregular Model			Regular Model		
Section	A	B	C	A	B	C
N [kN]	-389.8	-39.2	-205.6	-82.4	-125.8	-273.9
M [kNm]	280.8	277.8	271.7	270.9	281.9	284.3
V [kN]	-160.4	181.3	-172.5	-179.3	177.6	-187.6

Based on these values is possible to determine the entry parameters of equation (7). The simplification $\omega = \omega'$ was considered, implying that $\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c = f_c$ is always true and therefore the values of ω and ω' are irrelevant, and were not calculated. Table 19 presents the values necessary to determine the ultimate chord rotation capacity (θ_{um}).

Table 19 – Entry parameters of equation (7)

Model	Irregular Model			Regular Model		
Section	A	B	C	A	B	C
γ_{cl} [-]	1.5	1.5	1.5	1.5	1.5	1.5
h [m]	0.400	0.400	0.400	0.400	0.400	0.400
L_V [m]	1.751	1.532	1.575	1.511	1.588	1.515
ν [-]	0.064	0.006	0.034	0.014	0.021	0.045
f_c [MPa]	38	38	38	38	38	38
f_{yw} [MPa]	500	500	500	500	500	500
ρ_{sx} [-]	0.013	0.013	0.013	0.013	0.013	0.013
α [-]	0.595	0.595	0.595	0.595	0.595	0.595

After the definition of all the parameters, the values of θ_{um} were determined and are presented in Table 20 along with the computed chord rotations (θ_{calc}) and the ration between these two.

Table 20 – Values of the θ_{um}

Model	Irregular Model			Regular Model		
Section	A	B	C	A	B	C
θ_{calc} [mrad]	51.93	50.93	7.55	45.30	32.78	5.54
θ_{um} [mrad]	52.19	53.39	52.16	52.67	53.14	50.77
$\theta_{calc}/\theta_{um}$ [-]	0.99	0.95	0.14	0.86	0.62	0.11

From the results presented the following is concluded:

- In the irregular model, section A has a chord rotation very close to the ultimate capacity which indicates that the structure is very near the situation of collapse, also section B is close to maximum capacity. The regular model is not so close to it's the corresponding maximum capacity.
- Regarding the values for the section C both models are far from there maximum capacity. Although the chord rotation is higher in the irregular model, as could be expected.
- The discrepancy between the values for the columns in the 1st and 2nd floor are due to the formation of a soft-storey collapse mechanism.
- Comparing the values of $\theta_{calc}/\theta_{um}$ for the irregular and regular models it is possible to conclude that, in average the irregular model presents values 30% higher than the regular model. This may be considered as an indication that the reduction in the q_F of 20%, is suited for this irregularity.

5. Conclusions

5.1. Study 1: Parametric Linear Elastic Analysis

The development of the parametric study was supported in a reasonable number of cases. Although many other could have been considered, the set of layouts presented and compared proved to be sufficient to conclude the following:

- When a discontinued column is considered in the architectural design it is imperative for the cross-section of the transfer beam and of the columns to be re-dimensioned in order to endow the structure of a suitable behaviour for the serviceability and ultimate limit states.
- The existence of the irregularity, as considered in this study, leads to a structure with identical stiffness when compared with the corresponding regular situation. Moreover, in the direction of the transfer beam the structure is somewhat stiffer than in the perpendicular direction.
- Stiffer structures with the same plan layout have a higher aggravation, than softer structures, in the internal forces due to the irregularity. On the other hand, considering the in-plan layouts, the case with a shorter span (4m) presents a higher aggravation than the case with a longer span (7m) when considering the regular beams and the columns; however this trend is reversed when the TB is considered. The results for floor plan B, with shear walls being the main load-resisting system (and not the frames), show that the negative effects of the irregularity, in terms of the internal forces, are alleviated for the regular beams and columns at the cost of an aggravation for the transfer beams and shear walls.
- It was also concluded that the design of the transfer beam is determined by the gravity loads. This is an indicator that the problem associated with this irregularity is not one of resistance but of ductility.
- The effect of the irregularity is more detrimental in the perpendicular direction to the transfer beam.
- Other important result of this work is the fact that the seismic action acting in the direction perpendicular to the TB originates a bending moment on that beam with a magnitude similar to the bending moment originated by the seismic action on the direction of the beam.

5.2. Study 2: Non-Linear Inelastic Analysis

With the development of the non-linear study, even only considering one irregular situation, the following conclusions were draft:

- Considering the global behaviour of the structure the existence of the discontinued column leads to a structure with a similar response to regular case for the seismic action acting in the direction of the transfer beam.
- The displacements obtained for the regular and the irregular models are similar for both the linear elastic analysis and the non-linear inelastic analysis. However, the results for

each of these analyses are different, leading to values of q_d higher than anticipated. These results are also confirmed by the values presented for the total base shear, for which the q_F values were lower than expected. This outcome (regarding q_F) is mostly due to the decisions considered during the design of the steel reinforcement, as previously explained.

- The fact that the results for the displacements and base shear are similar in both the regular and irregular cases highlights the conclusion that the effect of the discontinued column is not a global one but local.
- The results obtained for the chord rotations confirm this assumption (regarding irregularity local effect) given that there is a clear concentration of inelasticity, mostly in the sections above the TB, but also in the ones below. This indicates that the presence of this irregularity leads to an aggravation of the seismic effects, which must be duly considered in the design. It is also possible to conclude that, due to the cross-section redesign of the TB, this element (TB) is not considerably affected by the seismic action. However it leads to a concentration of inelasticity in the columns precipitating the formation of a soft-storey mechanism
- Analysing the results for the limit state of near collapse is possible to conclude that the existence of this irregularity leads to the earlier failure of the structure.
- Also, the existence of the discontinued column affects the chord rotations which represents an increase approximately 30% comparing with the same column in the regular model.
- It is clear from the presented results that a linear elastic response spectrum analysis underestimates the actual axial force in the discontinued column. This means that the behaviour factor should be inferior to 1.0, which is an unusual situation. It is important to mention that in the EN1998-1 [6] the behaviour factor for the design vertical response spectrum is 1.5. The results presented in this work point in the direction that considering a behaviour factor, as proposed in the previously mentioned standard, leads to an underestimation of the bending moment in the TB, which might prove to be a serious problem.

5.3. General Conclusions

The major conclusion taken from this dissertation is that the existence of a discontinued column has a negative effect on the structure that should not be overlooked.

Although the EN1998-1 [6] envisages a reduction on the behaviour factor in order to consider this irregularity, this measure globally affects the structure, leading to excessive reinforcement on all the columns (regardless of their position with respect to the discontinued column).

A more rational alternate solution would consist in assuming special rules of detailing in the zones next to the TB, endowing these zones with adequate ductility, ensuring a suitable behaviour under seismic loading. This way, the local concentration of inelasticity would be properly addressed, assuring that the chord rotation capacity is higher in those zones.

6. Future Studies

The development of this work leaves a number of other aspects that were not considered and that need to be addressed in order to fully understand the effect of discontinued columns and the correct way to consider them at the design stage. Some of the possible studies that might be developed based on the forwarded results are:

Regarding the linear studies many other situations could be considered, although one parameter that should be taken in consideration is the number of discontinued columns.

Regarding the non-linear behaviour many other studies might be developed. One possible line of research could consist in the study of the adequate values of the behaviour factors (for both horizontal and vertical directions), to be considered at the design stage. Complementary to the former approach, the study of special detailing rules for the zones where the accumulation of inelasticity occurs could be a promising line of development.

The comparison of the two presented lines of research (either separate or joint) in terms of ultimate and serviceability limit states response, structural efficiency and cost, would also be a considerable enrichment to the state of the art.

Finally, the study of progressive collapse would be important. Not only regarding a regular structure that had a column failure situation, creating a case similar to the studied in the dissertation, but also the effects of the occurrence of some problems in the pre-stress tendon.

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