Abstract

The construction industry is continuously searching new optimal solutions. Mixed steel-concrete structures are therefore promising options that profit from both materials according to their best performance. This paper investigates the response of this type of structures by means of numerical computations. The multi-storey building structure erected in Cardington for testing in 1993 is used as study case. Two different models have been developed for comparison. One model is based on the "original" structure which represents a reference steel solution. In the other model a mixed steel-concrete solution is implemented. The analysis includes the influence of the joint behaviour on the structural response. Models with rigid and semi-rigid joint modelling are compared. Based on a 1^{st} order linear elastic, buckling and 2^{nd} order linear analysis, the results of both structural solutions are presented and discussed.

Keywords: joint, mixed, model, rigid, semi-rigid, steel-concrete, structure.

1 Introduction

In construction, like in other industries, the optimization of resources is the way to improve competitiveness and to face the continuous increasing challenges in engineering. New technologies such as mixed construction, where materials are combined according to their best performance, are serious options for the present and future of building construction. In the past, engineers choose to use either concrete or steel separately. Nowadays, the combined use of concrete and steel in form of composite or mixed structures is an efficient alternative to this traditional construction mentality. Good examples of such practice are many high rise buildings that combine central concrete cores with steel and composite members.

In multi-storey buildings, flexibility and consequent sensibility to horizontal actions increases with height. The functionality of the structure requires comfort to their users/occupants. Excessive deformations have then to be avoided by providing "centres" of stiffness in the structure, either by bracing systems in the case of steel structures or by concrete cores in the case of concrete structures. Additionally, the vertical communications of the building are fundamental for evacuation. These parts of the structure require robustness and therefore are often made of concrete. Usually, this option implies that all structure is in concrete, as it is the "easiest" solution for architects and designers. Even though a combined used of concrete and steel/composite members can provide an optimized solution. Often, the obstacle to these options is the fact that designers are required to have multi-disciplinary design knowledge, having to deal with steel and concrete.

It is common knowledge that concrete and steel provide different structural responses. Where concrete is better for compression members, steel performs better in tension. While rigidity may be gained with concrete, the desired ductility may be obtained with steel. Thus, the combined use of these two materials allows an optimisation of the members and of the structure. Limiting to the use of concrete and steel separately may be reflected on non-optimized solutions.

In mixed structures, designers have to design members of different nature and steel-to-concrete joints. If for the design of members, the European codes [1], [2], [3] are already integrated in the habits of designers providing essential guidance, the design of joints, in such structures, becomes an obstacle. The problem does not rely on the absence of options to connect both materials, as one can find in the market a varied system of fastening solutions. With the Technical Approvals (EOTA) [4] and the Technical Specifications (CEN) [5] design guidance and product specifications are available for designers. However, it still requires to designers some effort due to the lack of simplified design methods including these solutions.

In the present paper the mixed building technology is put in practice. A common multi-storey office building is used to compare the structural response between a steel and a mixed solution. The steelwork framed building erected and submitted to fire tests in Cardington in 1993 [6], is used as reference structure (see Figure 1). The building has been designed to be a typical example of the type of braced structure and load level that are commonly found in UK. Because it represents a typical modern multi-storey building and has been the subject for many researches, it was found suitable for the proposed analysis.

The comparison of structural response between a steel and a mixed structure is performed by means of numerical models developed in Sofistik Structural Desktop [7]. Two models have been developed, one for each structural solution. In the present analysis the effect of joints is included however, as mentioned before the absence of models for steel-to-concrete joints implies, at this stage of the research, a simplification in the approach. Hence, in what concerns the mixed structure, the joint properties of the steel solution are used instead of the real properties. Thus, models with rigid and semi-rigid joint modelling are compared using a 1st order linear elastic analysis, a buckling Eigen analysis and a 2nd order linear analysis.

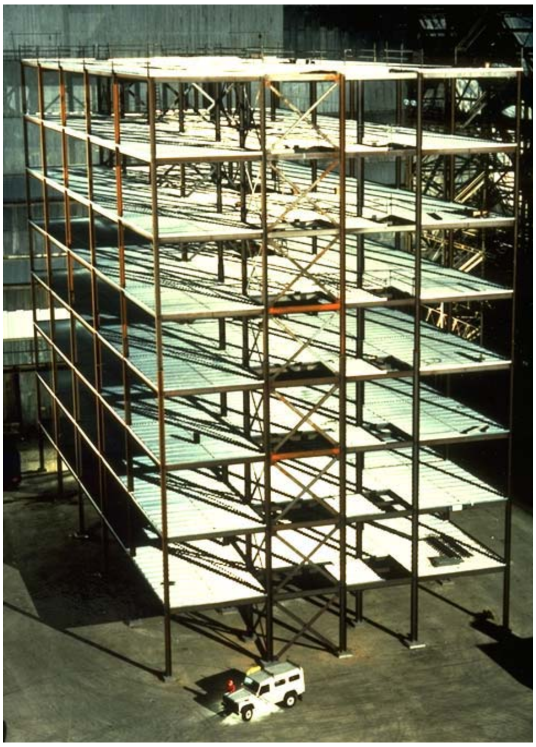


Figure 1 – Structure submitted to fire tests in Cardington in 1993.

2 Reference structure

The structure erected and subjected to several fire tests in Cardington was chosen as a typical example of an office building with a braced structure and subjected to load levels that are commonly found in the UK. On plan, the building covered an area of 21m by 45m. The total height is approximately 33m, composing 8 floors with approximately 4.3m height. The floor layout, illustrated in Figure 2, presents at each side of the building openings for the fireman's access and escape stairwell with 4m x 5m. In the west side there is a second opening, 4m x 2m, for a goods lift. In the middle of the building there is a central lift core with 9m x 2,5m. On the south elevation (between alignments C and D) there is a two storey ground floor atrium of 9m x 8m.

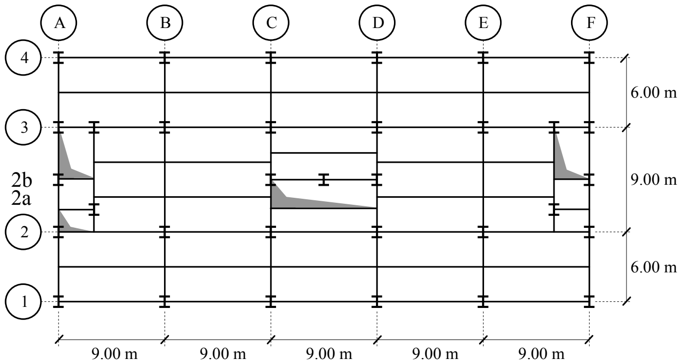


Figure 2 – Floor layout.

The building structure was designed according to the British Standard BS5950 [7]. The structure was designed as a braced frame with lateral restraint provided by cross bracing with flat steel around the vertical shafts. The central area, referred before, required the deviation, of two columns in the first two storeys, from alignment 2 in 2m (as result, two beams from alignment C1-C2a and D1-D2a have a span of 8m). Beams were designed simply supported acting compositely with the floor slab. The location of the columns and bracing system is illustrated in Figure 3.

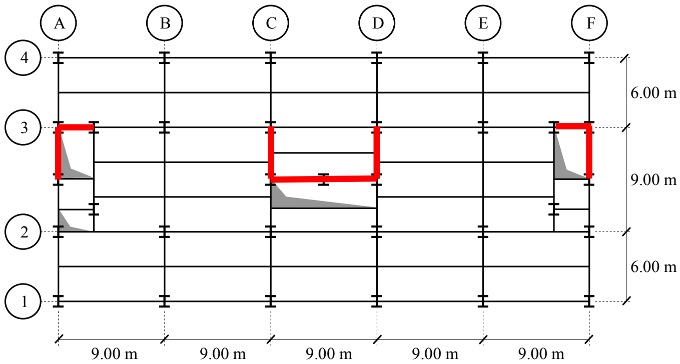


Figure 3 – Location of bracing systems.

3 Numerical Tool

The numerical models have been developed in Sofistik Structural desktop [7]. Sofistik is a finite element analysis (FEA) software with a modular structure for pre-processing, processing and post-processing. The software possesses a standard pre-processor based on Autocad. Sofistik allows the following analyses: 3D finite element analysis (ASE); 3D frame analysis (STAR); Analysis of slabs, walls, frames and grillages (Slab designer pro); Linear and non-linear strain/stresses analysis (TALPA); Soil structure interaction (HASE); Analysis of pile foundations (PFAHL); Analysis of potential problems (HYDRA); Multiphysical code for computation of fluid dynamics that can be used for dynamic wind analysis of bridges and buildings (PHYSICA).

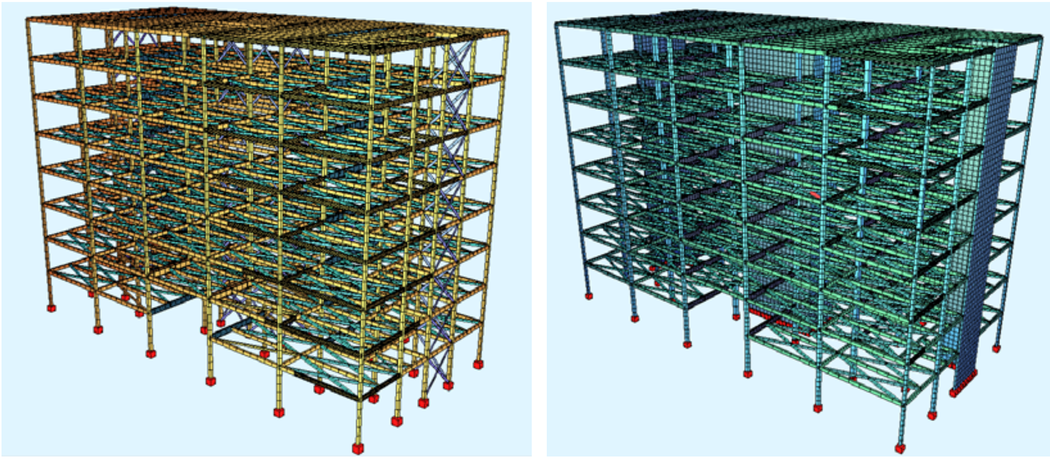
The following elements are available in this software: beam elements, pile elements, truss and cable elements, spring elements, boundary elements, shell elements and volume elements.

For more information about the software, reference is made to the Sofistik web site www.sofistik.com.

4 Numerical models

4.1. General description

For each structural solution a model has been developed. The models are 3D reproducing the entire structure as illustrated in Figure 4. The dimensions of the structure presented in §2 were followed. In order to differentiate the models, the steel solution, due to similarity to the real erected structure, is named Original while the mixed solution is identified as Mixed.



a) b)

Figure 4 – 3D developed models of the Cardington Building structure: a) Original model; b) Mixed model.

In order to have common steel profiles used in Europe, a conversion of initial cross-sections (British steel profiles) to equivalent ones (common European profiles) has been made in previous studies [9]. These modifications have been here adopted. From Table 1 to Table 3 are identified the geometric characteristics (cross-sections) of the different members. The lightweight slabs have not been modelled. Instead, equivalent diagonal elements in the floors plan have been used, (see Figure 5). These should provide to the models a stiffness equivalent to the slab (in the floor plan). Consequently, no composite behaviour of the floor beams has been considered. The composite behaviour was taken into account in the mentioned conversion of profiles. In what respects the differences between the models developed, these rely on the nature of the vertical elements and of the lateral restraining systems. In the steel model the entire structure is made of steel. In the mixed model, columns and walls are made of reinforced concrete. In the Original model, columns cross-section has been reduced with the height of the column. In the Mixed model, such reduction has not been performed.

	Original	**Mixed**
IPE 360	All longitudinal beams	
IPE 400	All transversal beams in alignments A, F, A', E', B1-2, B3-4, E1-2, E3-4, C and D except 1st and 2nd floor in alignments C1-2a, D1-2a (in all floors for Mixed model)	
IPE 600	1st floor in alignments C1-2a, D1-2a	All floors in alignments C1-2a, D1-2a
2 HE 700A	2nd floor in alignments C1-2a, D1-2a	-

Table 1 – Geometric characteristics of the beams.

Columns	Model	Ground floor – 2nd floor	2nd floor – 5th floor	5th floor – 8th floor
A1, A4, F1, F4	Original	HE 260 B		
	Mixed	0,25mx0,25m		
B2, C2, D2, E2, C2b, C'2b, D2b, B3, C3, B3, E3	Original	HE 340 B	HE 320 B	HE 260 B
	Mixed	0,45mx0,45m		
B1, C1, D1, E1, A2, F2, A3, F3, B4, C4, D4, E4, A2b, A'3, F2b, E'3	Original	HE 320 B	HE 260 B	
	Mixed	0,35mx 0,30m		
A'2^a, E'2a	Original	HE 320 B	HE 260 B	
	Mixed	0,25mx0,25m		

Table 2 – Geometric characteristics of the columns

Original	Flat plates 250x15 and 200x10
Mixed	Shear walls 0,3m thick

Table 3 – Geometric characteristics of lateral restraining systems.

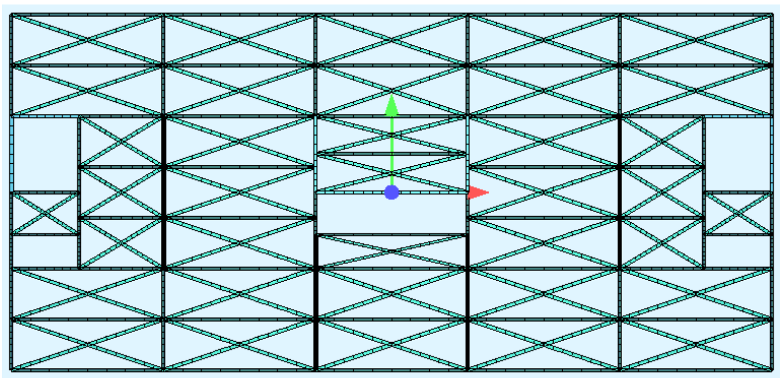


Figure 5 – Plan of the floors: equivalent diagonal elements modelling the slab stiffness on the floor plan.

A last difference between the two models are in the columns referred in §2 (C2a and D2a). In the Original model, these columns start in alignment 2a (position C and D) and from 2nd floor to top are "moved to alignment 2. For evident reasons, in the Mixed model, these columns are positioned in alignment 2a from bottom to top.
Concerning materials several have been defined for the different members as: structural steel (S355 and S275), reinforcing steel (A500) and concrete (C30/37). In the present paper only a linear elastic analysis is presented, consequently the fundamental material parameter is the Young modulus.
In what respects to finite elements, three types have been used: beam elements, plate elements (thick walled) and cable elements. The beam elements are 3D with six degrees of freedom, shear deformation effects are also included. The plate elements have twelve degrees of freedom and the structural behaviour is based on Midlin's plate theory. The cable elements can transfer only axial forces. The maximum size of the finite elements is limited to 0,54m.
A final remark to the Mixed model considering the modifications made from the steel model should be stated. The present work is part of a research work on steel-to-concrete joints therefore, it was of major interest to modify columns to

concrete even if, in practical point of view, it would be preferable to use steel or composite members.

4.2. Joint modelling

The influence of the joint behaviour, on the structural analysis, may be non-negligible according to its properties. In the present paper, the evaluation of the effect of the joint modelling in the response of the structure is included. For each model, two cases have been analysed concerning the behaviour of the joints transferring bending moment to columns: rigid and semi-rigi. The inclusion of the joint properties in the global analysis followed the simplified joint modelling prescribed by EN 1993-1-8 [10] as illustrated in Figure 6. Rotational springs are used to incorporate the behaviour of the joints in the analysis. In the case of linear elastic analysis, these springs are simply characterized by the rotational stiffness.

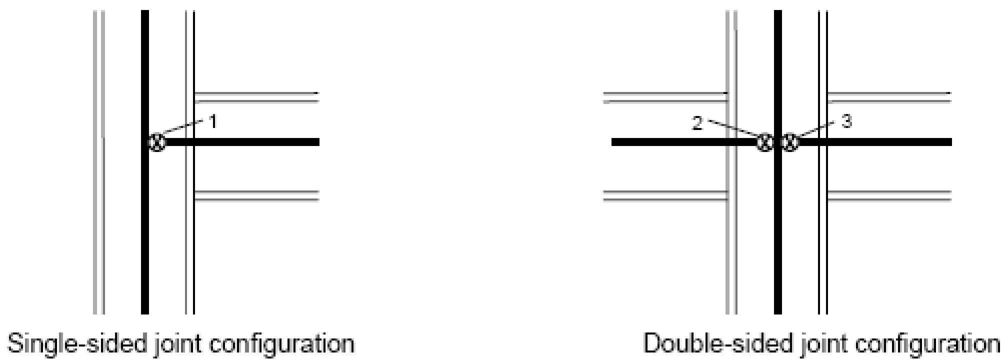


Figure 6 – Simplified joint modelling according to EN 1993-1-8 [10].

As mentioned before, in the real structure all beams have been designed as pinned. Here, different support conditions have been established. In the models where the joint behaviour is “neglected”, the following support conditions are used: pinned, for beams in longitudinal direction; fully rigid, for beams in transversal direction; fully rigid, for column bases. In the second case, which includes the joint behaviour, the following has been assumed: pinned, for beams in longitudinal direction; semi-rigid for beams in transversal direction; semi-rigid and pinned, around major and minor axis, respectively, for columns bases. In this second case, for the Mixed model, the columns bases support remained fully rigid.

In what respects to the joint properties, at this stage no simplified model has been developed to evaluate steel-to-concrete joints. Subsequently, the determination of joint properties has to be done using sophisticated models. The scope of the research, where the presented work is included, is to derived simple models, based on the component method, for such type of joints. So, as a first approximation, and in what concerns the Mixed model, joint properties determined for the Original model (steel joints) have been used in Mixed model.

In order to obtain the joint properties of the Original model, the software CoP [11] has been used. The software is a commercial program that has been developed for

the design of joints in steel building frames according to [10]. For more information about this software reference is made to the CoP web site (www.fw-ing.de/software). The design of joints with CoP follows the component method. The design joint loads have been obtained from the first case analysed which considers moment connections as rigid. The application of the component method implies an iterative procedure however; here, just one iteration has been used to determine the joint properties.
In such multi-storey building, many joint configurations emerge. In order to reduce the variety of joint geometries, group of joints have been defined according to the type and length of the members they connect. As an example, the values for portal frame of alignment B are presented (this portal frame is later used for discussion of results). The different joint configurations of this portal frame are indentified in Figure 7. Table 4 presents the joint properties obtained for JD4 and JS1.

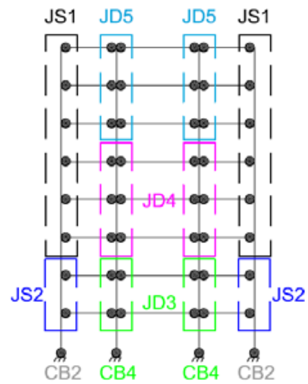


Figure 7 – Different joint configurations in portal frame of alignment B (and E).

Joint ID			$S_{j,ini}$ [kN.m/rad]	$S_{j,ini}/\eta$ [kN.m/rad]	$M_{j,Rd}$ [KN.m]	Classification Stiffness	Classification Strength	Failure Mode
JS1		M(-)	47254	23627	-207	Semi-rig.	Partial	CWS
		M(+)	27159	13580	120	Semi-rig.	Partial	CWC
JD4	Left	M(-)	106740	53370	-387	Semi-rig.	Partial	CWC
		M(+)	71616	35808	236	Semi-rig.	Partial	CWC
	Right	M(-)	37652	18826	-177	Semi-rig.	Partial	CWC
		M(+)	34275	17137	148	Semi-rig.	Partial	EPB

Table 4 – Joint properties of JD4 and JS1 joints.

In the above table, the sign (+) and (-) in the bending moment means tension flange of the beam is in bottom (hogging bending moment) and upper flange (sagging bending moment), respectively. The failure modes are: column web panel in shear (CWS); column web in compression (CWC); and end-plate in bending (EPB).

4.3. Load cases and combination of load cases

The actions applied to the models have been established according to EN 1990 [12] and EN 1991 [13], [14]. The loads considered are listed in Table 5. Frame imperfections have been included by means of horizontal forces determined according with [2]. The load cases defined for both models are identified in Table 6. Two cases have been considered for the wind action: W0º – wind acting in the longitudinal direction of the structure; W90º – wind acting in the transversal direction of the structure.

Actions	Loads
Permanents loads (G)	Self-weight of structural elements:78,5 kN/m^3 for steel and 25 kN/m^3 for reinforced concrete
	Slab self-weight with 13cm of thickness: 12,5kN/m^3
Live loads (Q)	Office building – floors and roof: 3kN/m^2
	Movable partition walls: 0,5kN/m^2
Wind (W)	According to [14]
Frame imperfections (I)	Equivalent horizontal forces according to [2]

Table 5 – Loads considered.

Load Case	Description
LC1	Permanent loads
LC2	Live loads
LC3	Wind for α = 0º
LC4	Wind for α = 90º
LC5	Frame imperfections in longitudinal direction
LC6	Frame imperfections in transversal direction

Table 6 – Identification of defined load cases.

Slabs are considered working in the transversal direction being supported by longitudinal beams. Subsequently, dead and live loads have been considered uniformly distributed and applied to these beams. No load was directly applied to the transversal beams (except self-weight). The application of wind loads to the structure has been assumed uniformly distributed along external columns. Geometric imperfections have been introduced as nodal loads applied at the top of the columns at each floor.

Concerning the combination of load cases, two groups have been considered according to [12]: ultimate limit states (fundamental combination) and serviceability limit states (frequent combination). Each group is defined by equations (1) and (2), respectively. In Table 7 are presented all the combinations considered and the factors adopted.

$$E_d = \gamma_{G,j} \sum_j G_{k,j} + \gamma_{q,1} Q_{k,1} + \sum_i \gamma_{q,i} \psi_{0,i} Q_{k,i} \tag{1}$$

$$E_d = \sum_j G_{k,j} + \psi_{1,1} Q_{k,1} + \sum_i \psi_{2,i} Q_{k,i} \tag{2}$$

Where: $\gamma_{G,j}$ is partial factor for permanent action j; $\gamma_{q,1}$ and $\gamma_{q,i}$ are the partial factor for variable actions for leading and non-leading variable, respectively; $\psi_{0,i}$ is factor for combination value of a variable action i; $\psi_{1,i}$ is the factor for frequent value of a variable action; and $\psi_{2,i}$ is the factor for quasi-permanent value of a variable action.

Designation	Leading Variable	Considered combination
Comb 1 (ULS)	Live load	$E_d = 1{,}35LC1 + (LC5 + LC6) + 1{,}5LC2 + 1{,}5 * 0{,}6LC3$
Comb 2 (ULS)	Live load	$E_d = 1{,}35LC1 + (LC5 + LC6) + 1{,}5LC2 + 1{,}5 * 0{,}6LC4$
Comb 3 (ULS)	Live load	$E_d = 1{,}35LC1 + (LC5 + LC6) + 1{,}5LC2$
Comb 4 (ULS)	W0º	$E_d = 1{,}0LC1 + (LC5 + LC6) + 1{,}5LC3$
Comb 5 (ULS)	W90º	$E_d = 1{,}0LC1 + (LC5 + LC6) + 1{,}5LC4$
Comb 6 (ULS)	W0º	$E_d = 1{,}35LC1 + (LC5 + LC6) + 1{,}5LC3 + 1{,}5 * 0{,}7LC2$
Comb 7 (ULS)	W90º	$E_d = 1{,}35LC1 + (LC5 + LC6) + 1{,}5LC4 + 1{,}5 * 0{,}7LC2$
Comb 8 (SLS)	Live Load	$E_d = 1{,}0LC1 + 0{,}5LC2$
Comb 9 (SLS)	W0º	$E_d = 1{,}0LC1 + 0{,}2LC3$
Comb 10 (SLS)	W90º	$E_d = 1{,}0LC1 + 0{,}2LC4$

Table 7 – Load case combinations.

5 Discussion of results

5.1. General

The analysis performed consisted in a 1st order linear elastic analysis, an Eigen buckling analysis and a 2nd order linear elastic analysis. The two structural solutions are compared in terms of: distribution of horizontal forces to vertical elements, deformations, internal forces in members, joint loads, buckling Eigenvalues. Simultaneously, the influence of the semi-rigid behaviour of moment connections on the structural response is included in the comparison. The following models are compared: Original model with rigid joint modelling of moment connections (OR); Original model with semi-rigid joint modelling of moment connections (OSR); Mixed model with rigid modelling of moment connections (MR); Mixed model with semi-rigid modelling of moment connections (MSR). Finally, the influence of second order effects on the different solutions is shown and commented in the last section of the present chapter. More detailed results are given in [15].

5.2. Distribution of horizontal loads to vertical members

The distribution of horizontal forces amongst vertical elements of the structure depends on their stiffness. Because of their higher stiffness, shear walls and bracing systems are expected to absorb much higher horizontal loads when compared to isolated columns. In Figure 8 are shown the horizontal load distribution in terms of percentage of the applied horizontal load. The loads on isolated columns have been added and compared with each bracing system/shear wall. The presented results correspond to combination 4 and 5. Steel bracing systems and shear walls are identified as B and W, respectively. The index refers to position of the respective system: 1 for left, 2 for middle and 3 for right (according to Figure 3). C is used to identify all isolated columns.

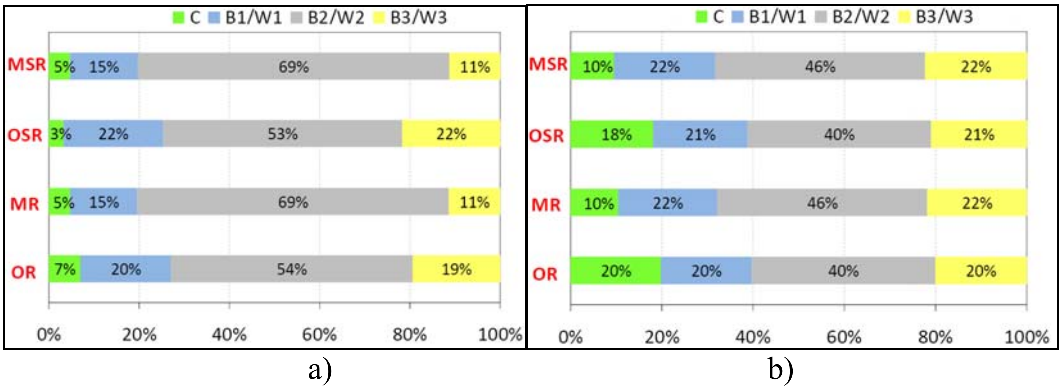


Figure 8 – Distribution of horizontal amongst vertical elements: a) Combination 4 (F_x); b) Combination 5 (F_y).

The results confirm that bracing systems/shear walls absorb the majority of the horizontal loads introduced in the structure. For W90 (combination 5), higher loads are taken by isolated columns when compared with W0 (combination 4). This may be justified by the fact that, for this wind direction, loads achieve columns before bracings systems/walls (see disposition of vertical members in Figure 13). Consequently, part of the loads is transferred to supports before reaching the centre of stiffness (bracing systems or walls). In what respects to the nature of the structure, main difference appears in higher load taken by the central wall in comparison with equivalent steel bracing systems. The difference is about: 15% for W0 and 6% for W90. Concerning the joint modelling, the distribution observed in the Original model shows a quite small influence. While in the Mixed model, because columns bases have been model as rigid, in both MR and MSR, no variation is noticed.

5.3. Deformations

The average values of lateral displacements measured at each floor, in the different alignments (see Figure 2), are shown in Figure 9. The presented results correspond to combination 4 and 5. For each combination only the displacements in the relevant direction are plotted (d_x in combination 4 and d_y in combination 5).

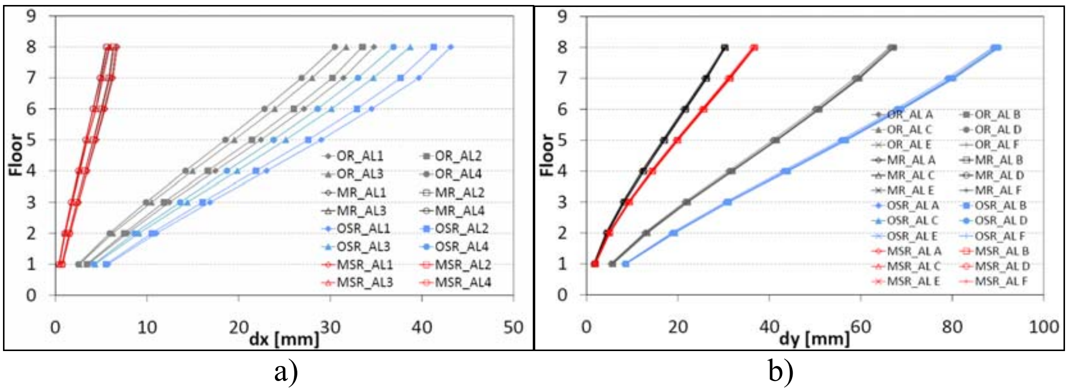
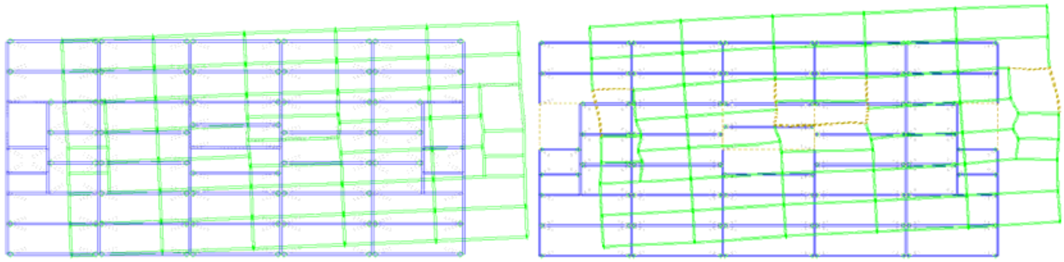


Figure 9 – Floor lateral displacements: a) Load Case Combination 4 (d_x); b) Load Case Combination 5 (d_y).

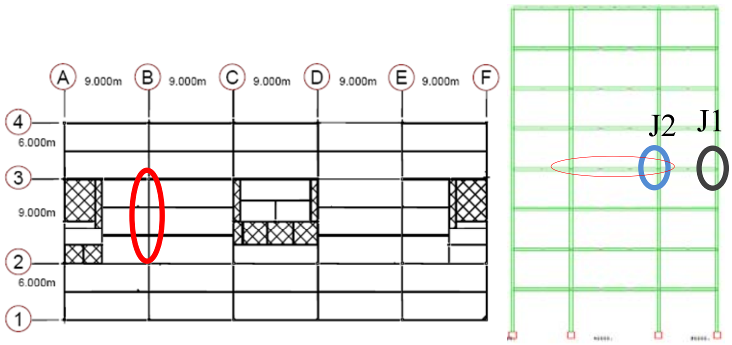
A more flexible behaviour of the Original model is observed in the above charts. The differences become clearer with the height of the structure (affected by the deformation of the bottom storeys). The average ratio (Original/Mixed) is 5,3 and 2,6 in the X direction and Y direction, respectively. Figure 9-a) allows the identification of structure rotation in the horizontal plane. In alignment 4, smaller deformations have been obtained. In this alignment no bracing system or shear wall exists. Consequently, being closer to the bracing systems/walls, one should expect that alignment 3 deforms less. The floor rotation, in the horizontal plane, is confirmed in Figure 10. The in plane floor deformation may be checked. Such behaviour is explained by the eccentric disposition of the bracing systems/shear walls in the X direction (remember Figure 3).
Concerning the joint modelling, differences are clear, for both directions, in the Original model. For the Mixed model, differences are mainly visible for the Y direction. The average increase, for the deformation in Y direction, in the Original model is about 17*mm* while in the Mixed model is 5*mm*. Here, not only the stiffness of the different models plays a role, but also the fact that the column bases in the Original model are semi-rigid.



a) b)

Figure 10 – Rotation of the structure on the horizontal plane: a) Original model; b) Mixed model.

In what respects to the deformation of beams, a beam from the portal frame of alignment B has been selected to compare results. The selected beam is identified in Figure 11. The max span deformation measured, for all the combinations considered, is compared in Figure 12. Here, the influence of stiffer columns in the Mixed model, supporting the selected beam, reflects the smaller deformation obtained in this model.
Concerning the joint behaviour, an obvious effect is verified. However, smaller deformations have been measured in the Mixed model than in the Original model. This confirms the influence of the columns stiffness on the beam behaviour.



a) b)

Figure 11 – Selected beam for member results: a) Floor plan; b) Portal frame of elevation B.

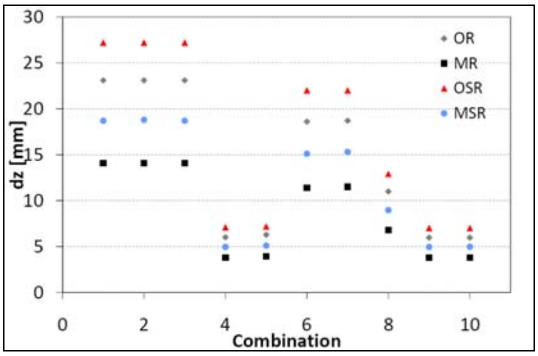


Figure 12 – Comparison of beam deflection (beam represented in the previous figure) for all combinations considered.

5.4. Internal forces on the structure

In Figure 13 are compared the bending moment diagrams developed in the beam represented in Figure 11. The presented results correspond to combination 3 where the leading variable is the live load. The deviations observed are summarized in Table 8. For interpretation of the referred table take in consideration the following: in XvsY, ↑ means that Y > X and ↓ means that X > Y, or ↑ increase of Y in relation to X and ↓ decrease of Y in relation to X. As noticed in the deformation of the beam, the effect of stiffer columns in the Mixed model is again verified. In comparison to the Original model, bigger bending moments develop at the beam ends and smaller in the beam span.

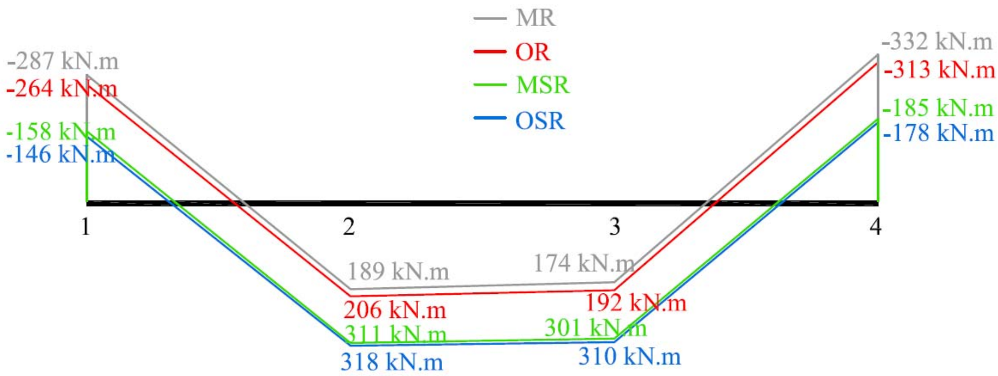


Figure 13 – Bending moment (M_y) distribution in member for Combination 3: beam represented in Figure 11.

Beam Sections	OR vs MR	OSR vs MSR	OR vs OSR	MR vs MSR
1	↑ 8%	↑ 8%	↓ 45%	↓ 49%
2	↓ 9%	↓ 2%	↑ 54%	↑ 69%
3	↓ 10%	↓ 3%	↑ 61%	↑ 73%
4	↑ 6%	↑ 4%	↓ 43%	↓ 44%

Table 8 – Variation of bending moments on beam sections.

In what respects to the joint modelling, the distribution of bending moment in the beam varies according to the joint properties. Consequently, the “desired” distribution of internal forces is dependent of the joint design. In practical terms, what is relevant is to reduce the complexity of the joint assuring the stability of the connected members and the safety performance of the joint. Hence, the presented results including the joint behaviour reflect the realized design of the joint. Other distribution of bending moments would be expected if a different joint design was performed.

5.5. Joint loads

In Figure 11, joints J1 and J2 have been identified. These have been selected to typify the developed joint loads. J1 is a single sided joint connecting a 6*m* span beam (IPE 400) to a façade column. J2 is a double sided joint connecting, on the left side a 9*m* span beam (IPE 600), and on the right, a 6*m* span beam (IPE 400) to an internal column. In OR and MR models, both joints have been considered fully rigid. From Table 9 to Table 11 joint loads are presented for combinations 1, 2 and 3. The sign minus means: hogging bending moment (M_y) in the beams; compression (N) in the beams; shear force (V_z) downwards direction according to member orientation (from left to right).

Combination	Original			Mixed		
	M [kN.m]	N [KN]	V [kN]	M [kN.m]	N [KN]	V [kN]
Rigid						
1	-147	69	-105	-120	55	-96
2	-181	25	-156	-137	18	-102
3	-149	12	-105	-121	2	-96
Semi-rigid						
1	-99	67	-109	-91	55	-105
2	-120	23	-116	-101	18	-108
3	-100	10	-109	-91	3	-105

Table 9 – Joint loads in J1 (see Figure 11).

Combination	Original			Mixed		
	M [kN.m]	N [kN]	V [kN]	M [kN.m]	N [kN]	V [kN]
Rigid						
1	-311	98	-170	-331	65	-171
2	-380	25	-186	-377	-6	-182
3	-313	35	-171	-332	5	-171
Semi-rigid						
1	-177	76	-164	-184	66	-164
2	-222	0	-174	-207	-5	-170
3	-178	10	-164	-185	5	-165

Table 10 – Joint loads in J2, left-hand side.

Combination	Original			Mixed		
	M [kN.m]	N [kN]	V [kN]	M [kN.m]	N [kN]	V [kN]
Rigid						
1	-151	82	105	-177	58	114
2	-122	15	94	-157	1	108
3	-150	18	105	-174	2	114
Semi-rigid						
1	-69	83	97	-83	59	102
2	-51	15	90	-75	2	98
3	-68	16	97	-83	3	102

Table 11 – Joint loads in J2, right-hand side.

As verified in the previous section, the presented results show higher bending moments in the Mixed model for J2 whilst in J1 the opposite is notice. Such results should be explained by the unequal stiffness of left (column B3) and right (column B4) beam supports in the Mixed model. In this model, column B3 is stiffer than the façade column B4. B3 attracts more load than B4. In the Original model, at the 4th floor, both columns have the same profile.

5.6. Buckling Eigenvalues

A buckling analysis allows the determination of the critical load factors (α_{cr}) of the structure, for a given loading. Over such loading conditions, the structure (or local member) reaches a great loss of axial stiffness and a lateral deflection results. Low critical load factors imply the consideration of second order effects in the determination of the internal forces and deformations. According to [2], these may be neglected if the critical load factor is bigger than 10 or 15, in the case of an elastic or plastic analysis, respectively. In Table 12 are presented the two first buckling Eigenvalues for the four models discussed in the present paper. The listed values have been determined for all the load case combinations at ultimate limit state.

Combination	OR		MR		OSR		MSR	
	Mode 1	**Mode 2**	**Mode 1**	**Mode 2**	**Mode 1**	**Mode 2**	**Mode 1**	**Mode 2**
1	5,83	6,29	8,25	8,29	5,52	6,34	8,20	8,24
2	5,79	6,24	8,31	8,31	5,44	5,49	8,26	8,26
3	5,84	6,29	8,26	8,27	5,53	6,34	8,22	8,22
4	14,81	22,21	29,91	30,65	13,65	21,32	29,75	30,49
5	10,66	11,39	16,83	17,50	7,81	8,24	14,97	15,50
6	7,23	7,79	10,16	10,24	6,87	7,08	10,10	10,18
7	6,61	6,90	10,31	10,31	5,10	5,29	10,24	10,25

Table 12 – Buckling Eigenvalues for modes 1 and 2 for all combinations at ultimate limit state.

The smaller Eigenvalues obtained in the Original model reflect a less stiff solution. Nevertheless, according to the obtained values, both solutions require the consideration of second order effects, even for an elastic analysis. In what respects to the loading case, Combinations 1 to 3 provide lower Eigenvalues justified by the fact that the leading variable is a live load on floors (higher compression on vertical members). Axial force in members is smaller in Combinations 4 and 5 as these loads are considered favourable. Thus, for these two combinations, an elastic analysis may be performed neglecting the second order effects. In the case of the Mixed model, these effects may be neglected even the case of a plastic analysis. The effect of a rigid or semi-rigid joint modelling appears to have a small influence on the first buckling load factors of the structure. This effect is even smaller in the case of the Mixed solution, where the average decrease smaller than 1%.

In Figure 14, the 1st buckling mode observed in OR is illustrated. One can verify the represented mode may be defined as local as the buckling deformation only affects one column. Analysing the other models, similar buckling modes have been observed however, the buckling column depends on the model and load case combination. The fact that the presented Eigenvalues represent local modes, does not allow a clear conclusion whether the second order effects affect significantly the analysed structural solutions. Due to complexity of the 3D models, when calculating more buckling Eigenvalues, it becomes difficult to identify easily the local and global modes. Thus, better conclusions may be taken if a 2nd order linear analysis is performed. In the next section are presented and discussed the obtained results of this type of analysis.

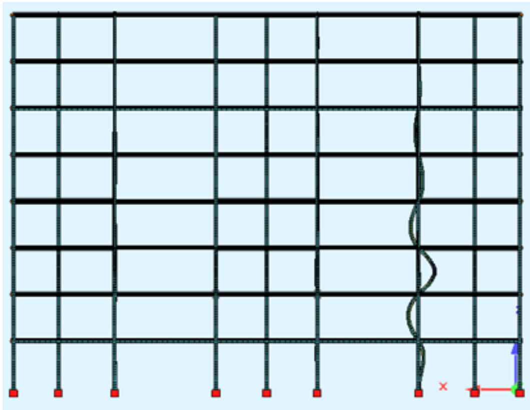


Figure 14 – Local buckling of column in OR, mode 1 of combination 2.

5.7. Second order linear elastic analysis

In Table 13 are presented the average top displacement of the structure obtained in the different models for combination 4 (dx) and combination 5 (dy) for the two types of analysis. The increase of displacement from a 1st order analysis to a 2nd order analysis has been included. In what concerns the Original model, it is clear that the 2nd order effects have influence in the response of the structure and should not be neglected increasing displacements more than 50%. In what respects to the Mixed model, this comparison shows that, although the low buckling eigenvalues presented in the previous section, the second order effects have an insignificant influence in the structural response.
From these results, it is possible to conclude that, in the case of the Original model, global modes have low Eigenvalues. As consequence, these global modes imply that 2nd order effects have great influence in the deformation of the structural solution. On the other side, the global modes for the Mixed solution should be much higher than the values presented in the previous section which showed to be local.

Model	Combination 4 (d_x)			Combination 5 (d_y)		
	1st order	2nd order	Increase	1st order	2nd order	Increase
OR	32,63	61,52	88,5%	66,99	82,24	22,7%
OSR	40,02	64,37	60,8%	89,76	113,68	26,6%
MR	6,08	6,10	0,3%	30,30	30,48	0,5%
MSR	6,19	6,22	0,4%	36,68	36,88	0,5%

Table 13 – Comparison between 1st order and 2nd order top displacements for combination 4 (d_x) and combination 5 (d_y).

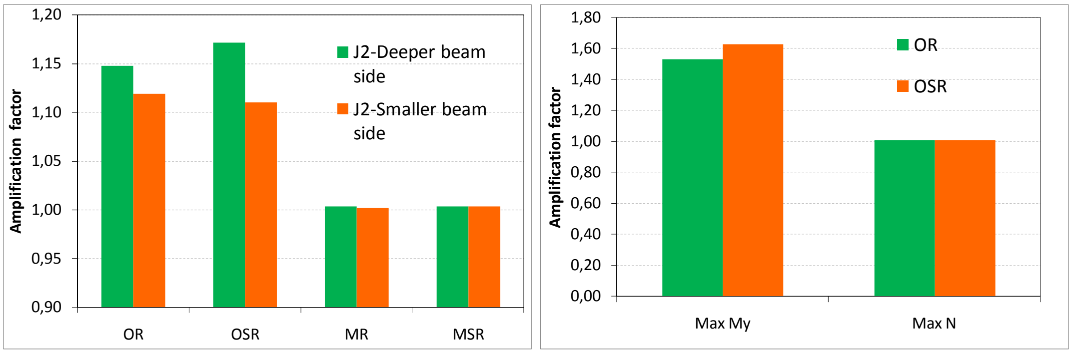
The second orders effects do not only affect the deformation of the structure but imply an amplification of the internal forces and consequently joint loads. Because joint loads are internal forces at the extremity of members, in order to be short, only joint loads are hereafter presented. In Table 14 are summarized the maximum bending moment (M_y) for joint J2 (see Figure 11) for both structural solutions. In Table 15 are the maximum column base reactions (M_y and N) for columns B2-B3-E2-E3 (see Figure 2) obtained with the 2nd order linear elastic analysis and only for the Original model. In Figure 15 are plotted the "amplification factors" ($2^{nd}/1^{st}$) calculated.

Joint side	OR	OSR	MR	MSR
Deeper beam	-444	-259	-385	-213
Smaller beam	-242	-121	-249	-121

Table 14 - Maximum bending moment (M_y – kN.m) in joint J2 for a 2^{nd} order linear elastic analysis.

		1^{st} order		2^{nd} order	
		OR	**OSR**	**OR**	**OSR**
Max My	M_y	147	112	225	182
	N	-2904	-2842	-2918	-2852
Max N	M_y	20	11	21	10
	N	-3622	-3565	-3645	-3588

Table 15 – Maximum column base reactions (M_y – kN.m and N – kN) in columns B2-B3-E2-E3 for a 2^{nd} order linear elastic analysis.



a) b)

Figure 15 – "Amplification factors" due to 2^{nd} order analysis for: a) joint loads in joint J2; b) column base reactions in columns B2-B3-E2-E3.

Again it is possible to observe that there is not significant influence of second order effects in the mixed solution. The results obtained for the Original model show that the amplification is bigger in the columns which was expected due to P-Δ effects.

6 Conclusions

The analysis of the performance of mixed steel-concrete structures was examined in this paper by means of numerical models. This type of structural solution has been compared with a common steel solution. In the presented study, the effect of the joint behaviour on the structural response has been included. Two numerical models have been developed, one considering a steel solution and the other a mixed steel-concrete option. For each model, two separate analyses have been performed: in the first, moment connections have been modelled as rigid; in the second, the semi-rigid behaviour of these joints was considered. The office building erected in

Cardington Laboratory submitted to fire tests in 1993 [6] has been used as reference structure.
The analysis performed allowed to verify a stiffer behaviour presented by the mixed solution. When compared with steel solution, this presented: lower lateral deformations to horizontal actions, as wind; greater bending moments at beam supports and smaller span bending moments; smaller beam deformations; higher buckling load factors.
In what respects the inclusion of the joint behaviour in the analysis, due to the absence of a simplified model for steel-to-concrete joints, the properties of the steel joints have been used in the Mixed model. This procedure has been taken as a first approximation. With the performed analysis, the obtained results showed the general expected response: lateral deformations increase due to reduction of global stiffness of the structure; beam deflections increase; distribution of bending moments on the members directly supported by springs is obviously affected; small reduction of the buckling load factors is observed, at least for the lower modes.
The 2nd order linear analysis performed revealed that 2nd order effects are relevant for the Original solution while for the Mixed solution an insignificant effect has been verified. This allowed concluding that the low Eigenvalues obtained for the steel-concrete solution are local buckling modes. Global modes should present much higher values.
Finally, it should be mentioned that at the stage of this research, the desired analysis is not yet completed. The inclusion of the real joint properties, for the mixed solution, should be after done.

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