< Zandonini Riccardo > (Hauptautor)
< Baldassino Nadia > (Co-Autor 1)
< Freddi Fabio > (Co-Autor 2)

<Robustness of steel-concrete flooring systems>

<An experimental assessment>

<*Robustheit von Verbunddecken*> - <Eine *experimentelle Bewertung*>. *In den letzten Jahren ist das Interesse an, durch extreme Ereignisse verursachte Schäden, gewachsen. Die Kenntnis vom Lastabtragungsmechanismus von beschädigten Strukturteilen zu den unbeschädigten Strukturteilen ist der Schlüssel zur Entwicklung von einem angepassten Bemessungskonzept. Trotz des wichtigen Beitrags der Membrandeckenwirkung zu diesem Mechanismus, ist die Forschung zu diesem Thema noch immer begrenzt. Ein europäisches Forschungsprojekt untersucht das Tragsystem von Stahl - Beton-Verbundrahmen mit Stahlbetonplatten. Zwei geometrisch unterschiedliche 3D- Versuchsrahmen werden auf die extreme Bedingung „Stützenausfall“ untersucht. Die Versuchsrahmen sind ein Unterteil in vollem Maßstab von einem 5-stöckigen Rahmen für ein Gebäude. Die Auflagerbedingungen und die Belastungsgeschichte der Versuchskörper würden durch zahlreiche numerische Untersuchungen ermittelt, um denjenigen der gesamten Rahmen zu entsprechen. Die vorliegende Arbeit zeigt diese Vorstudien und berichtet über einige experimentelle Ergebnisse aus dem ersten Versuch.*

<*Robustness of steel-concrete flooring systems*> - <An *experimental assessment*>. *Recently, the interest in the mitigation of the damages caused by accidental events increased significantly. The knowledge of loads transfer mechanisms from the damaged to the undamaged part of the structure is the key for developing adequate design strategies. Despite the potential important role of flooring systems, research of this subject is still limited. A European Research Project aims at achieving an insight into the case of steel-concrete composite frames with solid concrete slabs. Two geometrically different 3D composite sub-frames are subjected to the loss of an internal column. The sub-frames are a full-scale portion of 5 stories 3D frames. Equivalence between the sub-frames and the full-frames required a series of careful numerical analysis, which enabled definition of the boundary restraints of the specimens and of the loading sequence in the test. The present paper illustrates these preparatory studies and gives some preliminary information about the outcomes of the first test.*

Keywords: *Robustheit, Verbundrahmen, Experimentelle Untersuchung, 3D response, nachgiebige Knoten*

Robustness, Steel-concrete composite frames, Experimental analysis, 3D response, Semi-rigid joints

1 Introduction

Accidental events are characterized by a very low probability of occurrence but their effects may lead to extremely high human losses and economical consequences. An adequate design should reduce the risk in terms of human life, and at the same time minimize the extension of the collapse when individual components are damaged or even destroyed.

A number of disasters of different origin made the interest in the response of structures subjected to extreme loads such as impact or blast to continuously grow. Amongst cases with higher relevance are the collapse of the Ronan Point building (London, 1968), of the Murrah Federal Building (Oklahoma City, 1995) and of the World Trade Center (New York, 2001) [1]. Possible design strategies were set up, and incorporated in various design Specifications and Codes [2], [3], [4]. At the same time, research studies allowed to build up an increasing understanding of the structural response. Steel-concrete composite structures did not get sufficient attention. As a consequence, no practical rules are available that allow exploitation of their high bearing capacity and significant ductility. These behavioural features can provide an excellent resistance to extreme loading.

A European Research Project, aimed at developing new design concepts for steel-concrete composite frames against accidental actions, is currently in progress. The study focuses on the key aspects peculiar to the performance of this type of structures, and includes a large number of static and dynamic tests on individual members and sub-structures. The ability of the structure to transfer the loads from the damaged to the undamaged part is a central issue [5], and the study of the 3D structural system is an essential requirement in order to get an adequate insight into the problem [6]. The continuity of the members and the in plane floor action represent essential factors ensuring a robust structural response.

Two tests, different in geometry, are hence included in the project on full-scale 3D sub-frames. The main goal of these tests is to get an insight into the behaviour of steel-concrete composite frames subject to a sudden loss of an internal column. The one storey sub-frames with 2 bays in each direction are extracted from full-frame structures representative of construction practice. By simulating the total loss of a column, the experiments enable investigation of the redundancy of the floor system in terms of activation of different resistance mechanisms including slab membrane effects. Equivalence between the sub-frames and the full-frames required a series of careful numerical analyses, which enabled definition of the boundary restraints of the specimens and of the loading sequence in the test. The present paper illustrates these preparatory studies and gives some preliminary information about the outcomes of the first test.

2 The reference frames

Two five-storey steel-concrete composite structures were selected as case studies. The overall geometry of these buildings is the same. The total height is of 18m with an inter-storey height of 3.60m, and the plan dimensions are 34.2m in the longitudinal (X) direction and 11.4m in the transverse (Y) direction. The difference lies in the spans of the transverse bays: equal in length in the first case and unequal in the second one. The plan view of the first symmetric case is plotted in Fig. 1. Design was based on the relevant Eurocodes, and no seismic checks were carried out in order to decouple the issues of seismic and of robust design. In order to approximate real practice, the following was adopted: i) as to the materials, concrete C30/37, rebars grade B450C, structural steel grade S355, and bolts class 10.9; ii) a solid slab with a thickness of 150 mm; iii) full shear connection between steel beams and slab; iv) beam-column composite joints (including joints to the external columns) with flush end-plate steel connections. Rolled sections IPE 240 and HEB 220 were selected for beams and columns, respectively. Additional details of the case studies are reported in Baldassino et al. [7].

3 Design of the Sub-Frame Test

As mentioned before, the tests are performed on full-scale portions of the first floor of the corresponding full-frame. The floor-plan of the sub-frame is represented by the dotted area in Fig. 1, where the column is identified whose collapse is simulated. This column is ‘replaced’ by a hydraulic jack enabling to follow the response in different loading conditions.

The planned ‘loading’ sequence consists of three steps. In the first step, the factored vertical design load is applied onto the slab, so defining the condition before the column's collapse, in the second step the ‘central column’ is gradually ‘removed’ simulating the column collapse in consequence of an accidental action. In order to get an appraisal of the available safety margin, as a third step, a tension force is then applied at the lost column location and increased until a substantial distress of the system is observed. Figure 2 illustrates the simulated structural response of the sub-frame in the three phases.

In order to design the experimental tests, refined Finite Element Models of full-frames and sub-frames are developed by using the Abaqus program [8] and the details are reported in Baldassino et al. [7].

The sub-frame should be representative of the full-frame. Therefore, the design of a set of suitable restraints for the specimen becomes a central issue. These restraints have to enable adequate approximation of the presence of the remaining part of the structure. The numerical study preliminary to the test focused hence on this subject. In particular, two facets of the problem were investigated: the restraints at the floor level and the possible need of connecting the columns at a certain height above the floor. The sub-frame is ‘extracted’ from the ground floor of the full-frame and hence the columns have to be rigidly connected to the strong floor.

Interconnecting the column above the floor system appears to be necessary in order to account for the restraint offered to the columns by the upper storeys. Based on the frame-deformed shape, an interconnection by truss members at mid-height of the second storey is an adequate solution. This specimen’s configuration (Fig. 2) allows for approximating well the distribution of the moments in the columns.

How to restrain the floor system to the laboratory counter-walls was a far more delicate issue. Various solutions were considered and their adequacy checked. Among them, the possibility of restraining only the steel frame (and not the slab) and the complexity of the restraining system were investigated. As to the second factor, the greatest simplicity possible was dictated by restraint’s design and fabrication, and by the need for measuring the reactions accurately. The study took advantage of the presence of the bracings in the full-frame, which prevents any significant displacement along the frames in the planes A and C (Fig. 1): the relevant d.o.f. along X has to be fully restrained in the sub-frame. The analyses led to the restraining system in Fig. 3, where at B the X d.o.f. is left free (central beam in Fig. 3), while the vertical and lateral displacements (along Y and Z) are prevented. All the restraints are made up by truss elements connected to the steel frame. A further truss member connecting the specimen to the counter-wall was located at L (Fig. 3) in order to ensure stability of the sub-frame when it enters the large displacement field. Its influence on the response is modest.

The details and outcomes of these preliminary analyses are reported in Baldassino et al. [7]. Figure 4 shows the comparison for the vertical displacements and bending moments in the slab along the section S-S of the sub-frame (Fig. 3). The dotted lines refer to the sub-frame while the solid line to the full-frame. The responses are reported for the end of each of the three steps; step 3 is considered to range to an increase of the load on the slab equivalent to 30%. It is possible to observe that the behaviour of the full-frame is well approximated by the sub-frame. Similar results were obtained also for other sections and for other quantities (i.e., shear force, axial force, etc.). Moreover, the von Mises stresses for the bottom and top side of the slab were compared: Figure 5 refers to the end of the second step.

The adequacy of applying in the third phase a concentrated tension load with the aim of appraising the extent of the safety margin was a further point investigated. This loading option was compared with the increase of the distributed load on the floor. The results showed that while the load on the slab is increased up to 30%, the proposed solution approximates more than satisfactorily the behaviour of the full-frame in term of displacements and internal forces. A more detailed coverage of this point is given in Zandonini et al. [9].

4 The first test

The experimental activities started with the construction of the first specimen (symmetric case) inside the Laboratory of Materials and Structures Testing of the University of Trento. The steel skeleton was first erected (Fig. 6a) and connected to the strong floor and to the counter-walls by means of restraining systems detailed so as to reproduce the boundary conditions identified by means of the numerical analyses (Fig. 3). The slab was then cast (Fig. 6b). During these constructional phases the hydraulic jack simulating the central column was held in place in a non operating state. The central beams were hence held in position by means of props (Fig. 6) which were removed just before the beginning of the test. After formwork's removal the specimen measuring instruments were installed.

The considerable amount of parameters affecting the response required an accurate selection of the quantities to be measured during the test. The attention was mainly focused on the response of columns, beams and beam-to-column joints.

Columns B1, C1, D1, F1 and H1 (Fig. 7) were instrumented at two sections, located at a distance of 150mm and 3200mm from the base, respectively, to follow the evolution of the stress state. On each section, strain gauges were applied allowing measuring both the axial and the bending deformation in the X-X and Y-Y planes (Fig. 7). Strain gauges for the evaluation of the axial strain were also installed on the beams AB1, BC1, DE1, EF1, BE1 and EH1 approximately at mid-span. Moreover, beams DE1, EF1, BE1 and EH1 were instrumented with strain gauges at approximately 600mm from the centre of the column to appraise the shear load transfer at the beam-slab interface. At these sections strain gauges were applied also on slab rebars. Finally, the reactions at the restraints and the forces in the crowning members were also monitored.

Displacement transducers and inclinometers allowed measuring the rotations of the columns at the beams level and of the beam-to-column joints. The attention was focused on the response of the central joint (position E1) and of the joints between the beams connected to the central joint and columns D1, B1, F1 and H1. The instruments were positioned so as to decouple the joint and connection response. Additional transducers were also employed to measure the torsional rotations of the external beams, and the rotation of the column C1 and the horizontal displacement of the column H1 at the beam level. Furthermore, the vertical displacements at the central joint and at the centre of the four slab panels were measured

All the instruments' signals, including the load cell connected to the hydraulic jack, were acquired with a frequency of 2Hz.

The test comprises the following phases:

1- "Activation" of the hydraulic jack and removal of the props. The load measured by the loading cell was of 228kN and represents the self-weight portion of the specimen sustained by the central column;
2- Application of the vertical load to the slab. At this aim bags filled with sand were placed on the slab surface reproducing a uniform distributed load of 8,8kN/m^2 so as to approximate the factored design load including finishes, partitions and variable loads. At the end of this phase the load applied to the central column was of 668kN;
3- Gradual removal of the column simulated by reducing the pressure of the hydraulic jack down to zero. The vertical displacement of the central joint was of about 157mm;
4- Stabilization of the specimen with the vertical displacement of the central joint increasing to 163mm.
5- Application of a tensile force increasing up to the end of the test. The final tension load was of 300,9kN and the vertical displacement was 305mm.

Figure 8 illustrates the deformation of the specimen at the end of the test (Fig. 8a) and the load-central joint vertical displacement relation (Fig. 8b).

At a load of 286,8kN in tension the collapse of a bolt of the bottom row of the connection between the central column and the beam EH1 takes place (Point A in Fig. 8b). The test continued up to a load of 300,9kN when the second bolt in the same row fractured (point B in Fig. 8b). The remarkable plastic deformation of the connections and the state of 'distress' of the concrete at the central joint (Fig. 9a) suggested to stop the test.

The visual inspection of the specimen allowed also identification of significant deformations of the external columns H1, F1, D1 and B1, mainly concentrated at the beam-to-column joint (Fig. 9b). A concentrated ‘rotation' of the column reveals the plastic shear deformation of the column B1 web panel. Furthermore, the mechanism of force transmission between column and beam induces compression at the beam lower flange with associated instability phenomena (Fig. 9b). Horizontal cracks developed in the slab thickness on the outer side of the slab at columns F1 and D1 associated with the transmission of shear forces between concrete slab and column.

As to the crack pattern, the evolution observed at the bottom side of the slab confirmed the stress distribution evaluated with numerical analysis (Fig. 5).

The test has just been performed and no detailed conclusion can be drawn. However, it pointed out the potential robustness ensured by the flooring system. The ductility of the beam-to-column joints are confirmed to play an important role.

5 Conclusions

The paper highlights the first phase of a study of the robustness of steel-concrete composite frames. The project includes the experimental investigation of two 3D frames with 2 bays by 2 bays and one story, extracted from full-frame structures chosen as case studies. During the test the specimen is first subjected to the design vertical load and then the ‘central column’ of the frame is ‘removed’ simulating its collapse in consequence of an accidental action. Finally, the vertical load is further increased to appraise the residual margin of safety of the system. The preparatory numerical analyses and the preliminary outcomes from the first test are reported. The potential in terms of resistance to progressive collapse offered by the 3D floor system is confirmed.

6 Acknowledgements

The research is made possible by the European Grant RFSR-CT-2012-00029, which the Authors gratefully acknowledge.

References

[1] *Brett, C., Lu, Y.:* Assessment of robustness of structures: Current state of research. Front. Struct. Civ. Eng. (2013), 7(4), pp. 356-368.

[2] *European Committee for Standardization*: EN 1991-1-7:2006, Eurocode 1: Actions on structures – Part1-7: General actions – Accidental actions. Brussels (Belgium), CEN 2006.

[3] *British Standards Institution*: BS 5950: Structural use of steelwork in buildings, Part 1: Code of practice for design – Rolled and welded sections. London (UK), BS 2001.

[4] *American Society of Civil Engineers*: ASCE Standard No. 7-10: Minimum design loads for buildings and other structures. Reston (VA), ASCE 2010.

[5] *Starossek, U.*: Progressive collapse of structures: Nomenclature and procedures. Structural Engineering International: Journal of the International Association for Bridge and Structural Engineering (2006), 16 (2), pp. 113-117.

[6] *Kuhlmann, U., Roelle, L., Izzuddin, B.A., Pereira, M.F.*: Resistance and Response of Steel and Steel-Concrete Composite Structures in Progressive Collapse Assessment. Structural Engineering International (2012), Vol. 22, pp. 86-92.

[7] *Baldassino, N., Freddi, F., Zandonini, R.*: Robustness of moment resisting steel-concrete composite frames: the floor resisting mechanism in the case of column collapse. Proceeding of XXIV Giornate Italiane delle Costruzioni in Acciaio (CTA), 2013, Turin, Italy, pp. 129-136.

[8] ABAQUS – Analysis user's manual, version 6.10, Simulia Dassault Systems, 2010.

[9] *Zandonini, R., Baldassino, N., Freddi, F.*: Robustness of flooring systems in 3-D frames. An experimental assessment. Proceeding of 7th European Conference on Steel and Composite Structures (EUROSTEEL), 2014, Naples, Italy, (in print).

Autoren dieses Beitrages:

Prof. Dr.-Ing. Riccardo Zandonini

via Mesiano 77

38123 Trento (Italy)

Email: riccardo.zandonini@unitn.it

Dr.-Ing. Nadia Baldassino

via Mesiano 77

38123 Trento (Italy)

Email: nadia.baldassino@unitn.it

Dr.-Ing. Fabio Freddi

via Mesiano 77

38123 Trento (Italy)

Email: fabio.freddi@unitn.it

CAPTIONS

Bild 1. Grundriss Deckenkonzept vom Versuchsfall – symmetrische Konfiguration (Abmessungen in mm)

Fig. 1. Floor-framing plan of case study structure - Symmetric configuration (dimensions in mm)

Bild 2. Strukturelle Antwort vom Versuchsrahmen unter den folgenden Lastkombination: a) Vertikallasten mit Teilsicherheitsbeiwert; b) Mittenstützenausfall; c) Mittenstütze unter Zugkraft

Fig. 2. Sequence of response for the sub-frame structure under the following loading steps: a) Factored vertical load; b) Removal of column; c) Tension load applied at the central column location

Bild 3. Auflagerbedingungen vom Versuchsrahmen

Fig. 3. Restraining conditions for the sub-frame

Bild 4. Vergleich von Durchbiegungen und Biegemomente im Querschnitt S-S

Fig. 4. Comparison of the Vertical displacements and Bending moments on the Section S-S

Bild 5. Vergleich von von Mises Spannungen in der Decke

Fig. 5. Comparison of the von Mises Stresses in the slab

Bild 6. a) Stahlstruktur b) Betonieren der Decke

Fig. 6. a) The steel skeleton; b) The casting of the slab

Bild 7. Grundriss Stahlrahmen

Fig. 7. Layout of the steel frame

Bild 8. a) Versuchskörper am Versuchende; b) Last-Verschiebungskurve vom Mittenstütze

Fig. 8. a) Specimen at the end of the test; b) Load-displacement curve of the central column

Bild 9. Anschlussverformung am Versuchende: a) zentrale Knoten b) Knoten am Stützen H1

Fig. 9. Joint deformation at the end of the test: a) the central joint; b) the joint at the column H1