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Experimental behaviour of an innovative demountable balcony with sophisticated connection

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Abstract

For a general practice, reinforced concrete balconies are permanently fixed and simultaneously erected onsite with the rest of the structures in the buildings. This practice is problematic in the viewpoint of the onsite security, the speed of construction, and the maintenance of the balcony and the building. This problem calls for an innovative solution of a demountable balcony that has been conceptualized by INGENOVA, an R&D office of a French company GROUPE LEGENDRE. In the proposed system, the balcony is supported by H-profile cantilevers that are connected to the embedded H-profiles in the reinforced concrete slab by bolted joints. The design of the connection of the proposed balcony to the concrete slab, particularly the embedded H-profiles, requires an experimental characterization. This paper presents an experimental investigation of the behaviour of the innovative balcony system through a series of experimental tests on large-scale specimens. The main objectives of these tests are to quantify the local behaviour (distribution of concrete pressures onto embedded H-profiles) as well as the global behaviour (the bearing capacity as well as the deformability). The test results show that the failure mode of the specimen is governed by a punching failure of the concrete or by the rupture of steel bolts in the joint, following the configurations. Furthermore, the distribution of the concrete pressures on the embedded profiles obtained from the tests provides reference results for future development of the analytical design.

Keywords: *Demountable steel balcony, Embedded H-profile anchorage system, Hybrid bolted joints, Concrete contact pressure.*

1. Introduction

Balcony is an essential architectural element that provides more comfort to the people living in an apartment building. The balcony may experience damages due to constantly being exposed to the weather and therefore needs regular maintenance. Typical reinforced concrete balconies are usually erected onsite simultaneously with other structural elements and permanently fixed during the whole life of buildings. This practice on one hand slows down the construction process and on the other hand causes difficulties for the maintenance of the balcony and the building.

INGENOVA, an R&D office of a French company GROUPE LEGENDRE, in collaboration with INSA Rennes has conceptualized an innovative steel balcony that is supported by H-profile cantilevers connected to an anchorage system by hybrid bolted joints (Figure 1). The anchorage system is composed of reinforcement rebars and a steel module that contains H-profiles welded to an edge beam at the exterior end and to a steel tube at the other end. The detail of bolted joint is given in Figure 1c. This balcony system is therefore demountable, allowing an accelerated construction of concrete floors and walls as the balcony can be installed at the end of the construction phase, as well as providing a

convenient accessibility for maintenance. In addition, the anchorage system of the balcony is simpler and hidden in the slab, making the structure more aesthetic compared to that of typical steel balconies. However, the load transfer mechanism between the H-profile cantilevers and the concrete slab through the bolted joints is not straightforward. The pressure distribution of the embedded profiles onto the concrete is not fully known.

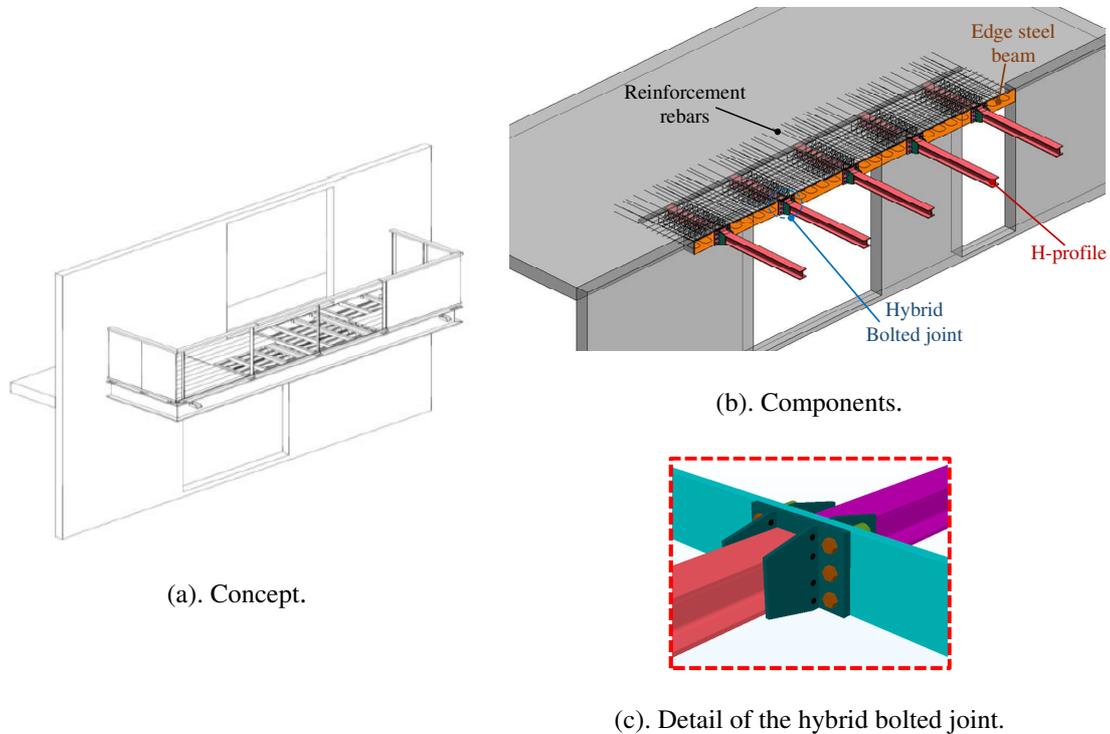


Figure 1. System of the innovative demountable steel balcony.

The investigation of the concrete pressure on embedded profiles can be referred to the study by Mattock and Gaafar (1982) who performed an analytical and experimental study of the strength of steel sections embedded in a reinforced concrete column as brackets. They made a simplification of the compressive stresses on the concrete below and above the embedded sections with an experimental validation in order to propose the analytical model for the design. On the other hand, Corley and Hawkins (1968) performed experimental tests of concentrically loaded slab-column specimens containing shearhead reinforcement made from structural shapes. A design procedure for shearheads at interior supports was then proposed based on the experimental results. However, these studies do not present the same situation as in the case of the embedded profiles in this study. Therefore, the design of the connection of the proposed balcony to the concrete slab, particularly the embedded H-profiles, requires an experimental characterization.

The objective of the present paper is to describe an experimental study on a series of large-scale specimens of the balcony system in order to quantify the local behaviour (distribution of concrete pressures onto embedded H-profiles) as well as the global behaviour (the bearing capacity as well as the deformability).

2. Experimental testing

2.1. Test setup

The test setup was designed to represent a concrete floor-to-balcony connection (Figure 2a). In order to have a simple test setup and a representation of the in-situ moment diagram (Figure 2b), the floor was simply supported at the theoretical inflection point while the load was applied at the free extremity of the cantilever (Figure 2c).

The test setup consisted of a specimen of the balcony, a framed back support, two front supporting columns, two lateral bracings, a force jack with a capacity of 1500 kN, and a loading cross-beam to distribute the load from the jack onto the cantilever beams (Figure 3). In this test setup, the specimen was first placed on the two front supporting columns with a contact surface of 300mm-by-160mm and on the mobile UPN200 cross-beam of the framed back support (see detail B in Figure 3b). The specimen was then put into contact with the HEA200 cross-beam of the framed back support by adjusting the level of the UPN200 cross-beam. Two lateral bracings were also installed in order to limit the lateral buckling of the profiles. Two layers of PTFE material were installed between the lateral bracings and the cantilever steel H-profiles to reduce friction. The loading was applied vertically by the force jack on the two cantilever profiles through the loading cross-beam.

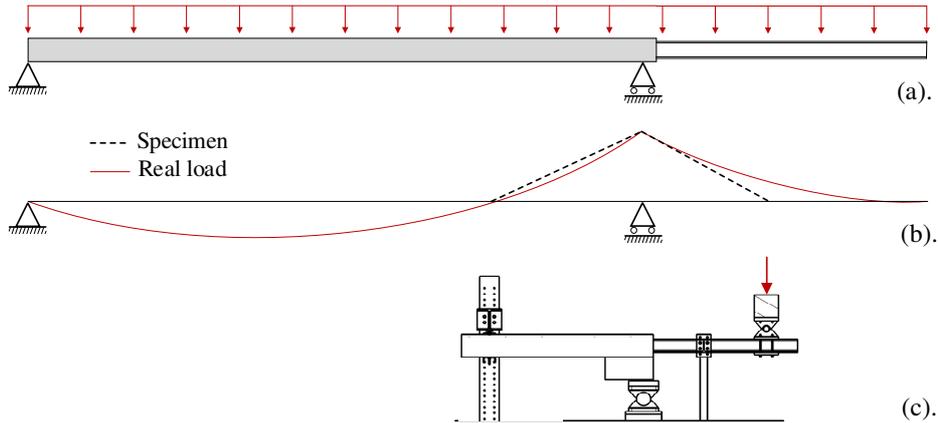
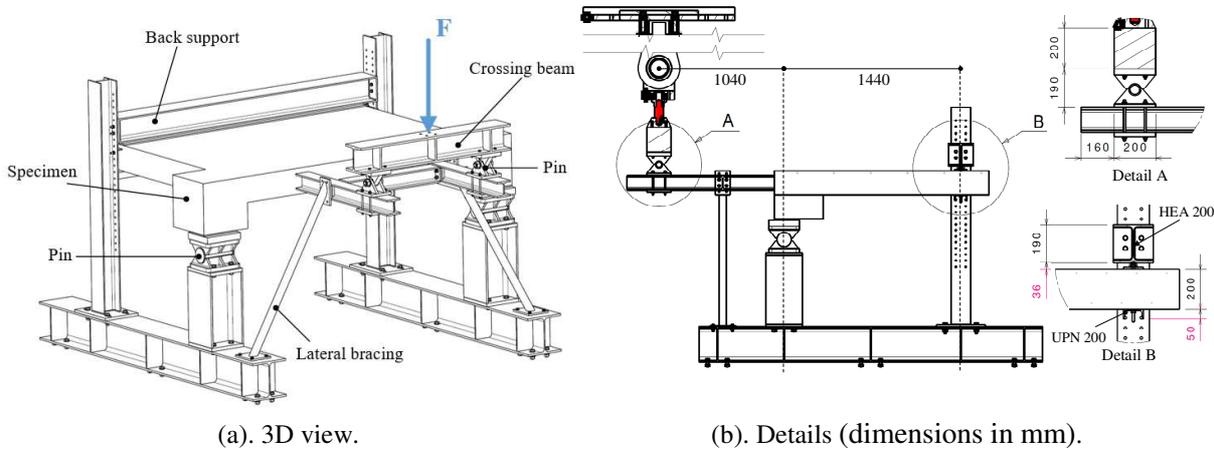


Figure 2. Specimen design: (a). Configuration of real loads on floor-to-balcony connection. (b). bending moment diagrams. (C). Configuration of testing.



(c). Photo.

Figure 3. Test setup.

2.2. Test specimen

In order to quantify the contribution of the joint, the edge steel beam and the lintel drop, three specimens were fabricated and tested. The first specimen (C1-A) consisted of two HEB120 profiles welded to a steel tube with a section of 50×50×3 mm and encased in a reinforced concrete slab (see Figure 4a). The specimen C1-B had the actual configuration of the balcony concept (Figure 4b). It was composed of two embedded HEB120 profiles, welded to a steel tube with a dimension of 50×50×3 mm at one end and to a steel edge beam at the other end, two HEB200 cantilever beams (each welded to an end plate with a dimension of 290×200×20 mm), bolted joints connecting the cantilevers to the embedded steel part, and a reinforced concrete slab with a thickness of 200 mm. The joint was composed of 6 M20 bolts with a grade of 8.8 fixing through the steel edge beam to 6 embedded threaded sleeves, which are welded to the edge beam (Figure 4d). The last specimen, called C1-C, had the same configuration as specimen C1-B except for the presence of the RC lintel drop with a height of 200 mm (Figure 4c).

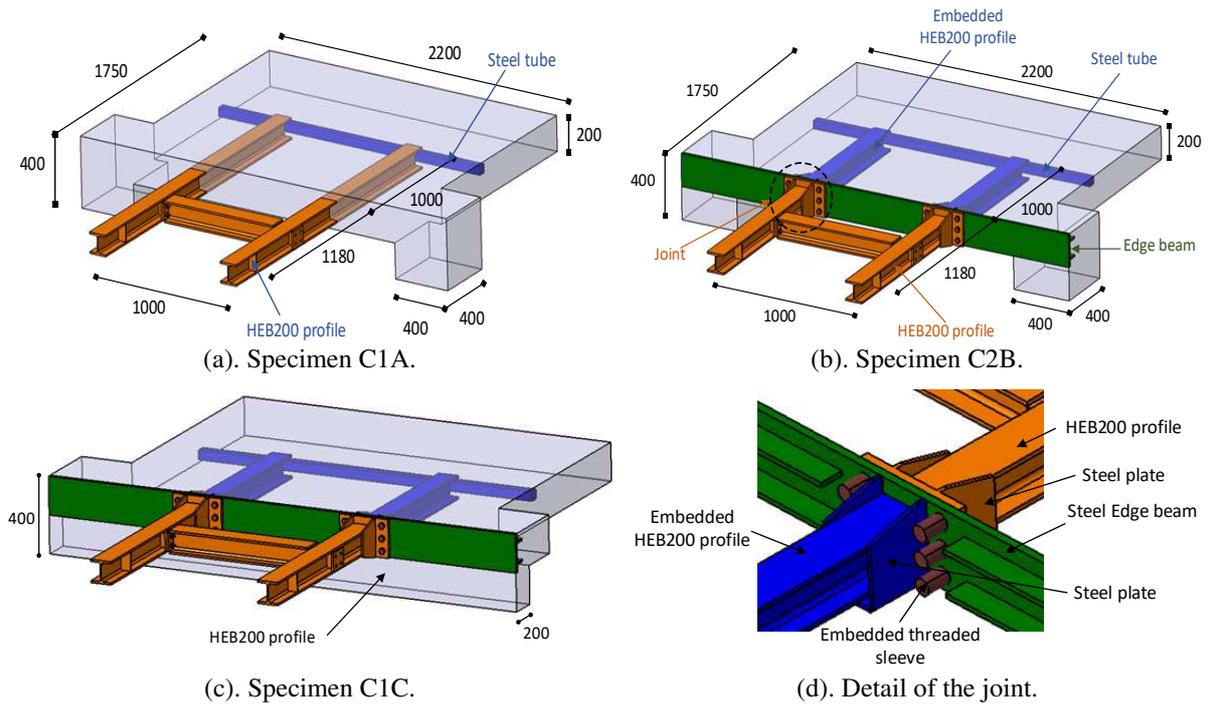


Figure 4. Test specimens (dimensions in mm).

2.3. Material properties

The concrete grade was C25/30. The compressive strength of concrete was determined according to NF EN 12390-3. Three 110 x 220 mm cylinder specimens were tested at around 28 days of age and three others at the time of testing. The results are given in Table 1, in which f_{cm} and $\sigma_{f_{cm}}$ are the mean value of compressive strength of the concrete and the corresponding standard deviation, respectively.

Table 1. Compressive strength of the concrete.

Specimen	Test date	Age (days)	f_{cm} [MPa]	$\sigma_{f_{cm}}$ [MPa]
C1-A	16/01/2020	30	28.8	1.2
	26/02/2020	71	36.9	1.0
C1-B	20/01/2020	31	30.7	1.9
	10/03/2020	81	41.7	1.0
C1-C	02/03/2020	28	34.5	0.3
	11/06/2020	129	42.6	1.2

For steel materials such as the HEB200 profile and the rebars, at least three coupon tests were made according to EN ISO 6892-1 in order to determine the actual characteristics, the results of which are

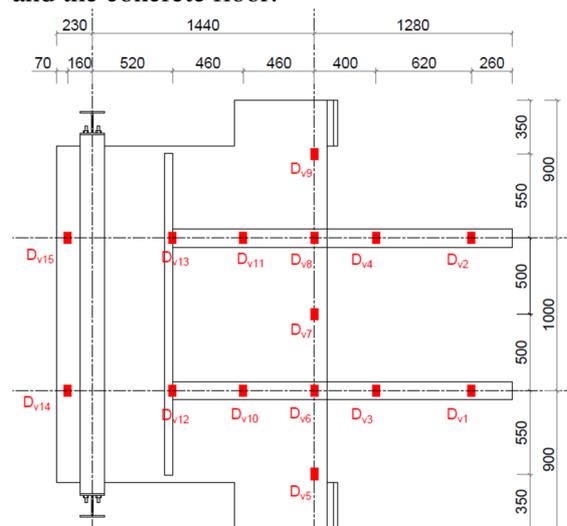
reported in Table 2. f_{ym} , $\sigma_{f_{ym}}$, f_{um} , and $\sigma_{f_{um}}$ are the mean value of yield strength, the standard deviation of yield strength, the mean value of ultimate strength, the standard deviation of ultimate strength, respectively.

Table 2. Properties of steel materials.

Element		Number	Yield strength		Ultimate strength	
			f_{ym} (MPa)	$\sigma_{f_{ym}}$ (MPa)	f_{um} (MPa)	$\sigma_{f_{um}}$ (MPa)
HEB 120	Flange	6	395.5	7.7	515.1	7.0
	Web	3	386.9	16.3	509.9	6.7
Rebar	HA8	3	467.0	31.1	621.2	14.8
	HA10	3	457.9	72.1	550.1	14.3
	HA12	3	513.5	26.8	598.9	19.7

2.4. Instrumentation

The force generated by the hydraulic jack was measured with 3 force sensors: a DELTECH ± 200 kN and a double scale sensor ± 500 kN and ± 1500 kN. 4 LVDTs were used to measure the vertical displacements of two steel profiles (Dv1 to Dv4) and 11 LVDTs were installed to measure the vertical displacements of the reinforced concrete slab (Dv5 to Dv15). The position of the LVDTs are shown in Figure 5. Measured data were recorded every half second by a data acquisition system. Three extra LVDTs were also added in tests C1B and C1C in order to measure the opening between the edge beam and the concrete floor.



(a). Positions of the sensors.



(b). Photo of the sensors.

Figure 5. Displacement sensors (dimensions in mm).

Apart from the displacement sensors, strain gauges were used to measure the deformation of the embedded HEB200 profiles at the locations of 30 mm, 80 mm, 180 mm, 280 mm and 380 mm from the front side of the concrete slab (see Figure 6). For specimen C1A, 40 gauges were used in total, meaning 20 gauges for each H-profile. For specimens C1B and C1C, strain gauges on the bottom flanges at the first two sections could not be installed due to the existence of the lateral stiffeners. Consequently, only 32 gauges were used, 16 gauges for each H-profile.

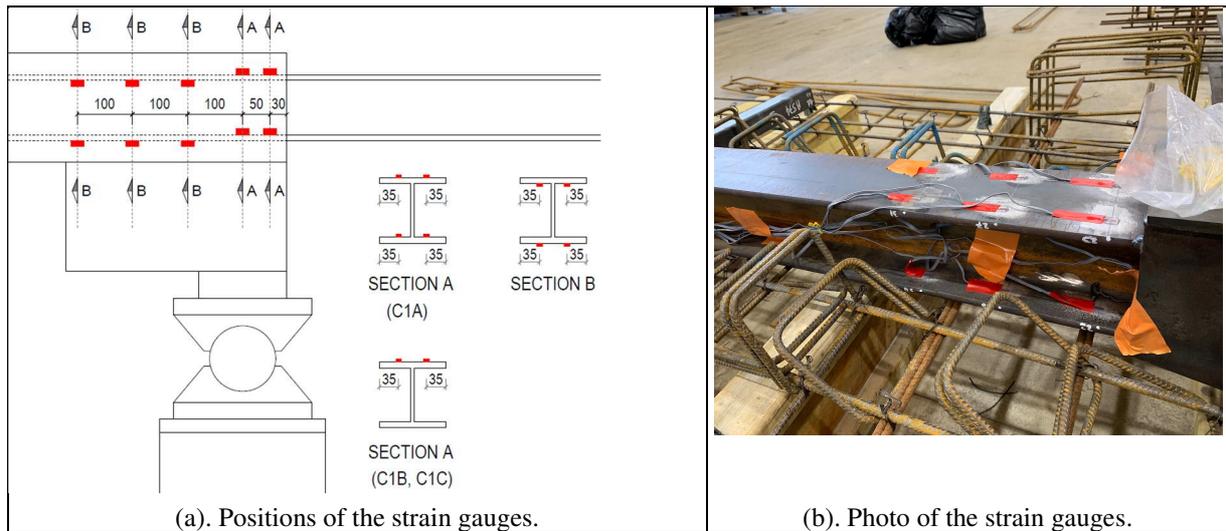


Figure 6. Strain gauges (dimensions in mm).

2.5. Test procedure

The load was applied according to the test procedure described in Annex B of Eurocode 4 part 1-1 (2004). Each test specimen was subjected to 25 loading/unloading cycles between 40 % and 5 % of the expected failure load, followed by a cycle in which the load was increased to the estimated ultimate value before being loaded to failure (see Table 3). The load was applied at a constant rate of 3mm/min during the first 3 cycles and was maintained afterwards at 6mm/min.

Table 3. Expected value of ultimate and failure loads.

Test	Ultimate [kN]	Failure [kN]
C1A	50	65
C1B	85	130
C1C	160	210

3. Experimental results

3.1. Failure mode

Figure 7 shows the evolution of the force in function of the elongation of the force jack for the three tests. With respect to the specimen C1A, cracks started to appear during the initial 25 cycles of loading (5 kN – 25 kN) on the bottom surface of the concrete slab, right under the H-profiles (Figure 8a). These cracks then propagated towards the back of the slab with the increase in the load applied. During the cycle between 5 kN and 50 kN, inclined cracks appeared on both sides of each profile (Figure 8b and Figure 9), corresponding to a punching shear phenomena. Inclined cracks were also noticed on the lateral sides and on the bottom surface of the slab (Figure 10). Cracks due to bending behaviour of the slab were only seen at a higher applied load, close to the failure one (Figure 11). The failure happened when the applied load stopped increasing at around 125 kN and excessive opening of inclined cracks on the lateral sides were observed. The specimen failed in punching shear at the supports.

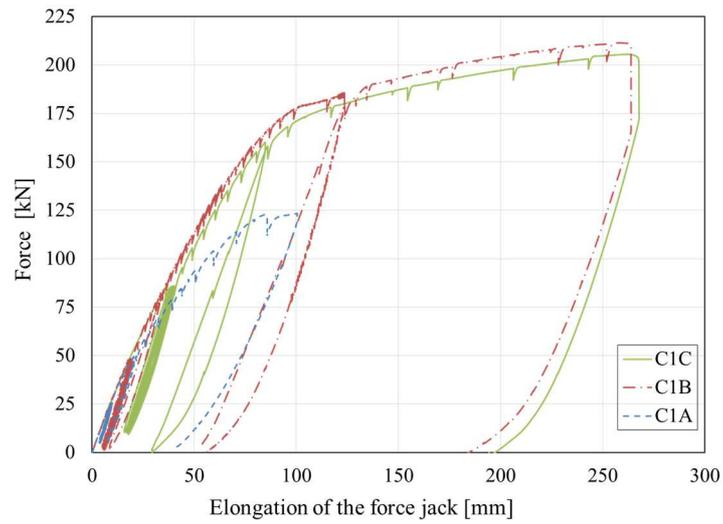


Figure 7. Force-displacement curve at the force jack.

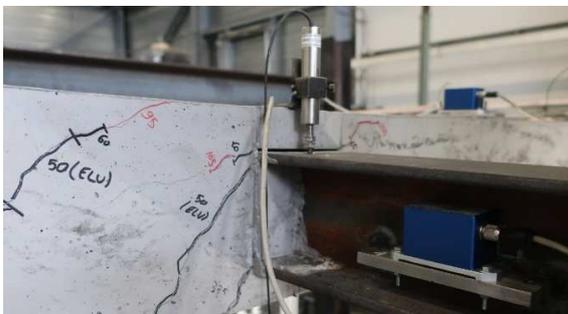


(a). During the 25 cycles between 5 and 25 kN.

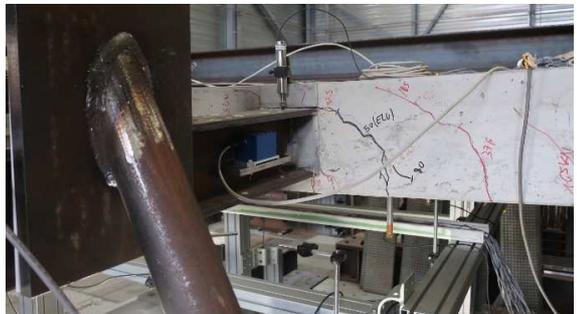


(b). At estimated ultimate load.

Figure 8. Test C1A: cracks on the bottom surface of the reinforced concrete slab.

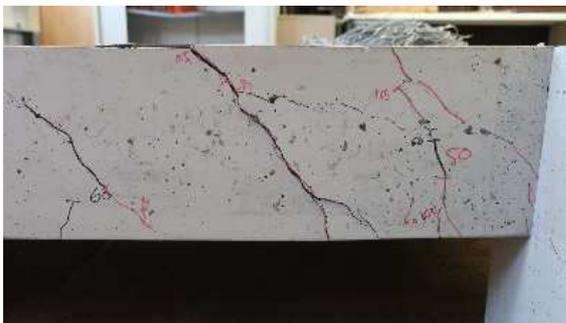


(a). Left profile.



(b). Right profile.

Figure 9. Test C1A: inclined cracks on the front surface of the reinforced concrete slab.



(a).



(b).

Figure 10. Test C1A: inclined cracks on lateral and on the bottom surfaces of the reinforced concrete slab.



Figure 11. Test C1A: Vertical cracks at the middle span of the slab.

For tests C1B and C1C, the punching shear phenomena were also observed at the supports of the concrete slab but with lower intensity (Figure 12). In addition, cracks also developed along the edge of the slab, causing the separation between the concrete slab and the steel edge beam (Figure 13). In the case of specimen C1C where a reinforced concrete lintel drop is present, vertical cracks were seen on both exterior and interior surfaces of the lintel since the first loading cycle (Figure 14).



(a). Bottom surface.



(b). Lateral surface.

Figure 12. Test C1B: cracks due to punching shear.



(a). Test C1B.



(b). Test C1C.

Figure 13. Crack due to the separation between the concrete slab and the steel edge beam.

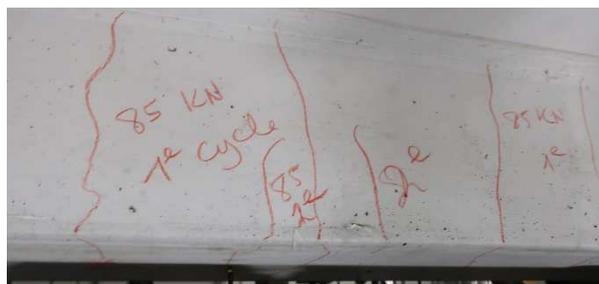


Figure 14. Test C1C: vertical cracks on the concrete lintel.

In both tests C1B and C1C, the failure occurred at the bolted joints (Figure 15). In test C1B, a bolt on the right side of the top row of the joint at the right profile was broken when the load reached approximately 210 kN, which prompted a sudden drop of load to around 165 kN. In test C1C, two bolts of the top row of the joint at the right profile were broken simultaneously at a load of 205 kN.

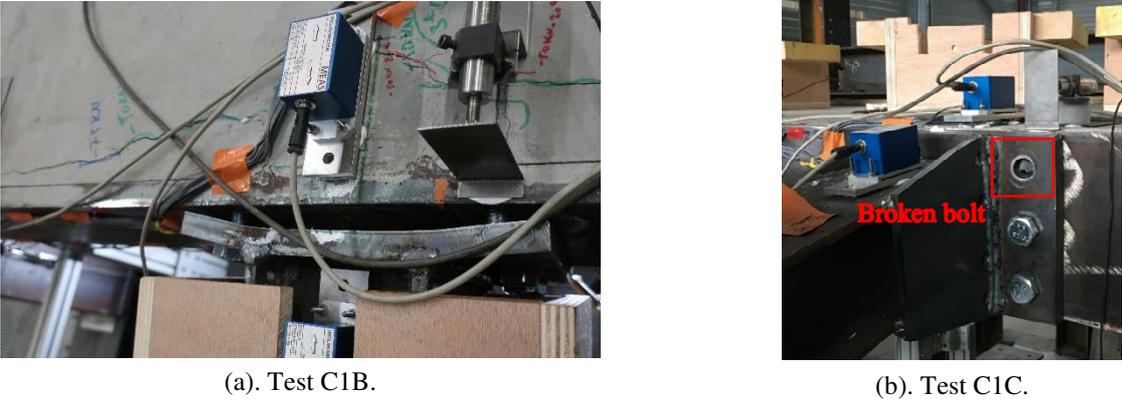


Figure 15. Test C1B: deformation of the gusset and position of the broken bolt.

3.2. Deflection of the HEB200 profile

The deflection of the cantilever H-profiles at the applied load location in function of the applied force per one profile in the three tests is shown in Figure 16. In case of test C1B, the measured displacement data provided by the LVDTs are only reliable up to 92 kN. The experimental setup was unable to measure the system response for higher values. In fact, the maximal forces resisted by specimens C1B and C1C are approximately equal to 104 kN and 102 kN respectively, 65% higher than the load supported by specimen C1A (62 kN). It is clear that the presence of the steel edge beam increases significantly the bearing capacity and the stiffness of the structure.

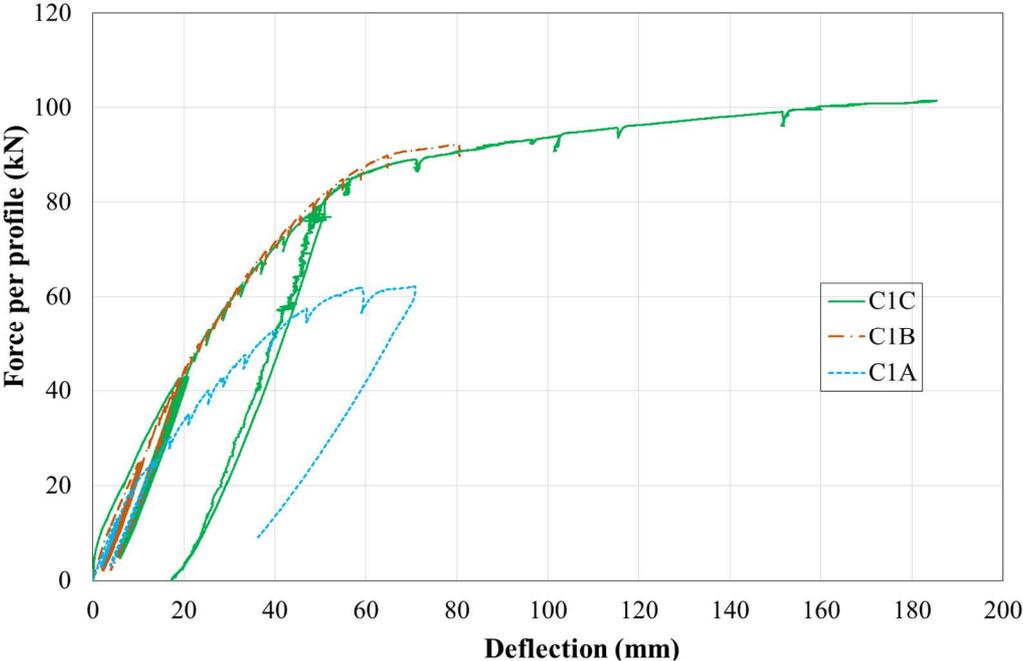


Figure 16. Deflection of the HEB120 profile at the applied load in function of force per profile.

3.3. Distribution of bending moment along the embedded H-profile

The bending moment in the embedded H-profiles can be determined from the strain values measured with strain gauges placed on their surfaces. By assuming that the strain varies linearly along the height of the section, it is possible to determine the strain distribution over the cross section of the profile

(Figure 17). Within the elastic domain, the normal stress σ is calculated from the strain using Hooke's law. Beyond the elastic limit, the normal stress is determined based on the stress-strain relationship obtained from tensile tests. The bending moment M acting on the section A can be computed as:

$$M = \int -y\sigma dA \quad (1)$$

where y is the distance from the neutral axis.

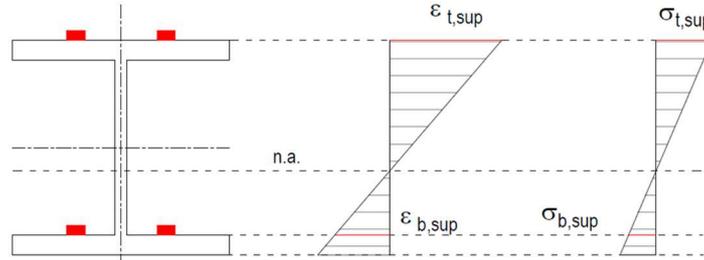


Figure 17. Strain and stress distribution over the height of the profile section within the elastic domain.

Figure 18 represents the comparison of the bending moment acting in the H-profiles for the three tests. The bending moment at the first two sections are missing in tests C1B and C1C due to the lack of data on the strain at the bottom flange of the profiles. It can be noted that the bending moment in the embedded H-profiles in test C1A is about two times larger than in test C1B. On the other hand, the bending moment obtained in test C1C is about 30% to 50% higher than in test C1B.

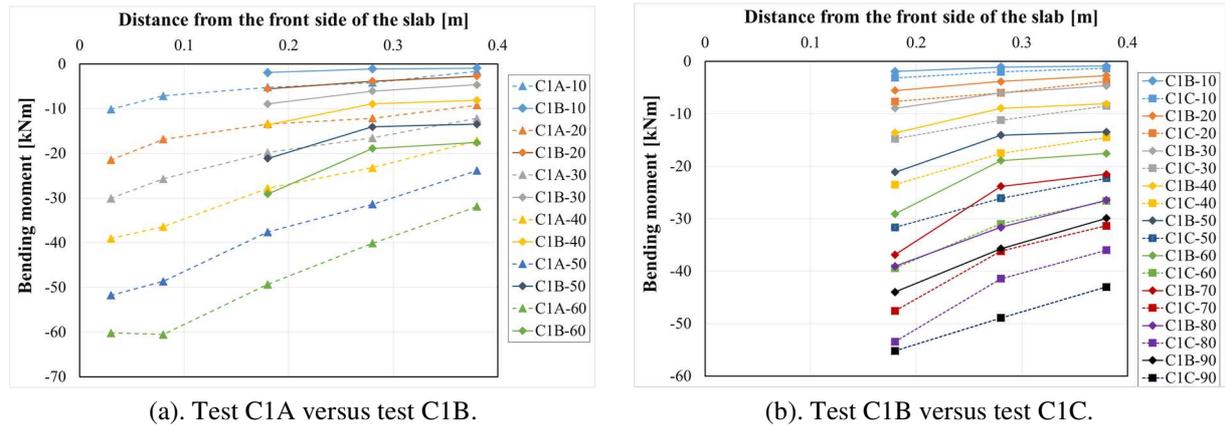


Figure 18. Distribution of bending moment in the embedded H-profile.

4. Conclusion

A series of three experimental tests on large-scale specimens was carried out in order to study the behaviour of a novel demountable steel balcony system. The test results show that the failure mode of the specimens is governed by a punching failure of the concrete in the case of test C1A and by the rupture of steel bolts in the joint in tests C1B and C1C. The presence of the steel edge beam in test C1B seems to improve the load transfer from the balcony to the reinforced concrete slab, as the applied bending moment is lower compared to test C1A for the same level of loading. On the other hand, the lintel drop in test C1C seems to provide a vertical reaction, which in turn increases the negative bending moment in the embedded profiles.

These tests could also quantify the bearing capacity and the deformability of the systems, providing reference results for the improvement of the design approach. The next step involves the development of a numerical modelling of the tests in order to gain a deeper understanding of the system behaviour. This model can be used in a parametric study of important parameters, the result of which will be exploited to develop a design approach for the embedded profile.

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