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Mechanical and fire tests on thermal breaks attached to concrete supports

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ABSTRACT

This paper deals with the mechanical and fire behaviour of simple solutions of thermal breaks for external steel structures (balconies, passageways, solar panels ...) attached on a concrete facade with external thermal insulation. The proposed solutions are composed of a PVC or plywood layer implemented between an extended end-plate connection and the concrete support (lintel, floor slab, concrete wall). A total of 12 monotonic mechanical tests have been performed on steel-to-concrete connections using intermediate insulation layers (PVC or plywood) in order to investigate the rotational stiffness, plastic/ultimate bending moments, the rotation capacity as well as the failure mode. Two types of fastening were used; bolted embedded end-plates and post-installed mechanical/chemical fasteners. In order to evaluate the influence of the studied solutions of thermal breaks on the behaviour of a balcony under fire, two external flame experimental tests were carried out on realistic balconies attached to a concrete support. These fire tests demonstrated that the implementation of the proposed thermal breaks between end-plates and the support does not affect the stability of the balconies subjected to fire.

Keywords: *Buildings; attached steel structure; thermal breaks; concrete support; mechanical tests; fire tests.*

31 **1. Introduction**

32 For the building industry, one of the biggest challenges right now is to reduce the impact of
33 buildings on the environment by reducing overall energy demand and CO₂ emissions. The
34 construction of Nearly Zero-Energy Buildings (NZEB) becomes a vital necessity [1]. NZEB
35 requires the improvement of the energy efficiency of buildings, and thus reducing heat losses
36 from envelope. Although improving thermal performances of the envelope components seems
37 to be easy to reach, the problem of singular points and interfaces between materials need to be
38 solved. In most cases, the singular points need to be analysed considering the interaction
39 between various normative requirements. The case of thermal breaks that must meet thermal,
40 mechanical and often fire performances requirements is a good illustration.

41 Thermal breaks are systems used to reduce heat loss through thermal bridges. Linear or point
42 thermal bridges are parts of the building envelope, where the otherwise uniform thermal
43 resistance varies significantly. Point thermal bridges typically occur where an insulated wall is
44 perforated (penetration) by an element with high thermal conductivity. They are characterised
45 by a point thermal transmittance (χ -value in W/K). It is possible to reliably determine the heat
46 transfer through a thermal bridge via numerical methods using finite element or finite difference
47 methods [2] in accordance with methods specified in EN ISO 10211 [3]. The implementation of
48 external steel structures, such as balconies, on a facade with external thermal insulation, reduces
49 thermal bridges, compared to concrete structures, due to the point fixings on the support instead
50 of continuous extension of the floor slab [4]. However, the thermal performance can be further
51 improved using thermal breaks between the external steel structure and the support. Thermal
52 breaks can be either standard manufactured products proposed by industrial suppliers or simpler
53 solutions implemented by companies. The advantages of these solutions are their reduced costs
54 and ease of erection. A simple solution consists in inserting an intermediate layer between the
55 end-plate of the steel beam and the steel/concrete support. This kind of solution has attracted a
56 lot of attention during the recent years by varying the **insulating material**. Nasdala et al [5] and
57 Sulcova et al [6] performed numerical simulations and adapted the Eurocode 3 component

58 approach to take into account the intermediate layer. The thickness of the layer as well as the
59 friction coefficients influence strongly the final behaviour of the connection. Cleary et al [7]
60 loaded under bending and shear force thermal break composed of Fiber Reinforced Resin and
61 highlighted a decrease of the stiffness of the connection. Peterman et al ([8], [9]) demonstrated
62 that Fiber-Reinforce Polymer don't modify significantly the behaviour of the connection and
63 don't yield in compression. Hamel and White [10] used neoprene as intermediate layers and
64 obtained low rotational stiffness and bending resistance mainly as a result of a significant
65 crushing of neoprene in the compressive area caused by the bending moment. Ben Larbi et al
66 [11] drawn similar conclusions inserting elastomer in conjunction with PVC. However, the
67 decrease of the ultimate bending resistance was quite limited. Elastomer and neoprene are too
68 flexibles for moment resisting connections. More rigid solutions such as PVC or wood seems to
69 be necessary to guarantee a minimum rigidity of the connection and to limit the deflections of
70 the balconies. Moreover, this solution is clearly cheaper than Fiber Reinforced Polymer.

71 Solutions of thermal breaks composed of a PVC or plywood intermediate layers placed
72 between end plates of beam attached to a steel or concrete support have been studied in the
73 framework of the French ADEME (Environment and energy management Agency) project
74 TREPOS [12]. The first results of this project concern the thermal and mechanical performances
75 for external steel structures attached on a steel support [13]. These solutions were able to reduce
76 the thermal bridge by 40 to 50 % and achieve around 70% of the performances of industrial
77 solutions [12]. The various mechanical test campaigns have shown that thermal breaks with
78 extended end-plate can develop an interesting behaviour with regard to the mechanical strength
79 requirements of steel cantilever beam and in particular the deflection criteria. Thermal breaks
80 composed of stiffened extended end-plate can develop a behaviour similar to that of a
81 conventional non-stiffened end-plate. These results for thermal break connecting steel beam to
82 steel support were encouraging. However, in practice many steel balconies are also connected to
83 concrete support. Investigations are thus needed to estimate the behaviour of these solutions of
84 thermal breaks in this situation. Ben Larbi et al [11] characterized solutions of thermal breaks

85 attached to concrete block with passing through anchor bolts, but this anchoring system was
86 different from common practices. In addition, the intermediate insulating layers, composed of
87 elastomer (for acoustic insulation) and PVC, were clearly more flexible. It is thus necessary to
88 analyse more realistic anchoring system such as post-installed fasteners or bolted embedded
89 plates with intermediate insulating layer made of PVC or plywood that develop an interesting
90 mechanical response for the connection of external steel structure to a steel support [13]. **The**
91 **numerical study performed in the framework of TREPOS project [12] highlighted that the**
92 **proposed solution attached to concrete support reduces the thermal bridge (γ) by about 35 to**
93 **50% in comparison with a connection without thermal break, and are able to achieve around**
94 **70% of the thermal performances of industrial solutions but are 5 to 10 times less expensive.**

95 The fire resistance is generally required for the attached steel structure in façade. In this case,
96 it should ensure the good structural performance of the attached structure with thermal breaks.
97 The use of specific materials for thermal breaks, such as PVC, FRP/FRR or plywood, could lead
98 to a loss of mechanical strength at high temperatures when these items are subjected to the
99 flames coming out of the façade openings. Moreover, the attached steel structure may have a
100 significant role to reduce the risk of fire spread to upper levels [14]. Therefore, it is essential
101 that the sealing hot gases and flames is ensured at the junction between the attached structure
102 and the facade. Among European countries, several tests **exist** to assess the impact of flames on
103 a façade. They are oriented toward fire reaction and the LEPiR II, a French large-scale test, is
104 one of them. It can be used as a basis to test the fire resistance of a structure. Within the
105 framework of the project TREPOS, the LEPiR II test setup was adapted and used for that goal
106 to test balconies with and without thermal breaks.

107 The present paper gives the results of a study assessing the mechanical and fire behaviour of
108 simple solutions of thermal breaks composed of a PVC or plywood layer implemented between
109 extended end-plate connections of steel beams and concrete support (lintel, floor slab, concrete
110 wall). Monotonic tests performed on 12 cantilever beams attached with and without the
111 proposed solution of thermal break are presented in section 2. The connections were subjected

112 to major bending moment and minor shear force typically applied to connections of balconies.
113 Two types of mechanical fastening typically used in practice have been studied; post-installed
114 mechanical/chemical fasteners and bolted embedded plates. The effect of bolt arrangement and
115 properties of thermal insulating layers **were** also studied and particularly the impact on the
116 rotational stiffness, **plastic**/ultimate bending resistance and failure mode. The results of two fire
117 tests performed on balcony attached with and without thermal breaks are presented in section 3.
118 During these fire tests, the LEPiR II test setup was adapted. These tests highlight the effect of
119 the presence of external **thermal** insulation around the connection of the balcony as well as the
120 impact of the intermediate insulating layer on the global behaviour of the balcony.

121 **2. Mechanical tests**

122 *2.1. Introduction*

123 The proposed solution of thermal break has been characterized mechanically by monotonic
124 tests on connections attached to concrete structures composed of a lintel and a slab. The
125 objective was to study the impact of the type of material constituting the intermediate insulating
126 layer (PVC, plywood), the shape of the steel beam (IPE, RHS or SHS) and the type of anchoring
127 system: post-installed chemical/mechanical fasteners or bolted embedded plates. This later
128 parameter is crucial as it affects the resistance of the tensile area and thus the bending moment
129 resistance of the connection. In previous experimental campaign on thermal breaks attached to a
130 concrete block [11], failure was governed by anchor bolt rupture in tension. The present
131 campaign was designed to obtain realistic failure modes and observe possibly failure of the
132 concrete components such as concrete cone/edge failure.

133 *2.2. Test presentation*

134 *2.2.1. Test set-up*

135 The test set-up presented in Figure 1 is representative of a balcony attached to a concrete
136 slab and a lintel. A load-jack apply a vertical force at 1150 mm from the connection in order to

137 produce the bending moment due to gravity loading that are significant for the connection of a
138 cantilever balcony. The concrete block is maintained at the front and the back by two
139 rectangular steel frames. On the front side, the concrete block is placed directly on two neoprene
140 supports resting on the lower beam of steel frame A. At the back (portal frame B), rollers are
141 welded to a load distribution plate positioned on the concrete block (see Figure 1). Lateral
142 supports located at mid-length of the beam were used to prevent lateral torsional buckling of the
143 steel profile.

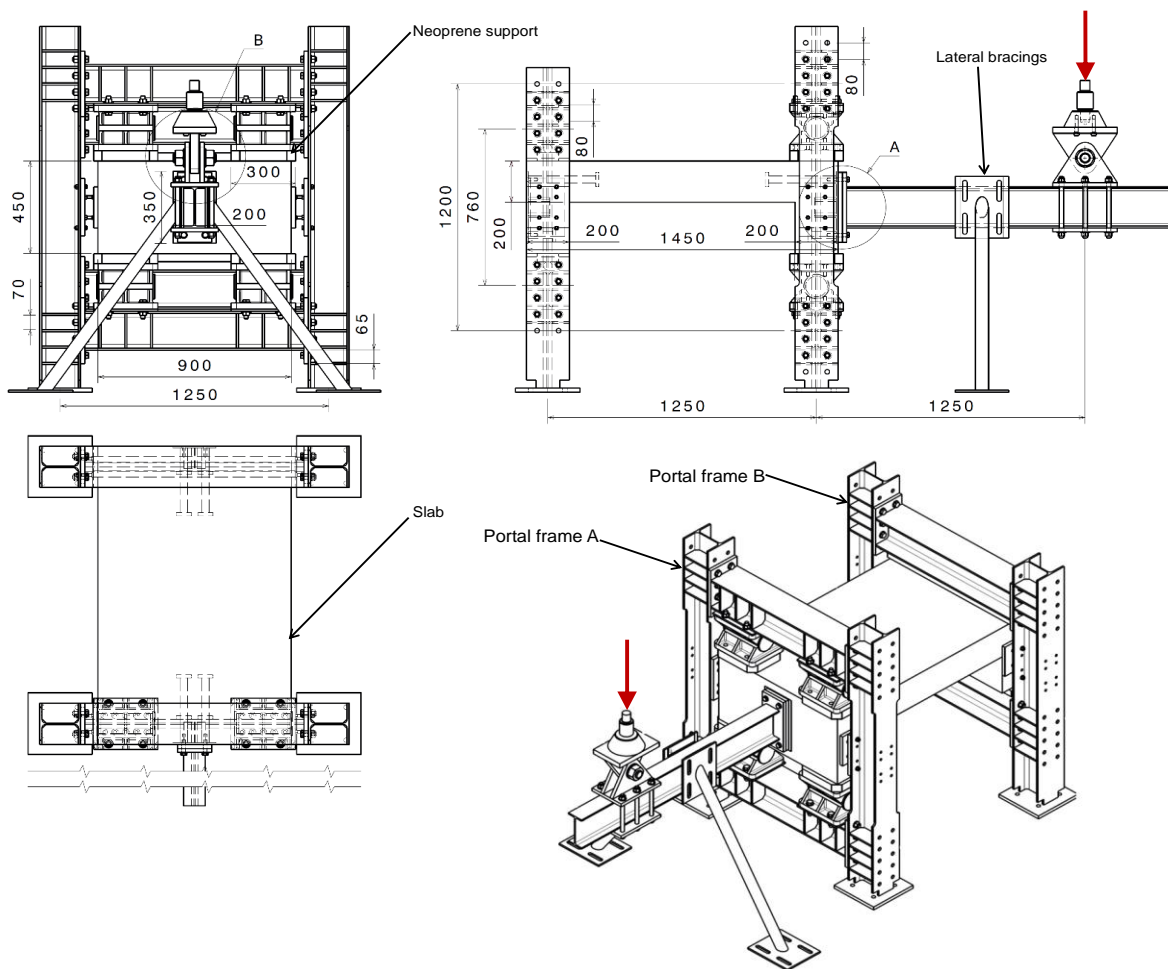


Figure 1 : Mechanical test set-up (dimensions in mm)

144 2.2.2. Connection geometry

145 The connection geometries have been defined in order to test the influence of the materials
146 used as thermal insulation (PVC or plywood), the configuration of the connection (stiffened or
147 non-stiffened extended end-plate), the shape of the beam attached (I-beam section,

148 rectangular/square tubes) and the anchoring system (chemical/mechanical post-installed
 149 fasteners, bolted embedded plates). The end-plates are made of S235 steel. The bolts are M16
 150 class 8.8. The IPE 200s are made of S275 steel and the tubes of S355. For connections RTPB-1-
 151 P/B, RTPB-2-P/B and RTPB-3-P, the thickness of PVC and plywood are equal to 20 mm and
 152 30 mm, respectively. Connection RTPB-5-P1 use PVC of 10 mm thickness. The main
 153 geometric parameters used are shown in Figure 2 and Table 1. The bolt length is equal to 60
 154 mm for end-plate directly bolted to embedded plates (PB-1, PB-2 and PB-3) but to 90 mm in
 155 presence of intermediate later. Bolts are fully and partially threaded respectively for the length
 156 of 60 and 90 mm.

157 **Table 1.** *Geometry of specimens tested*

Specimen	Beam	t_p	t_a	Insulating	Anchoring	Stiffener	Concrete block
PB-1	IPE200	20	-	-	Embedded plate n°2	No	B3-1
RTPB-1-P	IPE200	20	20	PVC	Embedded plate n°2	No	B2-1
RTPB-1-B	IPE200	20	30	Plywood	Embedded plate n°2	No	B3-1
PB-2	IPE200	15	-	-	Embedded plate n°1	Yes	B1-1
RTPB-2-P	IPE200	15	20	PVC	Embedded plate n°1	Yes	B1-1
RTPB-2-B	IPE200	15	30	Plywood	Embedded plate n°2	Yes	B2-2
PB-3	RHS 200×120×6	20	-	-	Embedded plate n°2	Yes	B3-2
RTPB-3-P	RHS 200×120×6	20	20	PVC	Embedded plate n°2	Yes	B3-3
PB-4	IPE200	15	-	-	Post-inst. HIT-Z-R	Yes	B1-2
RTPB-4-P	IPE200	15	20	PVC	Post-inst. HIT-Z-R	Yes	B1-2
PB-5	SHS 80×5	10	-	-	Post-inst. HST 3	No	B3-3
RTPB-5-P1	SHS 80×5	10	10	PVC	Post-inst. HST 3	No	B3-2

158

159 Embedded plates have been designed to carry the bending resistance of the connection and
 160 are composed of four rods threaded to be bolted. A shear lug permit to transmit the shear force.
 161 A washer plate of 60×135×10 is welded to the lower and upper rods for Embedded plate n°1
 162 (see Figure 2-b). For embedded plate n°2, a washer plate of 60×60×10 was welded on each
 163 upper rod (see Figure 2-a/c). The dimensions of these embedded plates were reduced to ease its
 164 implementation in steel reinforcements of the lintel.

170 Connections PB-2 and RTPB-2-P/B (see Figure 2-b) are made of stiffened extended end-plate
171 of 15 mm thickness and are similar to connection RTPA-5-P attached to a steel support [13].
172 For thermal breaks attached to steel structures, it was highlighted that stiffeners increase
173 significantly the mechanical properties under monotonic and cyclic loadings. The end-plate is
174 also bolted to an embedded plate with shear lug and four rods.

175 Specimens PB-3 and RTPB-3-P (see Figure 2-c) are composed of stiffened extended end-
176 plate connecting a RHS 200×120×6. These connections are similar to specimens PA-4 and
177 RTPA-4-P attached to steel support [13] but the end-plate thickness is increased to 20 mm in
178 order to limit weld tearing at failure. The end-plate is also bolted to an embedded plate with
179 shear lug and four threaded rods.

180 The steel connection of PB-4 and RTPB-4-P (see Figure 2-d) is similar to that of PB-2 and
181 RTPB-2-P but it is connected to the concrete block by means of post-installed chemical
182 fasteners HIT-Z-R M20 with an embedded length of 140 mm. This solution can be used in the
183 case of rehabilitation of concrete building.

184 The steel connection of PB-5 and RTPB-5-P1 (see Figure 2-e) is identical to that of PA-6 and
185 RTPA-6-P1 [13] and is fastened by post-installed mechanical fasteners HST3 with an
186 embedded length of 70 mm.

187 The dimensions of the concrete block, presented in Figure 2-f, are typically used in practice
188 with a slab and a lintel of 20 cm thickness. The height of the lintel, equal to 45 cm and widely
189 used in practice, has been chosen to maximise the lever arm of the connection and thus
190 mechanical properties. The concrete block were casted in three times (B1, B2 and B3) and one
191 block was used for two tests except B2-1 and B2-2 that failed during the first test by rupture of
192 the slab reinforcements. Hence reinforcement of concrete block B2 were typical of common
193 practice. The steel reinforcement consists in a steel reinforcing mesh ST 25C. The cover to
194 reinforcement was 25 mm. Additional C-shape plain round bars were placed along the support
195 zones. Shear reinforcements were added near the supports. For blocks B1 and B3, HA 12 of
196 steel grade S500 were added to avoid concrete slab failure due to bending and permit to reach

197 the full capacity of the connection. The reinforcement of concrete blocks B1 and B3 is presented
198 in Appendix A. Concrete block B2 is identical to block B3 except that the HA 12 have not been
199 added.

200 2.2.3. Loading-procedure

201 Specimens were subjected to a monotonically increasing vertical force. All specimens were
202 tested under displacement controlled loading. The loading was divided into elastic and plastic
203 phases. The load range applied to the specimen during the first phase was estimated by ensuring
204 that the connection remains elastic. The elastic phase consisted in three equal amplitude
205 loading-unloading cycles with the load magnitude being equal to 50% of the elastic loading F_{el}
206 and a fourth cycle where the load is increased to the full elastic loading. This loading was equal
207 to 20 kN for all specimens except for PB-5 and RTPB-5-P1 where it was limited to 2,5 kN. In
208 the final phase, specimens worked in the plastic domain and were loaded until failure of one or
209 more connection components.

210 2.2.4. Instrumentation

211 Displacement transducers (LVDT) were positioned to measure the displacements at different
212 locations of the test specimen as shown in Figure 3. Displacement sensors (V_1 to V_8 and U_1 to
213 U_4) allowed a complementary estimation of the rotation. Finally, four displacement sensors
214 placed horizontally were used to evaluate the crushing of the thermal insulation (U_{b1} and U_{b2}),
215 and the elongation of the tensile part of the connection (U_{t1} and U_{t2}). An inclinometer, I_2 , was
216 positioned on the beam web at half the beam height from the connection and another one on the
217 concrete slab, I_1 .

218

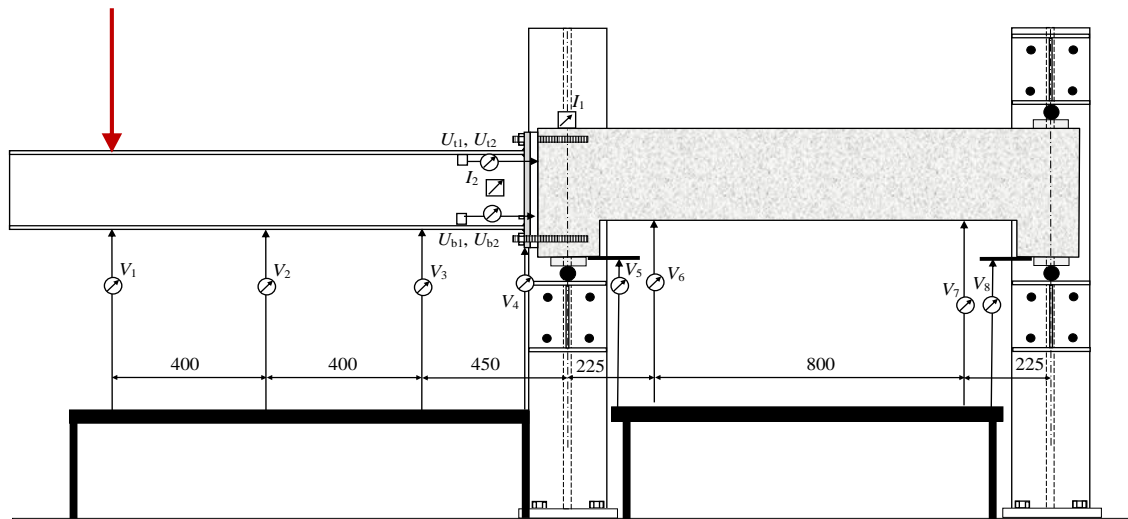


Figure 3 – Displacement transducers and inclinometers (dimensions in mm)

219 2.3. Mechanical characterization

220 Steel coupons were extracted from the IPE 200 beams, end-plates and bolts (3 coupons per
 221 samples) whose average mechanical characteristics are listed in Table 2. An extensometer was
 222 used at the mid-gage of the samples to estimate the Young modulus.

223 Table 2. Mechanical properties of steel components

Component	Young Modulus	Yield strength	Ultimate strength	Necking
	N/mm ²	N/mm ²	N/mm ²	%
End-plate 15 mm	206355	277,5	446,8	70
End-plate 20 mm	206140	229,1	396,7	63
IPE 200 web	220255	340,1	483,5	64
IPE 200 flange	218827	333,0	489,9	65
Bolt M16 8.8 L90	198004	1063,5	1082,2	69
Bolt M16 8.8 L60	210000 ⁽¹⁾	1022	1056	69
Anchor bolt HIT-Z-R M20	188765	724,4	764,5	78

⁽¹⁾ This value correspond to the nominal one's because of the absence of reliable results

224

225 PVC and plywood were from the same samples as those used for tests on thermal breaks
 226 attached to steel structures [13], the corresponding mechanical properties are presented in this
 227 reference. For each tests, 3 concrete cylinders have been tested under compression the day of
 228 the test. The average concrete compressive strengths are summarized in Table 3 for each test.

229

230

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Table 3. Average concrete compressive strengths

Test ID	f_{cm}
-	N/mm ²
PB-1	35,3
RTPB-1-P	27,8
RTPB-1-B	34,6
PB-2	34,1
RTPB-2-P	33,6
RTPB-2-B	31
PB-3	38,0
RTPB-3-P	36,7
PB-4	36,2
RTPB-4-P	34,1
PB-5	36
RTPB-5-P1	35,7

232 **2.4. Mechanical test results**

233 The moment rotation curves of the different types of connections are compared in Figure 4.

234 The rotation of the connection is evaluated with the displacement transducers U_t and U_b . In
 235 addition the following parameters are extracted and provided in Table 4:

- 236 • the **secant rotational stiffness** $S_{j,sec}$ evaluated for $2/3M_{j,pl}$,
- 237 • the **plastic bending moment** $M_{j,pl}$ calculated according to ECCS requirements [15] that
 238 corresponds to the frontier between the elastic and elasto-plastic ranges of behaviour,
- 239 • the **ultimate bending moment** $M_{j,u}$ corresponding to the maximum bending moment
 240 applied during tests,
- 241 • elements that yield/crack before failure and the **failure mode**.

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Table 4. Rotational stiffness 's, failure modes, plastic/ultimate bending moments

Connection	$S_{j,sec}$ kNm/rad	$M_{j,pl}$ kNm	$M_{j,u}$ kNm	Failure mode -	Yielding/cracking -
PB-1	8452	50,4	71,6	Bolt	Bolt, end-plate, slab, lintel
RTPB-1-B	8371	33,0	80,7	Bolt	Bolt, end-plate, slab, lintel, wood
RTPB-1-P	5207	30,0	61,6	Reinforcement	Bolt, end-plate, slab, lintel, PVC
PB-2	21134	47,6	86,0	Bolt	Bolt, end-plate, slab, lintel, weld
RTPB-2-B	13043	32,6	54,9	Reinforcement	end-plate, slab, lintel, wood
RTPB-2-P	5259	37,6	91,5	Bolt	Bolt, end-plate, slab, lintel, PVC
PB-3	7518	52,7	80,8	Bolt	Bolt, slab, lintel
RTPB-3-P	6165	42,0	72,6	Bolt	Bolt, slab, lintel, PVC
PB-4	7617	41,0	46,8	Concrete edge	Concrete, end-plate
RTPB-4-P	7748	28,6	38,5	Concrete edge	Concrete, PVC
PB-5	530	3,3	6,6	Concrete cone/sliding	Concrete, end-plate
RTPB-5-P1	526	3,4	6,0	Concrete cone/sliding	Concrete, end-plate, PVC

250

251 Four types of failure modes have been obtained depending on the anchoring system. For
 252 embedded end-plates with concrete slab sufficiently reinforced (blocks B1 and B3), the failure
 253 corresponds to bolt rupture in tension with or without end-plate yielding. End-plate yielding was
 254 observed for configurations PB-1 and PB-2 and thus in presence of non-stiffened extended end-
 255 plate, and stiffened extended end-plate of 15 mm thickness. The addition of intermediate
 256 insulating layers did not affect the final failure mode. The same results can be drawn for the
 257 proposed solution of thermal break attached to a steel support [13].

258 On the contrary, when the concrete slab was not sufficiently reinforced (using only steel
 259 reinforcing mesh ST 25C), the final failure was caused by rupture of reinforcement of the ST 25
 260 C mesh (see Figure 5-b). The bending resistance of the connection was greater than the capacity
 261 of the slab in bending. For post-installed chemical fasteners (connections PB-4 and RTPB-4-P),
 262 concrete edge failure is obtained and the addition of intermediate insulating layer does not affect
 263 significantly the result. For post-installed mechanical fastener (PB-5 and RTPB-5-P1), the
 264 failure corresponds to anchor bolt sliding and finally concrete cone failure.

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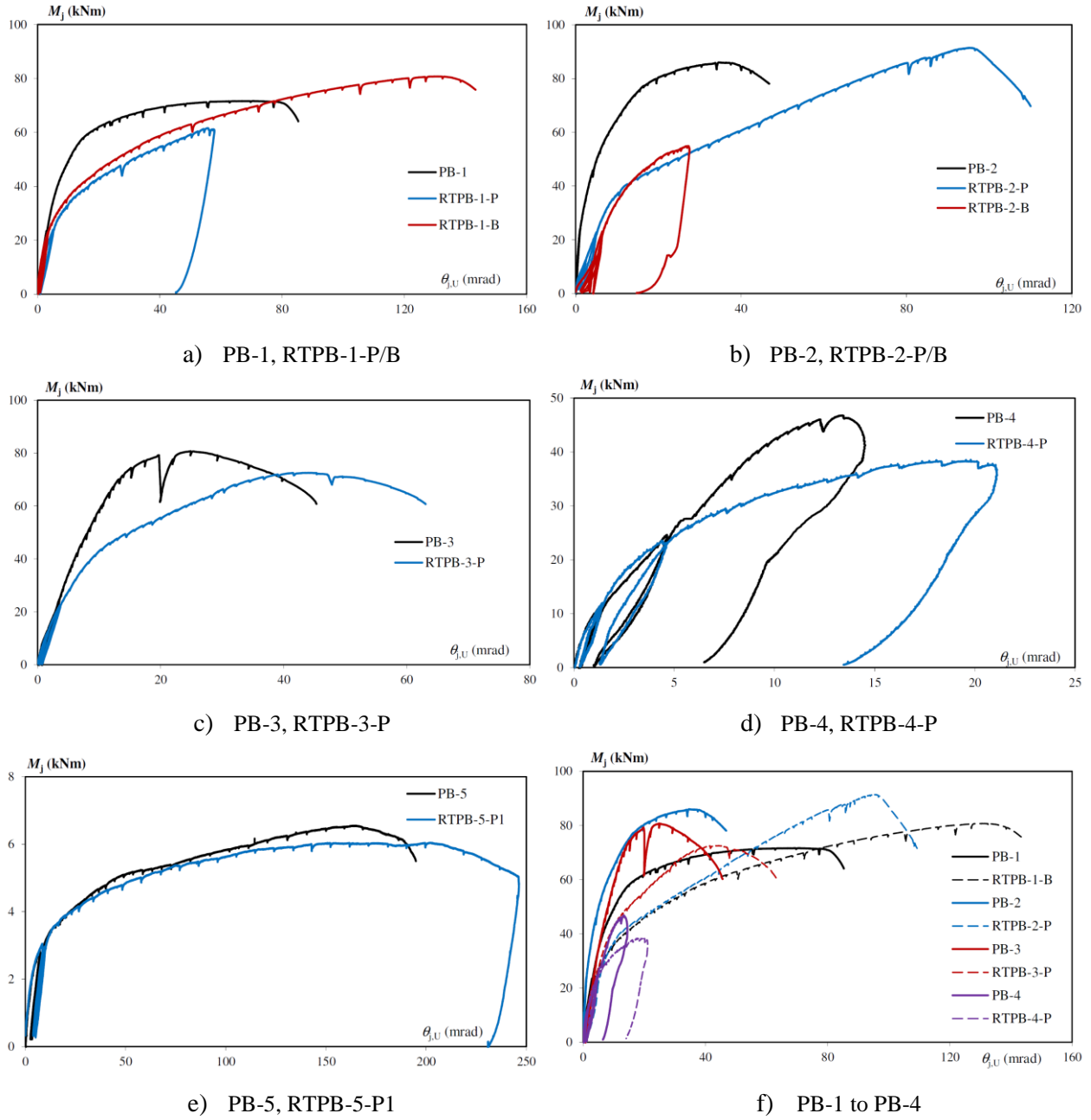
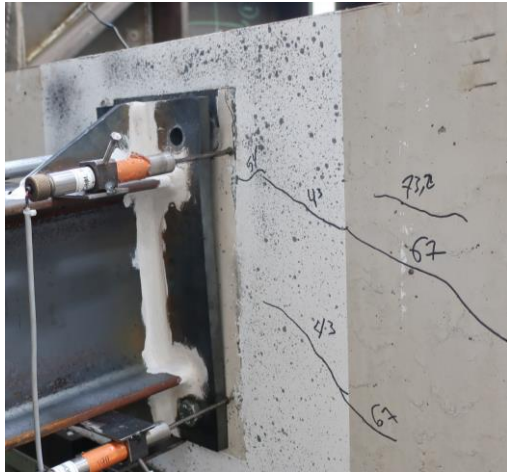
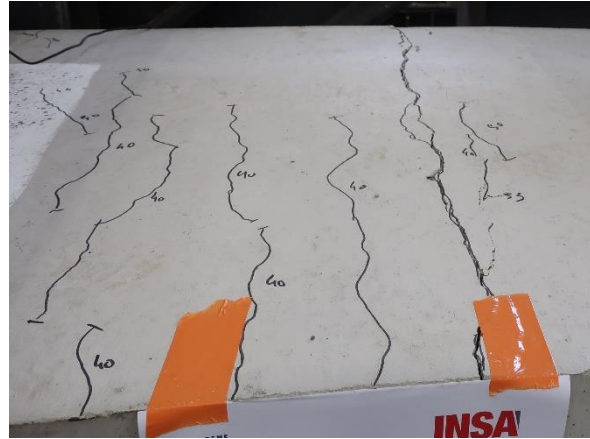


Figure 4. Moment-rotation curves



a) Lintel cracking



b) Slab reinforcement rupture



c) Concrete edge failure



d) Concrete cone failure/anchor sliding

Figure 5. Concrete lintel and slab cracking and failure

267 A yield line parallel to the beam flange develops on the end-plate in the tensile area for PB-1
268 and RTPB-1-P/B (see Figure 6). The deformation of RTPB-1-P is less pronounced due to an
269 early failure of concrete slab reinforcements. End-plate yielding is the main source of ductility
270 of connection PB-1. For connections using insulating layer, yielding in compression of the layer
271 also affects the rotation capacity and explains its increase comparatively to the conventional
272 connection (see Figure 4-a). The addition of plywood and PVC decreases the secant stiffness of
273 1 % and 38 %, respectively. This difference between the two materials is quite surprising
274 comparatively to results obtained with other configurations but can be partially explained by
275 different initial preloading and the variability of plywood mechanical properties. The plastic
276 bending moment decrease of 34 % and 40 % with the addition of plywood and PVC,

277 respectively. This reduction is caused by the yielding of the insulating layer, PVC or plywood,
278 in the compressive area that appears before end-plate yielding in the tensile area. For the three
279 connections cracks, initiate on the lintel close to the upper face of the end-plate for a bending
280 moment of 45-50 kNm and propagate to the support (see Figure 5-a). A similar trend was
281 observed for connections of IPE 200 and RHS 200. Cracks also develop on the slab but with a
282 final rupture of steel reinforcements for RTPB-1-P. For PB-1, the prying force is positioned at
283 the outer edge of the end-plate (see Figure 6-a) whereas for specimens RTPB-1-P/B, the contact
284 area extends between the free edge of the end-plate and the bolts (see Figure 6-b/c). A similar
285 trend was observed for the proposed solution of thermal breaks attached to a steel support [13].
286 In presence of intermediate layer, the contact between the end-plate and the layer extends from
287 the extended end-plate to a portion of the beam web in the compressive area (see Figure 6-b/c).
288 In absence of thermal break, the compression area only develop at the bottom of the extended
289 end-plate. The use of plywood increase the ultimate bending resistance of 12,7 % (see Table 4).
290 This difference can be explained by the fact that the length of the threaded area is different for
291 specimens with and without thermal insulating layers but also by the reduction of prying effect
292 in presence of intermediate layer. The tensile resistance of bolts can be affected by the length of
293 the threaded area ([16], [17]). Hence, the connections without insulating layers used bolts that
294 were fully threaded of 60 mm length. On the contrary, the connections with thermal break used
295 partially threaded bolts of 90 length whose tensile resistance is potentially higher. The addition
296 of an intermediate layer also reduce prying effects. This difference was not observed for thermal
297 breaks attached to a steel structure [13] because the same bolts were used.

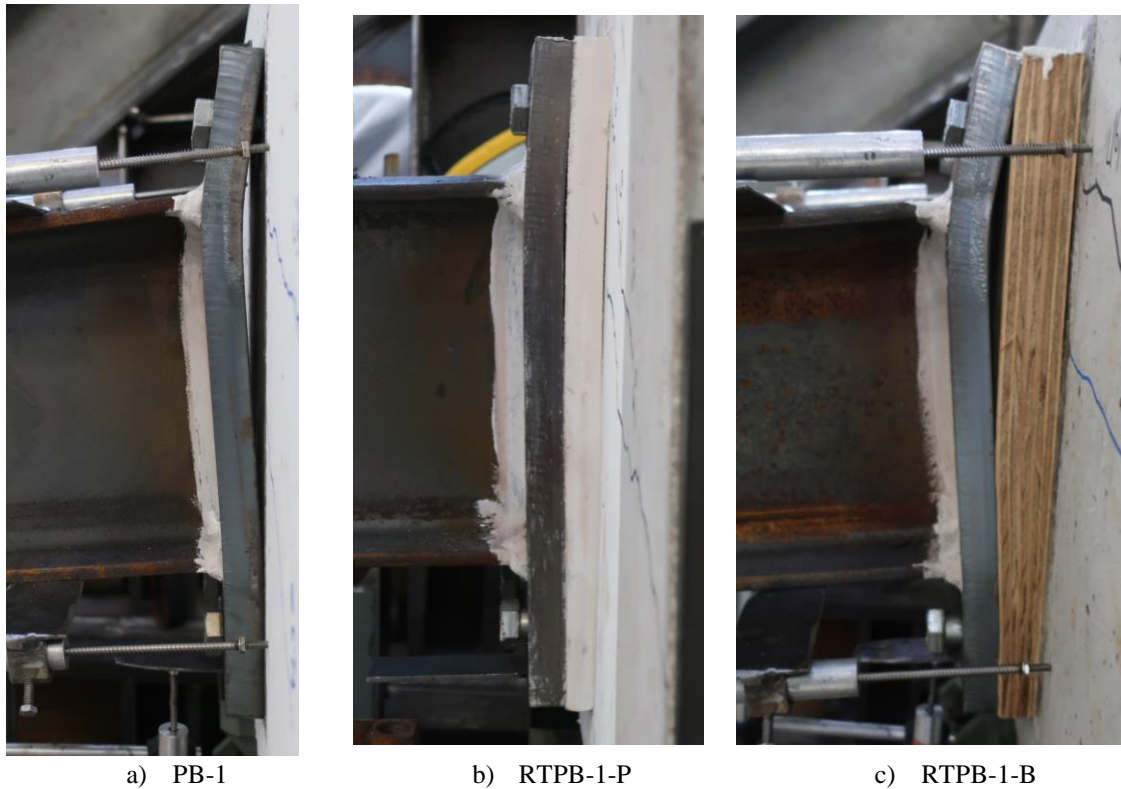


Figure 6 – Deformed shape of specimens PB-1, RTPB-1-P/B at failure

298 For stiffened extended end-plates with I-profile, the addition of intermediate insulating layers
299 modifies significantly the moment-rotation curve (see Figure 4). The secant rotational stiffness
300 decrease of 38 % and 75 % respectively with plywood and PVC. This reduction is more
301 significant than for non-stiffened extended end-plates, however similar conclusions could be
302 drawn for thermal breaks attached to a steel support [13]. In fact, the centre of compression
303 positioned close to the bottom end-plate in absence of intermediate layer, for PB-2, gets closer
304 to the beam centre in presence of PVC and plywood. The lever arm decrease and consequently
305 the rotational stiffness. This phenomena is less pronounced for non-stiffened extended end-
306 plates. The rotation capacity is doubled by the addition of PVC due to its crushing in
307 compression. For the three specimens, the end-plate yields in bending in the tensile area and it is
308 the only source of ductility for PB-2. This intermediate layer yielding in compression is also at
309 the origin of the decrease of the plastic bending moment of 31,5 % and 21 % for plywood and
310 PVC, respectively. The addition of PVC also increases the ultimate resistance of 6,4 % mainly
311 because of a decrease of prying effect and of the use of different bolts, partially/fully threaded.

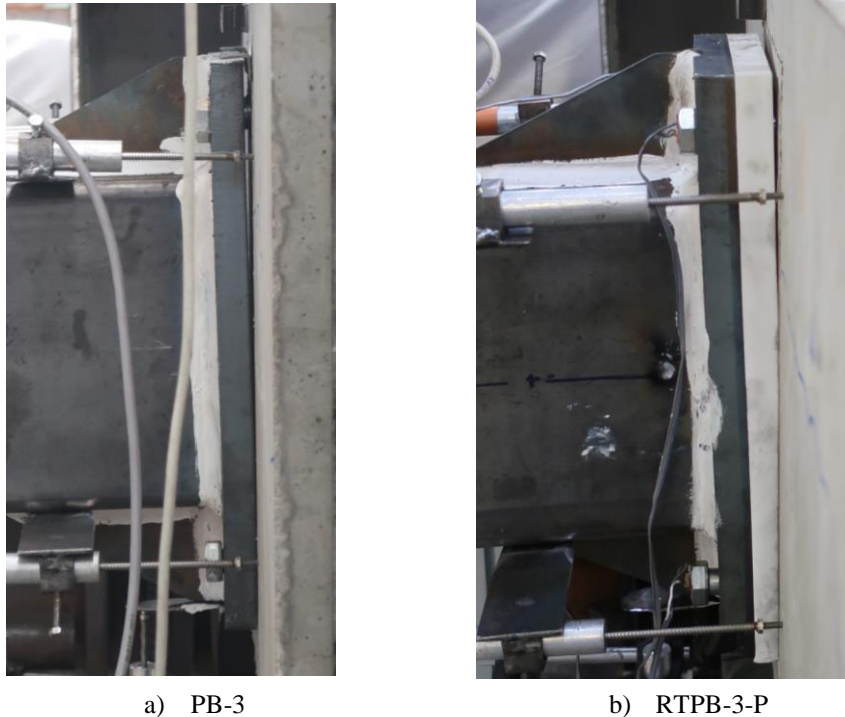
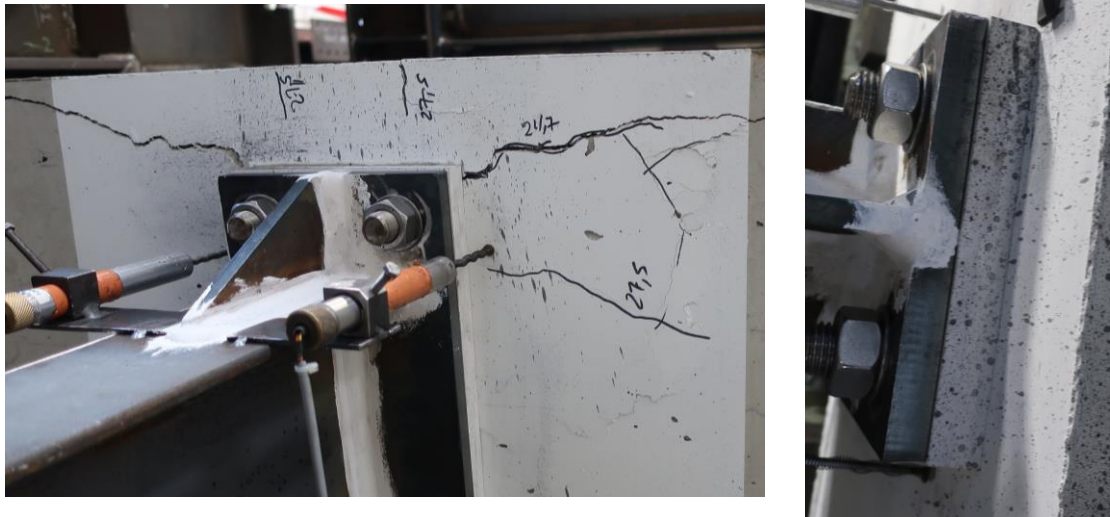


Figure 8 – Deformed shape of specimens PB-3 and RTPB-3-P at failure

327 The secant rotational stiffness is not impacted by the addition of PVC (see Table 4 and Figure
328 4). Nevertheless the plastic and ultimate bending moments decrease of 30,2 and 17,7 %, respectively.
329 These reductions are caused by the reduction of the lever arm and an early yielding of PVC in the compressive area.
330 The rotation capacity is almost doubled due to PVC crushing in compression but is very limited comparatively to connections fastened to the lintel by bolted
331 embedded end-plates (see Figure 4-f). The geometry of connections of PB-4 and PB-2 are identical except fastening to the concrete **block** that corresponds to post-installed mechanical
332 fastener and bolted embedded plates, respectively. The ultimate resistance of PB-4 is almost divided by 2 comparatively to PB-2 because of the limited anchoring of the first one. The
333 stiffness is also divided by 3. These differences are typically observed between these two fastening systems. However, in presence of intermediate layers, the difference of plastic
334 bending moment is not significant. Connections using post-installed fasteners are not really affected by the addition of insulating layers because the tensile area is already flexible.
335
336
337
338
339



a) PB-4



b) RTPB-4-P

Figure 9 – Specimens PB-4 and RTPB-4-P at failure

340 The failure mode of connections PB-5 and RTPB-5-P1 starts by anchor sliding in the tensile
341 area and is followed by the concrete cone failure (see Figure 10). Prying effects did not develop
342 for the two specimens due to the flexibility of the mechanical fastener. Nevertheless, the **failure**
343 **pattern** is quite different, hence a yield line parallel to the tube develops in the compression area
344 to accommodate the rotation of the connection for PB-5. For the connection with intermediate
345 layer, this rotation is ensured by the PVC crushing in compression, the end-plate deformation is
346 reduced. The concrete lintel started to crack for a bending moment around 4-4,5 kNm. The
347 addition of the PVC layer does not impact the secant rotational stiffness as well as the plastic

348 bending moment, the flexibility of the connection is mainly affected by the mechanical fastener
349 elongation. The ultimate bending moment decrease of 10%, this reduction can be caused by the
350 modification of the position of the centre of compression.

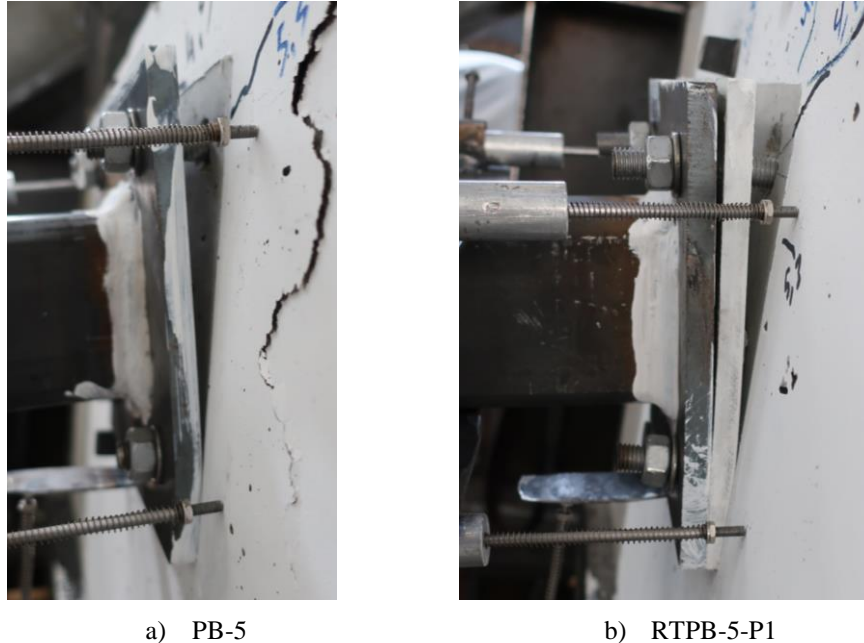


Figure 10 – *Specimens PB-5 and RTPB-5-P1 at failure*

351 **2.5. Summary on mechanical tests**

352 The main conclusions that can be drawn from the monotonic mechanical tests are the
353 following:

354 • The secant rotational stiffness is not strongly affected by the addition of insulating
355 layer in presence of post-installed mechanical/chemical fasteners mainly because the
356 tensile area is already quite flexible. In presence of embedded end-plates, the
357 addition of PVC or plywood can significantly decrease the secant rotational stiffness
358 because **these connections** are quite rigid without intermediate layer. However the
359 rotational stiffness value was greater than 5000 kNm/rad for connections of beam of
360 200 mm depth and was largely acceptable for a balcony.

361

362 • Except for PB-5, the plastic bending resistance is reduced (between 20 and 40 %) by
363 the addition of intermediate layer whatever the material used. This decrease is due to

364 yielding of PVC/Plywood in the compressive area. The plastic bending moment of
365 PB-5 is not affected by the addition of PVC probably because the anchoring sliding
366 causes firstly the non-linearity.

367 • The failure was not affected by the addition of intermediate layer and corresponds to
368 bolt rupture in tension for bolted embedded plates (with reinforced slab) or concrete
369 cone/edge failure for post-installed mechanical/chemical fasteners. The reduction of
370 the ultimate bending moment of the connection was quite limited by the addition of
371 intermediate layer, lower than 20 %. On the contrary, the resistance could be
372 increased by the addition of PVC or plywood mainly because of a reduction of the
373 threaded length and the modification of prying effect.

374 • The rotation capacity is significantly increased by the addition of intermediate layer
375 whatever the design of connection because of yielding of the layer in the
376 compressive area. The rotation capacity of non-stiffened extended end-plate
377 (connections PB-1 and RTPB-1-P/B) is increased compared to that of stiffened
378 extended end-plate (connections PB-2 and RTPB-2-P/B) as a result of the
379 development of an important plastic yield line parallel to the beam flange. The
380 rotation capacity of connections PB-4 and RTPB-4-P is very limited comparatively
381 to that of PB-2 and RTPB-2-P mainly due to the absence of steel component
382 yielding. The curves of PB-5 and RTPB-5-P1 are quite close mainly because the
383 deformability come mostly from sliding of fasteners in the tensile area.

384

385 The behaviour of bolted end-plate connections attached to concrete support with
386 intermediate layer is similar to that obtained with a steel support by Couchaux et al [13] and
387 corresponds to a reduction of the rotation stiffness as well as the plastic bending but to identical
388 failure modes and limited reduction of the ultimate bending resistance. For post-installed
389 fasteners, the effect of the intermediate layer is more limited particularly concerning the
390 rotational stiffness.

391

392 **3. Fire tests**

393 *3.1. Introduction*

394 The fire resistance is generally required for the attached steel structure in façade [14]. In this
395 case, it should ensure the good performance of the attached structure with thermal breaks. The
396 use of specific materials for thermal breaks could lead to a loss of mechanical strength at high
397 temperatures when these items are subjected to the flames coming out of the facade openings.
398 Moreover, the added structure may have a significant role to reduce the risk of fire spread to
399 upper levels. Therefore, it is essential that the sealing hot gases and flames is ensured at the
400 junction of the attached structure and facade. For this reason, two fire tests have been performed
401 on balconies attached with and without thermal breaks that were exposed to external flames
402 generated by a generalized fire in a furnace. The fire test set-up as well as measurements are
403 presented in section 3.2. The tests results are discussed in section 3.3 in term of temperature
404 evolution and beam displacement.

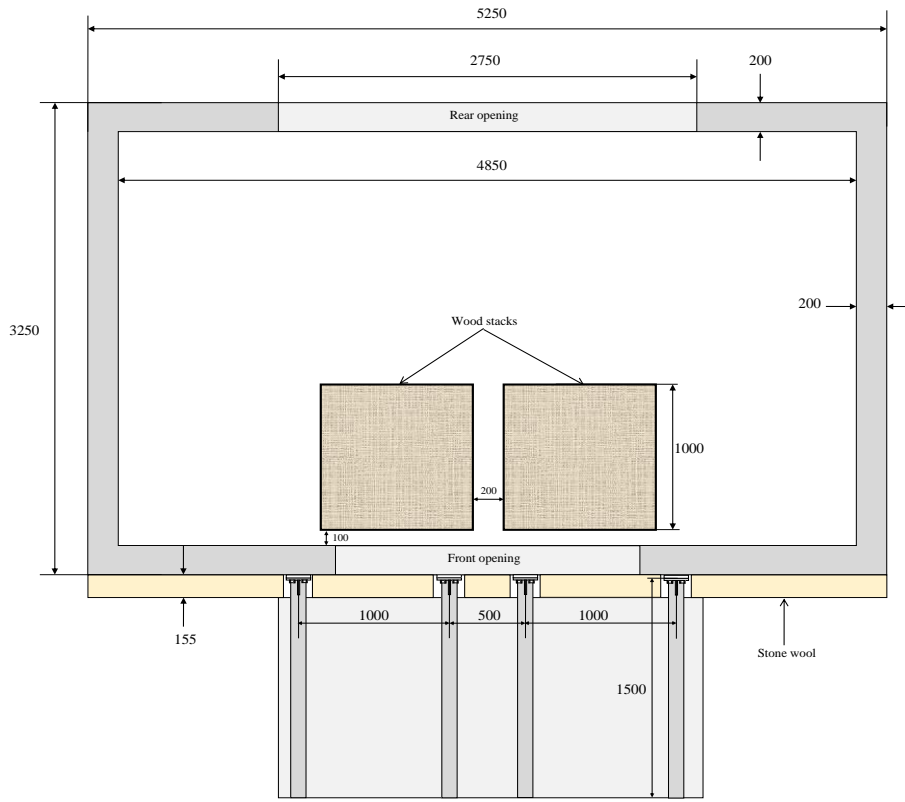
405 *3.2. Fire test presentation*

406 3.2.1. Fire test furnace

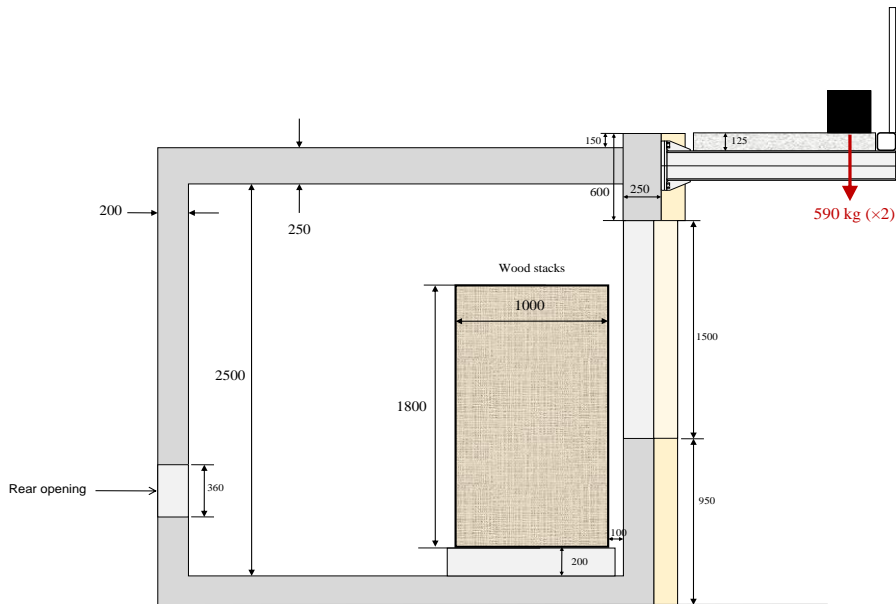
407 Among European countries, several tests exist to assess the impact of flames on a facade.
408 They are oriented toward fire reaction and the LEPİR II, a French large scale test [18], is one of
409 them. It can be used as a basis to test the fire resistance of a structure. The test facility consists
410 in a block usually used for LEPİR II normative tests (experimental test facility with two levels
411 under natural fire). For a LEPİR II test, the furnace has two blocks : one for the ground level,
412 the other for the upper level. Each block has two standardized openings. In the present tests,
413 only the ground block is used and slight modifications have been done.

414 The internal dimensions of the furnace are 2850×4850×2500 mm. The front face is closed
415 with a 200 mm thick concrete block, and at the top, the reinforced concrete slab is 250 mm

416 thickness, with a lintel of 600 mm high. The front face had one opening of dimensions
 417 2000×1500 mm, located at 950 mm from the ground (see Figure 12-a).



a) Plan view



b) Lateral view

Figure 11. Plan and lateral view of the test set-up (dimensions in mm)

418 This size allows to have two balconies per test and also to have one of the beams highly
419 exposed to fire. A LEPiR II tests involves two openings of 1000×1500 mm. Thus, the opening
420 area is the same and also the ventilation. The rear face had an opening of dimensions 2750×360
421 mm (see Figure 12-b) in its lower part used for air supply. It allows also the ignition of the
422 wood stacks.



a) Front opening

b) Rear opening

Figure 12. Test furnace before implementation of balconies

423 The fire load in the furnace consists of two wood stacks of 300 kg composed in the same way as
424 for a LEPiR II test. This is also consistent with recommendations in the European Standard EN
425 1991-1-2 [19] and the French national annex [20]. Cribs are made of softwood with density 480
426 ± 50 kg/m³ and moisture content between 9% and 15%. Each crib consists in 3 stacked piles.
427 For each crib, the total dimensions are 1000×1000×1800 mm. The cribs are located at 100 mm
428 from the front wall, 200 mm above the ground and at a distance of 200 mm from each other (see
429 Figure 13).



Figure 13. Wood stacks

430 3.2.2. Fire test configuration

431 Due to the dimensions of the test cell, two balconies were connected and characterized for
432 each test:

- 433 • Balconies B1-1 and B1-2 during test n°1,
- 434 • Balconies B2-1 and B2-2 during test n°2.

435 The balconies tested had a range of 1,5 m and were supported by two consoles separated by
436 a distance of 1 m. The beams are composed of IPE 200 of steel grade S275 and of end-plate in
437 S235. The geometry is similar to connections PB-4 and RTPB-4-P tested under monotonic
438 loading (see Figure 2-d) that develop the lower mechanical properties under monotonic loading
439 for the connection of beams of 200 mm height.

440 The beams positioned in front of the opening were significantly exposed to flames while the
441 other one's had a more limited exposition. The consoles were installed on a reinforced concrete
442 lintel of 600 mm high. The connections were fastened to the lintel with four chemical post-
443 installed fasteners M20 HIT-Z-R. The balcony slab was in cellular concrete with a thickness of
444 125 mm and a density of 550 kg/m³. The external thermal insulation of the facade is made of
445 140 mm stone wool with a finishing coat of 15 mm.

446

447

Table 5. Fire test configurations

Test	Balcony	Connection	Beam	Insulating	t_a (mm)	Additional external insulation
Test n°1	B1-1	PB-4	IPE200	No	-	No
	B1-2	RTPB-4-P	IPE200	PVC	20	No
Test n°2	B2-1	RTPB-4-B	IPE200	Plywood	30	No
	B2-2	RTPB-4-P	IPE200	PVC	20	Yes

448

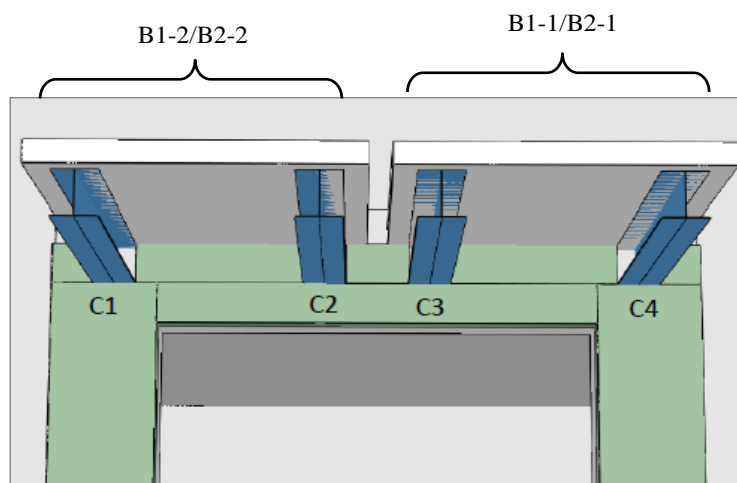
449 The balcony without thermal break, balcony B1-1, constitutes the reference case and is
 450 representative of a balcony commonly used in practice. Then two configurations have been
 451 tested without the addition of external insulation around the connection (right beam in Figure
 452 14-b).



a) Global view



b) Local view



c) Beam labelling

Figure 14. Balconies and test cell before fire test

453 These configurations labelled balcony B1-2 and B2-1 used PVC and plywood,
454 respectively. In practice, this additional insulation is used, the objective of this test was to
455 highlight the impact of this implementation on the performance of the proposed solutions of
456 thermal break. For this reason a last configuration, balcony B2-2, was tested with PVC and
457 additional external insulation around the connection (left beam in Figure 14-b). PVC was
458 chosen because it is more prone to mechanical degradation at high temperature than plywood.
459 The thickness of PVC and plywood layers was equal to 20 and 30 mm, respectively.

460 The load applied on each balcony was close to 590 kg and distributed on the upper face of
461 the cellular concrete slab at the end of the consoles (see Figure 15). **This position is conservative**
462 **as the connection bending moment is overestimated comparatively to a uniformly distributed**
463 **load. This load have been designed considering the fire combination $G + \Psi_1 Q$, with $\Psi_1 = 0,5$.** The
464 total mass per unit area of a balcony, equal to 475 kg/m², is thus equivalent to a permanent load
465 (G) of 300 kg/m² and live loads of 350 kg/m² (Q) according to EN 1991-1-1 [17], in the fire
466 combination $G + \Psi_1 Q$. This loading is typical of balconies with a common concrete slab.



467

468 **Figure 15.** Load applied on the slab of balconies

469 3.2.3. Measurements

470 A total of 25 to 30 thermocouples was positioned on each beam to measure evolution of the
471 temperature during the fire test (see Figure 16). Thermocouples W, X and Y measure the air
472 temperature below the beam. Moreover, temperatures in the test cell were measured and 6
473 thermocouples were placed under the front opening to measure external flames temperature.

474 The temperature of beam web/flange is measured by thermocouples I to T. In addition, the
 475 objective being to study the behavior of the thermal break, thermocouples (from A to H) have
 476 been positioned to measure the temperature at the lintel/layer interface, and layer/end-plate
 477 interface. By the end, thermocouples U and V were placed to measure the external insulation
 478 temperature. In the following, beams were labelled C1 to C4 (see Figure 14-c). Beams C3 and
 479 C4 corresponds to balconies B1-1/B2-1, C1 and C2 to balconies B1-2/B2-2. In addition, a gauge
 480 heat flux was positioned at 3 m in front of the opening, and a video camera recorded the tests.

481 The mechanical behavior of balconies during the fire tests was also a key aspect and
 482 particularly the response of the connections. Vertical displacement at the top of the beams, V_{edge} ,
 483 was measured, as well as the rotation of the connection with an inclinometer, I_c .

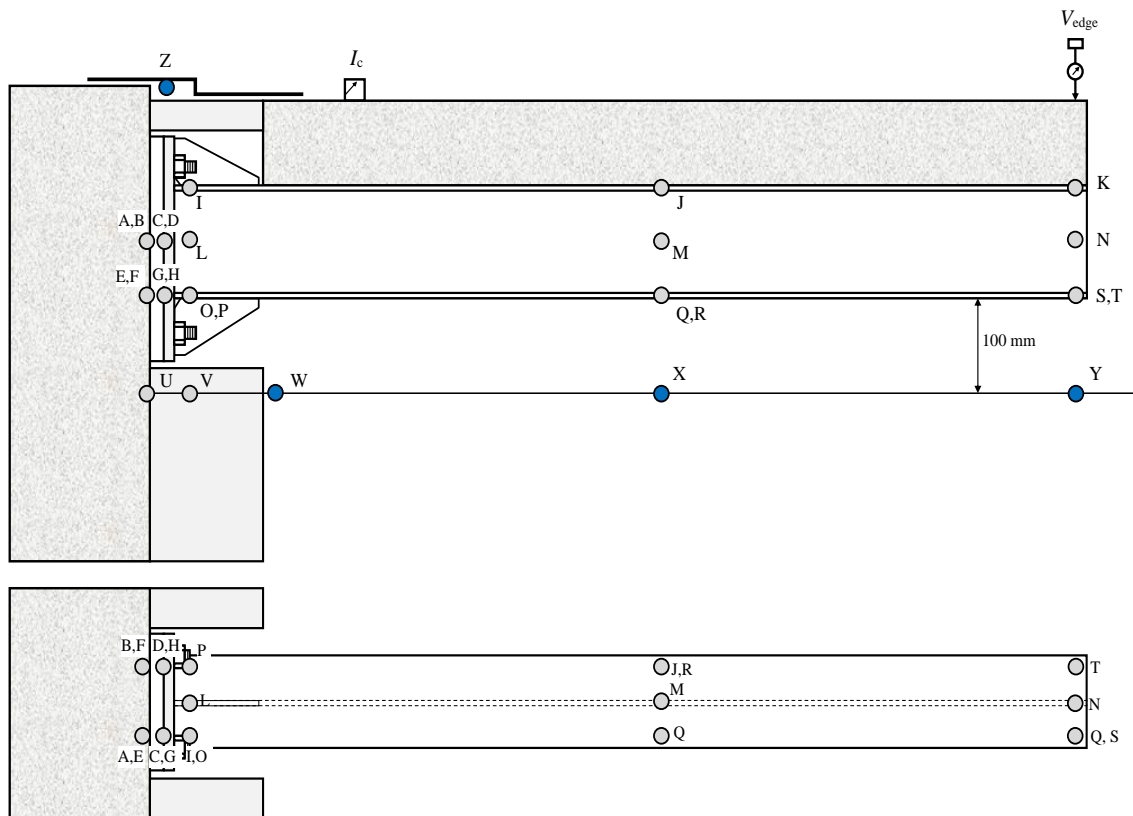


Figure 16. Thermocouples and displacement positions

484

485 3.3. *Fire test results*

486 3.3.1. Test description

487 Each test lasts approximately 45 minutes and proceeds as follows (see Figure 17):

- 488 • Fuel tanks placed under the wood stacks ignited the fire.
- 489 • Smoke exits through the opening of the facade one minute after ignition (see Figure 17-
490 a).
- 491 • After 5 minutes, the first external flames appeared (see Figure 17-b).
- 492 • At 10 minutes, the fire fully developed and the flames reached their maximum length
493 left the opening with an angle of 45° (see Figure 17-c). Thus, these flames directly
494 impact half of the console.
- 495 • From 15 minutes, the fire gradually decreases, and the stacks begin to collapse (see
496 Figure 17-d).
- 497 • At 25 minutes, the wood stacks collapsed and no flame was visible outside (see Figure
498 17-e). Hot gases continue to escape through the opening of the facade, and the
499 temperature under the balcony remains high for another 20 minutes.

500



a) 1st min



b) 5th min

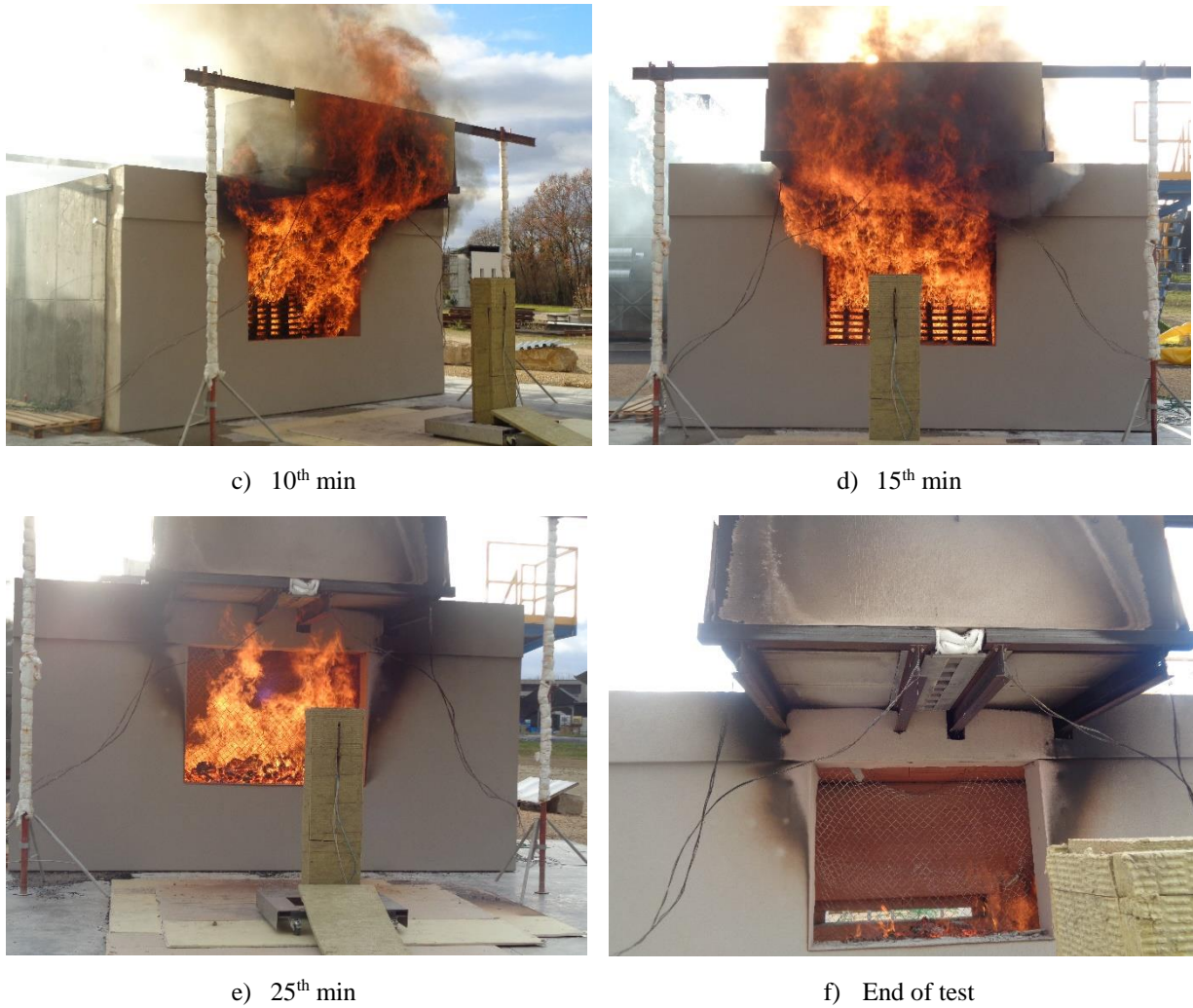


Figure 17. Flame development during test n°2

501 Measurement acquisition was cut off after 45-50 minutes. There was no longer any visible
 502 flame above the collapsed wood stacks. For the two tests, the stability of balconies was ensured.
 503 However, the vertical displacement of balcony B1-2 equal to 144 mm was substantial
 504 comparatively to other configurations.

505 The weather conditions are quite different during the two tests (see Table 6) and this explains
 506 the difference concerning the maximal temperature of gases measured (see section 3.3.2).

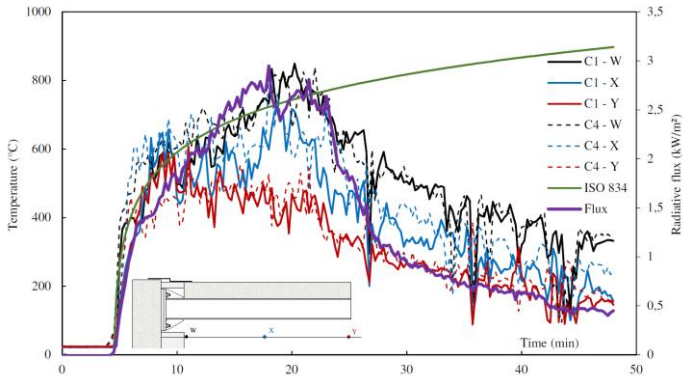
507 **Table 6.** Weather parameters

Test	Temperature °C	Air Humidity %	Wind km/h
Test n°1	22	45	15
Test n°2	9	73	23

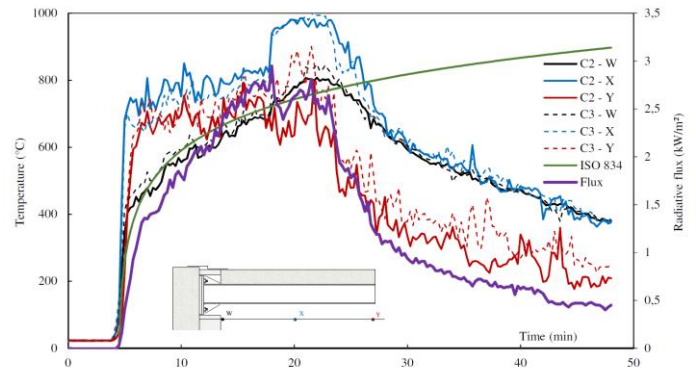
508 3.3.2. Global temperature evolution

509 The temperature in the test cell reached 900° when the fire fully developed. The temperature
510 of flames and gases through the front opening was close to 800-900°C under the consoles
511 directly above the opening (Thermocouples, or Tc, W, X, Y) C2 and C3. Temperatures were
512 logically higher under those beams (C2 and C3) than under the C1 and C4 external beams,
513 which were less exposed (see Figure 18). **These temperatures are quite close to that obtained**
514 **with ISO 834 until 25 min particularly during the first test.**

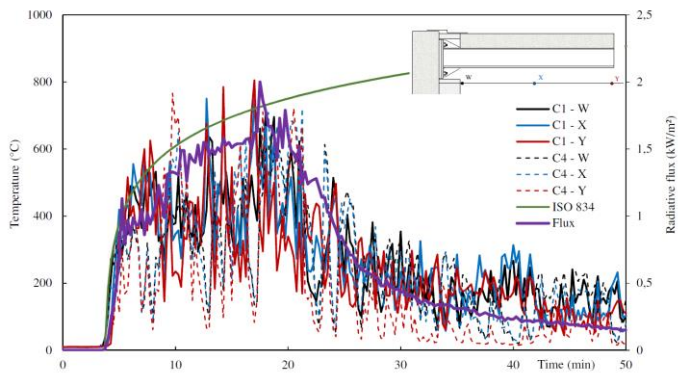
515 In the first part of tests, gas temperature close to the facade (Tc W) is lower than at mid-
516 range and at the extremity (Tc X and Y). This phenomenon is due to the flame tilting through
517 the opening (see Figure 19). At 18 minutes, a sharply increase of the temperature is measured
518 up to 25 minutes for the two tests and particularly test n°1. This phenomenon may be due to the
519 collapse of the wood stacks which causes momentarily a greater heat release, and therefore
520 higher temperatures under the consoles C2 and C3. The temperature in the external insulation of
521 the facade above the opening increases progressively during the test. It reaches 600 °C in the
522 heart of the external insulation (thermocouple Tc V), but only 100 °C at the interface between
523 the insulation and the concrete support (Tc U). These measures highlight the importance of
524 external insulation and its proper implementation around beams, even in case of exceptional fire
525 situation.



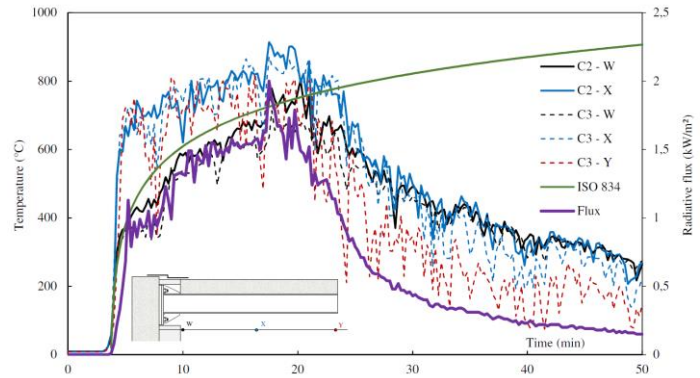
a) Test n°1 : External beams C1 and C4



b) Test n°1 : Internal beams C2 and C3



c) Test n°2 : External beams C1 and C4



d) Test n°2 : Internal beams C2 and C3

526

Figure 18. Gas temperature of flames through the opening (T_c W, X, Y)

527



Figure 19. Flames through the opening (test n°1 – 17 min)

528

The central beams located directly above the opening were the most exposed. In the same

529

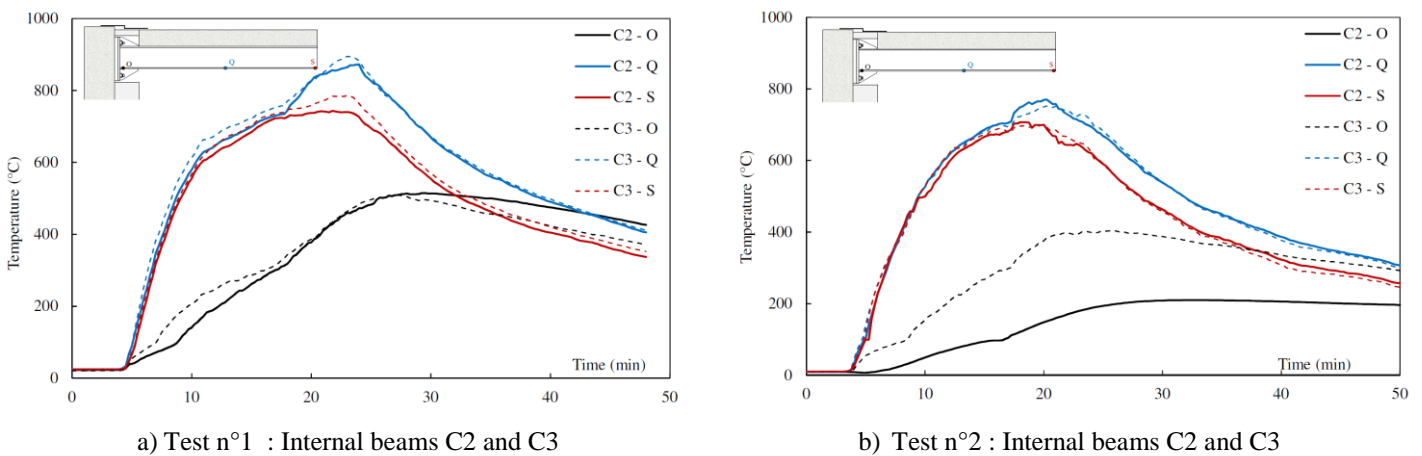
way as for gas temperatures, the temperature of steel section closest to the facade was lower

530

than the temperature at mid-range or at the beam extremity, namely 400-500°C (T_c O) versus

531 700-900°C (Tc Q, S). The highest temperature is at mid-range (Tc Q). At mid-range and at the
 532 extremity, the steel temperatures follow the gas temperature (see Figure 20). The maximum
 533 temperature of the end-plate connections (Tc O) was around 500 °C when they were exposed to
 534 flames and only 200°C for the end-plate connections protected by the additional external
 535 insulation (Test n°2, beam C2 in Figure 20-b).

536 The temperatures were homogenous through the steel beam section. Web and lower flange
 537 temperatures were very close, the temperature of the upper flange is slightly lower
 538 (approximately 100°C) because the concrete slab slows the heating.



539 **Figure 20.** Lower flange temperature at facade (Tc O), mid-range (Tc Q) and extremity (Tc S)

540 3.3.3. Connection fire behaviour

541 The evolution of the temperatures measured at the interface between the lintel and the
 542 intermediate layer (Tc E and F), and between the layer and the end-plate (Tc G and H) is
 543 depicted in Figure 21. For connections without intermediate layer, the temperature of the
 544 interface between the end-plate and the lintel is only provided (Tc E and F). The maximal
 545 temperature measured at interfaces as well as the maximal displacement are also added in Table
 546 7. The maximal air temperature measured by Thermocouple W is also added.

547
 548
 549
 550

551

Table 7. Maximal temperatures and displacements

Test	Balcony	Intermediate layer	Additional external insulation	Air temperature (°C)	Layer/endplate temperature (°C)	Lintel/layer temperature (°C)	V _{max} (mm)
Test n°1	B1-1	No	No	848		375	58
	B1-2	PVC - 20 mm	No	807	449	281	144,6
Test n°2	B2-1	Plywood – 30 mm	No	713	320	51	43,4
	B2-2	PVC – 20 mm	Yes	804	164	32	58,1

552

553 By comparing balconies B1-2 and B2-2, the influence of the additional external insulation is
 554 clearly highlighted: the decrease between the air temperature (T_c W) and the end-
 555 plate/intermediate layer interface temperature (T_c G, H) goes from 358°C (807°C-449°C)
 556 without the additional external insulation, to 640°C (804°C-164°C) with additional external
 557 insulation. For balcony B2-2, with external thermal insulation, the temperature at the interface
 558 between the end-plate and the layer is lower than that between the layer and the lintel for
 559 balcony B1-2 that do not use external insulation.

560 The presence of intermediate layer plays also a role in the temperature decrease at the lintel
 561 interface (T_c E, F). In the reference case (without thermal break), the maximal end-plate/lintel
 562 temperature interface is equal to 375°C. The plywood intermediate layer allows a decrease of
 563 this temperature to 50°C even without additional external insulation around the connection.
 564 With PVC intermediate layer and additional external insulation around the connection, this
 565 temperature decreases to 30°C. The interface temperatures continues to increase very slowly
 566 after the end of the fire pick because of thermal conductivity (see Figure 21-d).

567 However for balcony B2-1, with PVC and no external insulation, this decrease is more
 568 limited, the maximal temperature of the layer/lintel interface was around 280 °C mainly because
 569 the PVC intermediate layer was partially melted and consumed (see Figure 22-a and Figure 23-
 570 a/b). With the deteriorating of the intermediate layer, hot gases were introduced and heated the
 571 lintel (see Figure 22-a). This partial consumption, non uniform, induces the rotation of the end-
 572 plate and as a consequence a significant vertical displacement equal to 144 mm at the top of the

573 beam, against 44-58 mm for the other configurations. However, this deformation that causes a
 574 rotation of the end-plate did not affect the overall stability of the attached beam.

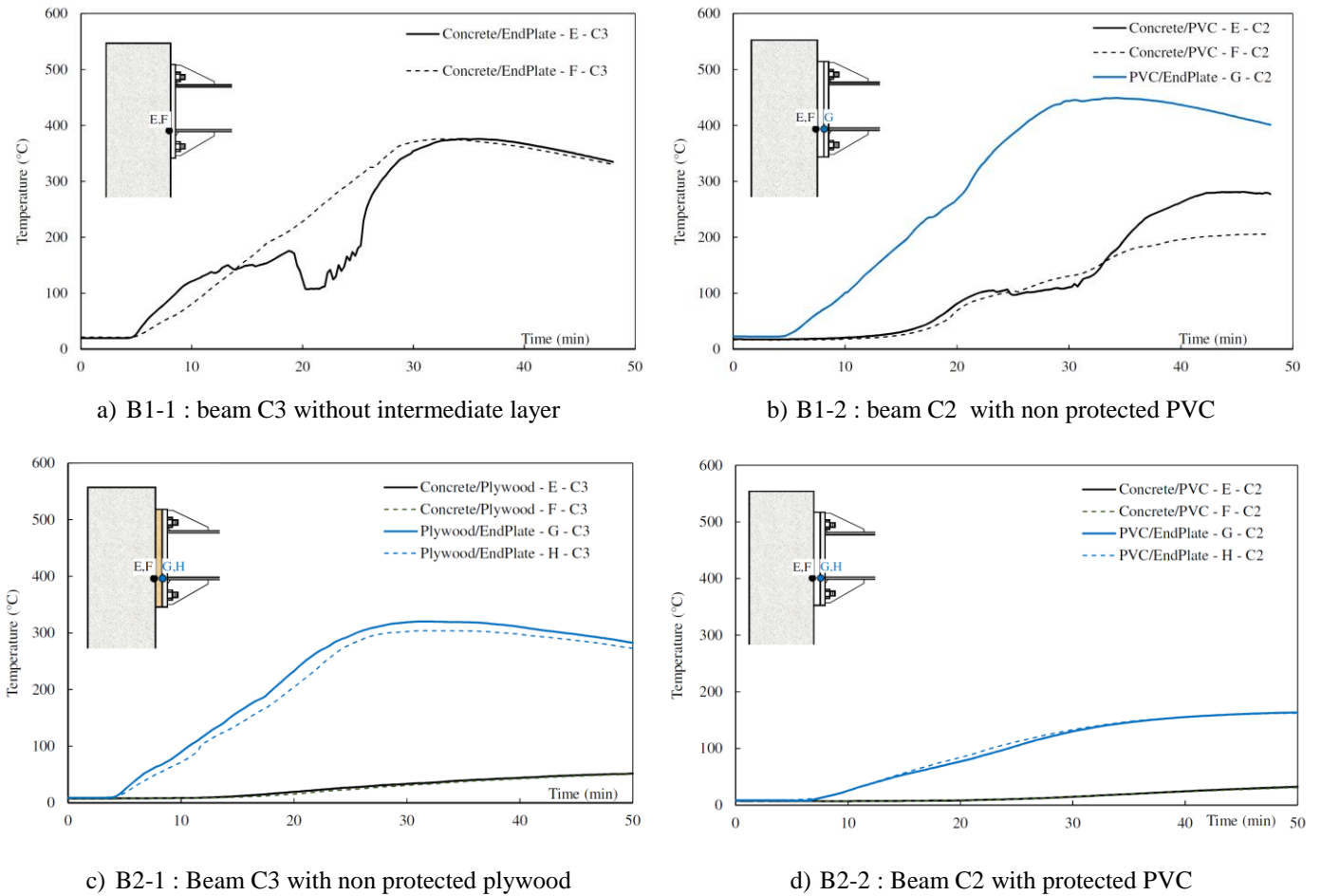


Figure 21. Temperatures of layer/end-plate (T_c G, H) and layer/lintel (T_c E, F) interfaces

575 The plywood layer started to burn (see Figure 23-c) but only at the interface between the
 576 end-plate and the layer (see Figure 22-b). This layer keep its thickness and the maximal
 577 displacement at the top of the balcony does not exceed 43,4 mm. This balcony was the less
 578 impacted by the fire despite the absence of additional external insulation around the connection.



a) PVC, no External Insulation



b) Plywood, no external insulation



c) PVC and external insulation

Figure 22. *Connections after fire test*

579 For balcony B2-2 with PVC and additional external insulation, the PVC did not burn (see
580 Figure 23-d) but its thickness decrease in the compressive area (see Figure 22-c) due to a
581 reduction of its mechanical properties at high temperatures (around 160 °C). The nut of bottom
582 anchor bolts is no more in contact with the end-plate. This reduction of the PVC thickness in the
583 compressive area also induce a global rotation of the connection and maximal displacement of
584 the edge of the balcony of 58 mm. A similar displacement has been obtained for balcony B1-1
585 without thermal break and additional external insulation around the connection. A thermal break
586 using PVC and additional external insulation is able to develop a fire behaviour close to that of
587 a conventional connection (without intermediate layer and additional external insulation). The
588 solution with plywood layer but without additional external insulation permit to draw the same
589 conclusions.



a) Test B1-2 : PVC surface in contact with the lintel



b) Test B1-2 : PVC surface in contact with the end-plate



c) Test B2-1 : Plywood surface in contact with the end-plate



d) Test B2-2 : PVC layer

Figure 23. *Intermediate layer after fire test*

590 The evolution of the displacement measured at the top of the balcony is presented in Figure
 591 24-a. In addition, this displacement is given in Figure 24-b in function of the temperature
 592 measured at the interface between the end-plate and the layer or concrete for B1-1. Except for
 593 balcony B1-2, with PVC and no external insulation, the maximal displacement is around 40-60
 594 mm. For balconies without thermal break, B1-1, and PVC but without additional external
 595 insulation, B1-2, the increase of the displacement is quite fast and tends to an asymptote after
 596 20-25 min. For the two other balconies, the increase of the displacement is slower as well as the
 597 temperature at the interface between the layer and the end-plate (see Figure 21). We can clearly
 598 observe with Figure 24-b that for the two first specimens, the displacement is stabilized for a
 599 temperature of 200-300 °C. When the temperature decrease, the displacement is not modified.
 600 On the contrary for plywood, temperatures greater than 300° do not affect strongly the
 601 displacement. The displacement continues to increase even after the temperature decrease
 602 because of the plywood burning. For balconies with PVC and additional external insulation, the
 603 temperature is lower than 200 °C and the combustion didn't start as well as major rotation of the
 604 connection. Nevertheless, the displacement-temperature curves obtained in presence of PVC
 605 layer with and without additional external insulation are almost parallel (see Figure 24-b).

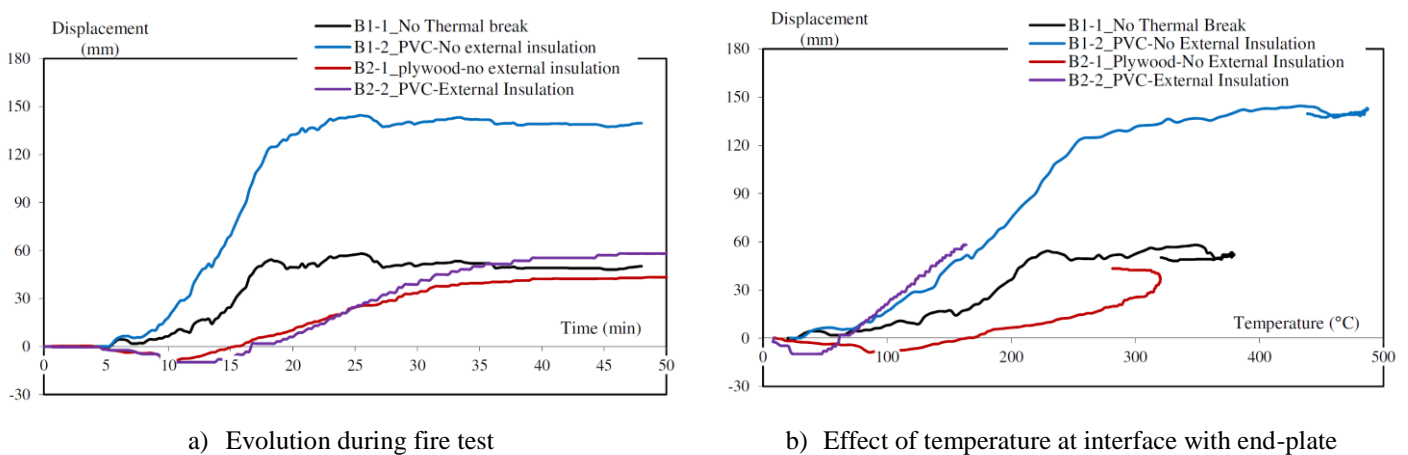


Figure 24. Vertical displacement of the edge of the balcony

606 3.4. Summary on fire behaviour

607 The in situ fire tests highlighted the impact of the different connections configurations,
 608 previously tested at ambient temperature, on the global behaviour of the balcony. The balcony

609 directly attached to the concrete support without additional external insulation behaves very
610 well under this fire scenario and the maximal displacement was limited to 58 mm. The addition
611 of intermediate layer composed of PVC or plywood didn't affect this stability even in absence
612 of external thermal insulation around the connection. Without additional external insulation, the
613 PVC layer burns partially loses its thickness that generates a global rotation of the connection
614 and an important displacement at the top of the balcony of 144 mm. The plywood starts to burn
615 but keeps its integrity, the rotation was clearly more limited and as a consequence the top
616 displacement was equal to 44 mm. This latter was lower than that obtained with the
617 conventional connection in absence of thermal break but also of additional external insulation.
618 The addition of external insulation around the connection permits to avoid PVC burning and
619 limit significantly the global rotation of the connection. The use of intermediate layer such as
620 plywood isolates strongly the lintel and reduces the temperature, this phenomena can be
621 amplified by the addition of external insulation around the connection.

622 ***4. Conclusions***

623 This paper gives the results of a study assessing the mechanical and fire behaviour of simple
624 solutions of thermal breaks for external steel structures (balconies, solar panels ...) attached on
625 a concrete facade with External Thermal Insulation. The studied solutions are composed of a
626 PVC or plywood layer implemented between attached steel structures and support. The main
627 advantages of these solutions compared to manufactured products proposed by industrial
628 suppliers are their reduced costs and ease of erection.

629 Experimental tests performed on the proposed solutions loaded monotonically under bending
630 moment highlighted that the mechanical properties of these solutions are close to that of
631 standard connections directly fastened to the concrete support. The decrease of the ultimate
632 bending resistance is hence quite limited. On the contrary, the plastic bending resistance
633 decrease substantially due to the early yielding of the insulating layer in the compressive area.
634 The same phenomena was also observed for thermal breaks attached to a steel structure [13].

635 The secant rotational stiffness decreases due to the addition of intermediate layer but the values
636 obtained, greater than 5000 kNm/rad for connections of beam of 200 mm depth, were largely
637 sufficient for the connection of a balcony. For post-installed mechanical/chemical fasteners, the
638 addition of insulating layer did not modify the rotational stiffness but decrease slightly the
639 ultimate bending resistance. Connections with bolted embedded end-plates were able to develop
640 large bending resistances compared to post-installed fasteners.

641 In order to evaluate the influence of the thermal breaks on the behaviour of a balcony under
642 fire, external flame experiments were carried out. These tests have demonstrated that the studied
643 solutions of thermal break does not affect the stability of the balconies under fire. These results
644 remain valid even in absence of external insulation around the connections leading to a direct
645 exposure of the thermal break to flames. Plywood starts to burn but keeps its integrity and initial
646 thickness. On the contrary, the PVC of non protected connections burns, generates a global
647 rotation of the connection and thus a large displacement of the balcony but without collapse.
648 With additional external thermal insulation, temperature of the assemblies remains low and
649 avoids PVC burning. Thus, a correct implementation of the proposed solution (*ie* with
650 additional external thermal insulation around the connection) is strongly required.

651 The experimental campaigns performed on the proposed solutions of thermal attached to
652 steel [13] and concrete supports highlighted that the mechanical and fire behaviours are close to
653 that of conventional connections directly fastened to the support. Nevertheless, design methods
654 are necessary to ease the application of these solutions in practice. The calculation of the
655 rotational stiffness as well as the plastic/ultimate bending resistances are of primary importance.
656 The contact distribution being different from standard connections in the tensile and
657 compressive area, new analytical models can potentially be developed. By the end, the long-
658 term effect should be considered particularly for the evaluation of the rotational stiffness.

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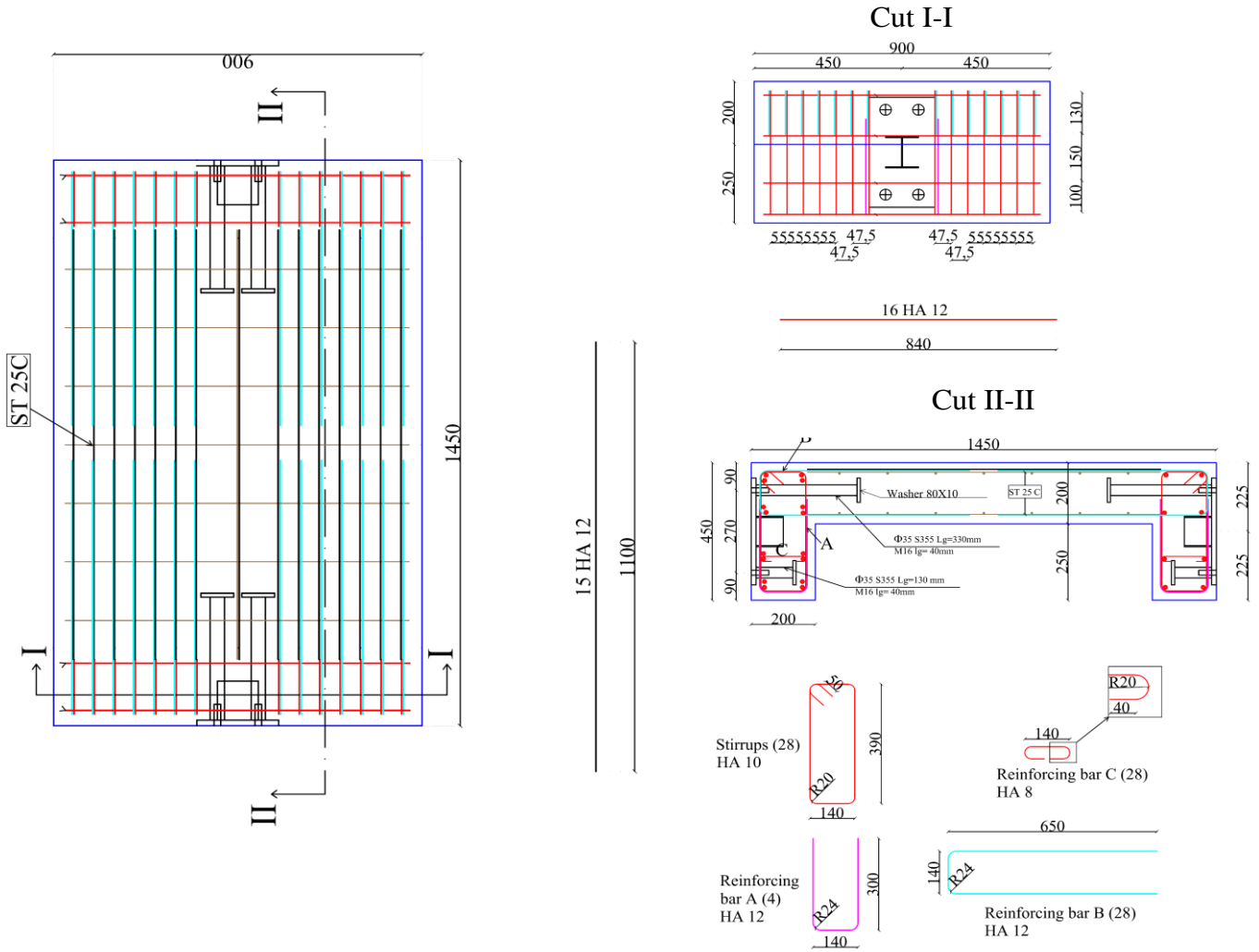


Figure A.1. Reinforcement of concrete Block B1

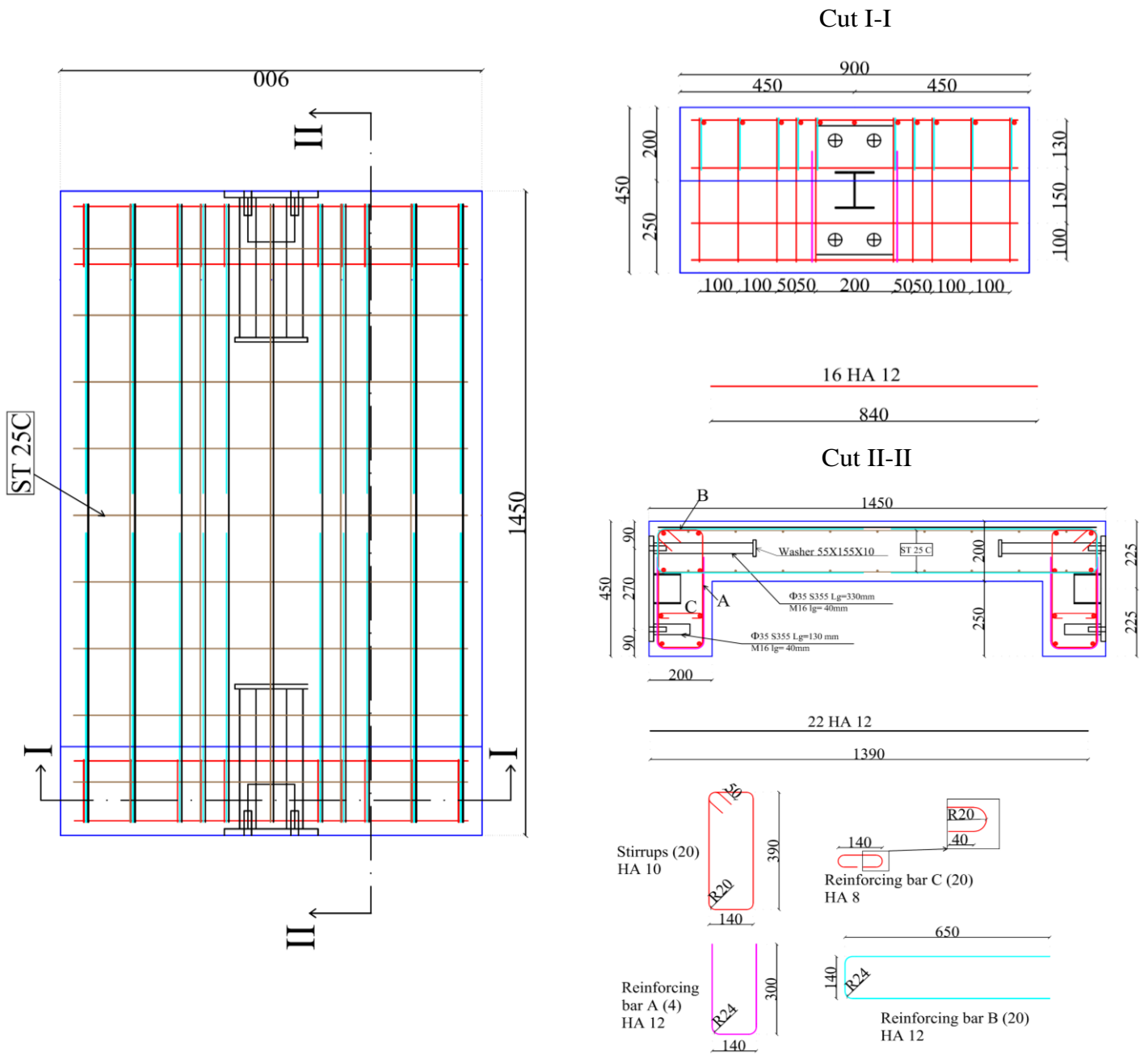


Figure A.2. Reinforcement of concrete Block B3s