DESIGN PROCEDURES FOR TYPICAL CLADDING PANEL CONNECTIONS IN RC PRECAST BUILDINGS

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Abstract

Precast buildings represent a considerable part of industrial and commercial infrastructure in Europe. The potential losses in the case of the strong earthquakes are therefore large. This was confirmed during the recent earthquakes. For example, during the earthquakes in Emilia-Romagna in Italy, RC precast buildings were amongst most vulnerable types of structures. Damage was observed on structural as well as on non-structural components. Particularly vulnerable appeared to be the cladding panels and their connections with the main structural system.

For instance, Bournas et al. [1] observed the collapse of cladding panels in 75 % of all precast buildings in the area. Libeatore et al. [2] observed severe damage of cladding panels in 50 % of 34 surveyed industrial buildings. It was reported by several authors ([3] – [8]) that the cladding-to-structure connections were designed only for seismic forces acting in the direction perpendicular to the panel plane, which are related to the local mass of the panels, and for low out-of-plane horizontal actions such as wind loads. The prevailing opinion was that the main cause for the failure of the cladding-to-structure connections was their inadequate resistance to the horizontal seismic actions in the panel plane.

The cladding panel connections, which are typically used in RC precast buildings in Europe, have been analyzed experimentally and analytically within recently concluded European project SAFECLADDING [11]. Vertical as well as horizontal panels were addressed.

The typical cladding panel connections were mostly designed to be used in the non-seismic regions. Even though a little was known about their seismic response they were extensively used in the seismic regions. The basic mechanisms of their seismic response were identified for the first time within the SAFECLADDING project. Contrary to the prevailing opinion that cladding panels are most vulnerable in their out-of-plane direction, it was proved that the most critical components – connections typically fail due to the seismic effects in the plane of the panels. This observation was in a good agreement with the damage observed after recent earthquakes.

Based on the experimental and analytical studies of complex response of typical cladding panel connections, appropriate robust engineering numerical models were defined, which can be used in the design practice. The design procedures were proposed, and the expressions, which can be used to estimate their displacement and strength capacity, were derived.

Keywords: RC precast buildings, Cladding panels, Connections, Experiments, Numerical models, Design rules

1. Introduction

Precast buildings represent a considerable part of industrial and commercial infrastructure in Europe. The potential losses in the case of the strong earthquakes are therefore large. This was confirmed during the recent earthquakes. For example, during the earthquakes in Emilia-Romagna in Italy in 2012, RC precast buildings were amongst most vulnerable types of structures. Damage was observed on structural as well as on non-structural components. Particularly vulnerable appeared to be the cladding panels and their connections with the main structural system.

For instance, Bournas et al. [1] observed the collapse of cladding panels in 75 % of all precast buildings in the area. Libeatore et al. [2] observed severe damage of cladding panels in 50 % of 34 surveyed industrial buildings. It was reported by several authors ([3] – [8]) that the cladding-to-structure connections were designed only for seismic forces acting in the direction perpendicular to the panel plane, which are related to the local mass of the panels, and for low out-of-plane horizontal actions such as wind loads. The prevailing opinion was that the main cause for the failure of the cladding-to-structure connections was their inadequate resistance to the horizontal seismic actions in the panel plane.

According to [9] - [10], a high flexibility of typical precast industrial buildings led to displacement incompatibility between structural elements and precast panels in the plane of panels, which caused several failures of cladding-to-structure connections. The authors therefore suggest that such connections should possess adequate ductility to accommodate the seismic displacement demand.

Taking into account the observed damage of cladding panels' connections and lessons learnt from the recent earthquakes the cladding panel connections, which are typically used in RC precast buildings in Europe, have been analyzed experimentally and analytically within recently concluded European project SAFECLADDING [11]. Vertical as well as horizontal panels were addressed. The investigated connections were mostly designed to be used in the non-seismic regions. Even though a little was known about their seismic response they were extensively used in the seismic regions. The investigated connections are briefly overviewed in Section 2.

Based on the extensive experimental research, reported in Section 3, the basic mechanisms of the seismic response of typical panels' connections were identified for the first time (see Section 4). Taking into account the observed response, the appropriate numerical models, which can be used in the design practice, were developed and the design procedure proposed (Section 5).

2. Investigated connections

Three most common types of connections: hammer-head strap, angle, and cantilever connections were tested. They are schematically presented in Figure 1:

1) Hammer-head strap connections consist of two channels. One channel is mounted in the panel and the second one in the beam. They are connected with the hammer-head strap, which is bolted into the channel placed in the beam (see Fig. 1a). This type of connections is used to link vertical as well as horizontal panels to the main structural system of RC precast buildings.

2) Angle connections also consist of two channels, mounted in the panel and the beam/column. These two channels are tied together with bolted angles as it is shown in Figure 1b. This connection is used for both types of panels, horizontal as well as vertical panels.

3) The specific type of connection, which is mainly used to attach the horizontal panels to the main structure of RC industrial buildings, is a cantilever connection, presented in Figure 1c.

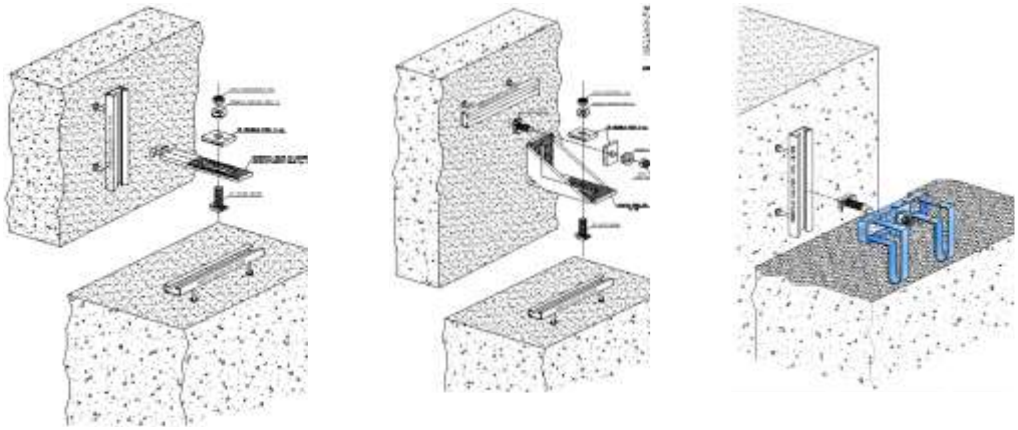


Fig. 1 Typical connections of cladding panels: a) Hammer-head tie connections, b) Angle connections, c) Cantilever connections

3. Experiments

All types of connections were examined experimentally by means of the series of cyclic tests. The test setup was similar in all cases (Fig. 2). All connections were mounted to the foundation beam and panels. The inverted T foundation beam (see Fig. 2a) was fixed to the lab's floor (see Fig. 2b and Fig. 2e). Panel was placed at one side of the foundation beam. It was connected to the actuator (see Fig. 2d) and mounted on rollers in order to be able to slide it in parallel to the foundation beam. Special attention was devoted to the construction of rollers (see Fig. 2c) in order to reduce the amount of friction to a minimum possible level. A construction, using special ball bearings, provided movements of the panels with almost no friction. The force and displacement capacity of the used actuator were 250 kN and ± 20 cm, respectively. In order to optimize the execution of the tests, the same foundation beam and panels were used to examine the response of several connections.

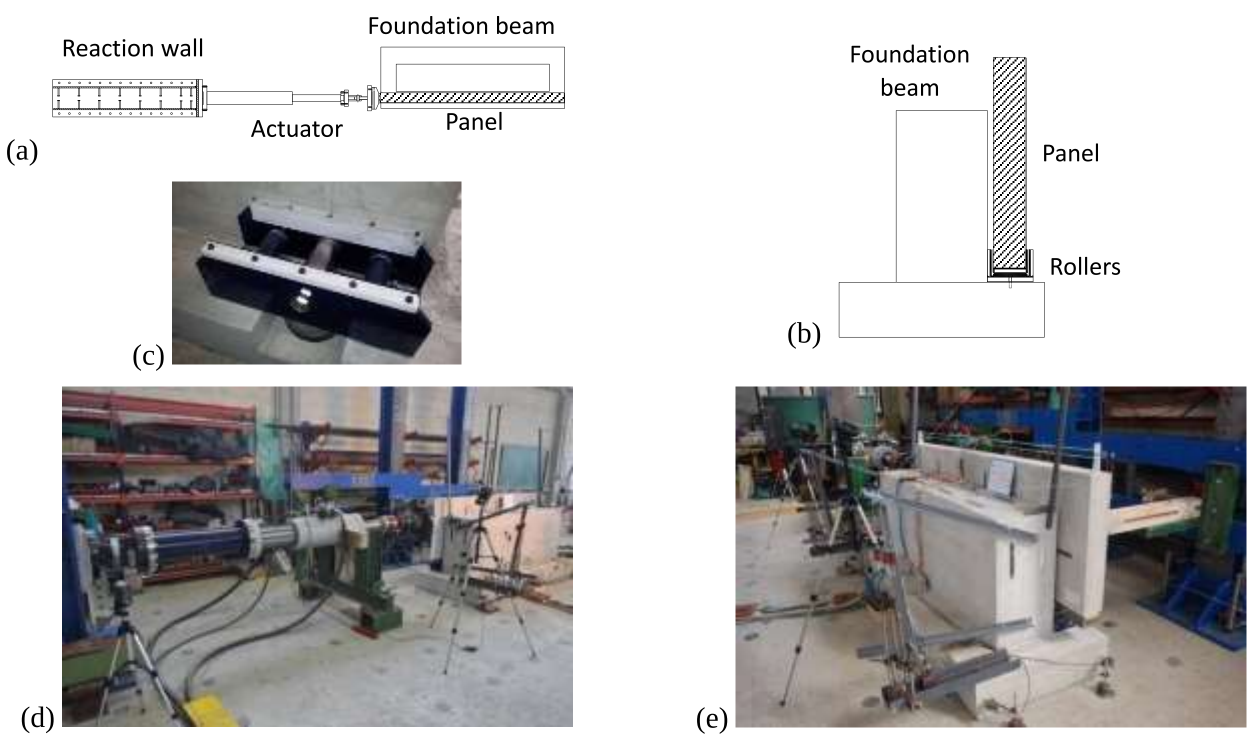


Fig. 2 Set-up of the tests: (a) plane-view, (b) side-view, (c) roller bearings, (d) the actuator, connected to the panel, (e) side-view of the test set-up

3.1 The testing protocol

Most of the tests were cyclic. The load was applied in the horizontal direction in parallel to the longitudinal axis of the panel. The direction of the load was perpendicular to the channel mounted in the panel and perpendicular to the connections. The load was applied according to the recommendation of the FEMA 461 [12]. The amplitudes were increased exponentially. Two full cycles per amplitude were applied (see an example of the scheme, presented in Fig. 3).

Beside uniaxial, several biaxial experiments were performed on the hammer-head strap connections. The specimens were subjected simultaneously to variable cyclic load in the direction parallel to the longitudinal axis of the panel and two invariable horizontal forces perpendicular to the plane of the panel.

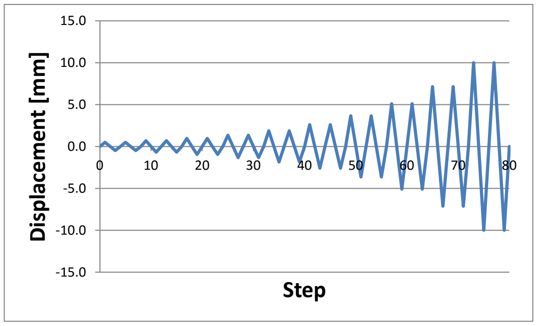


Fig. 3 An example of the testing protocol

3.2 Hammer-head strap connections

The hammer-head strap was in all cases the same (TA 135-GV length 210 mm – see Fig. 4). In all cases the strap was bolted to the channel in the beam, using bolts HS 40/22 M16, with diameter of 16 mm. Two types of channels were investigated: a) Hot rolled HTA 40/22, and b) Cold formed HTA 40/23. In all tests the length of the channels was 25 cm and they were anchored to the concrete using two anchors. Altogether 16 tests were performed. More details about the performed tests can be found in [13] and [14].

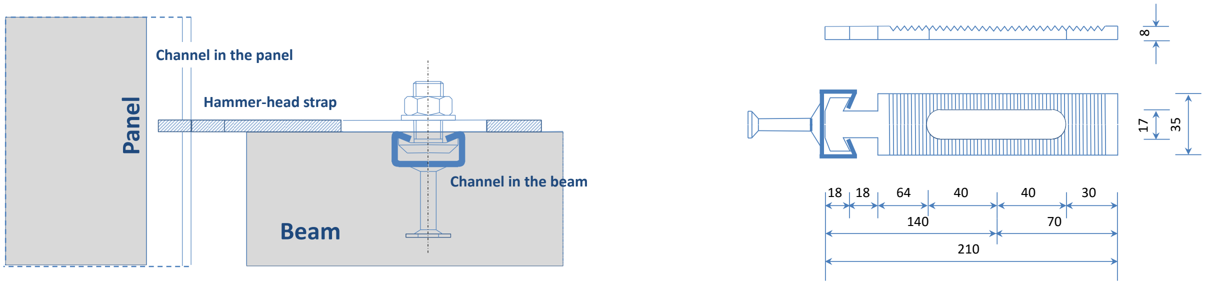


Fig. 4 The hammer-head strap connection

3.3 Angle connections

The examined angle connections consisted of channels mounted in the panel and the foundation beam, which were tied together with bolted angles. The same channels HTA channels 40/25 were used in the beams and panels. Bolts HS 40/22 M16 were used to attach the angles to the channels. More details about the tested angles are presented in Fig. 5. Four cyclic tests of these connections were performed. More details about the performed tests can be found in [14].

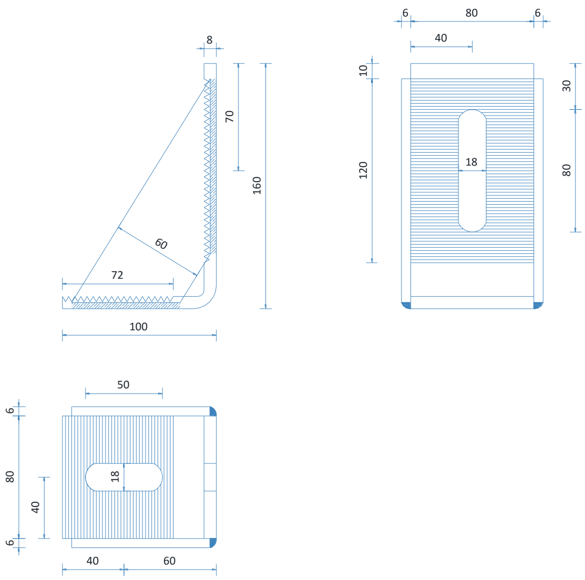


Fig. 5 The angle connection

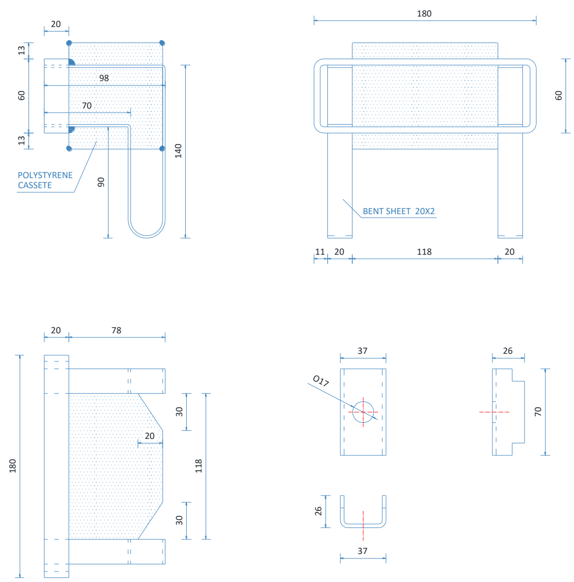


Fig. 6 The cantilever connections

3.4 Cantilever connections

Cantilever connections consisted of the channel mounted in the panel and special element (see Fig. 6) mounted in the foundation beam. They were tied together with bolt as it is shown in Fig. 6. The HTA channels 40/23 and bolts HS 40/22 M16 were used. More details about the tested connections can be found in Fig. 6. Two cyclic tests were performed. In each of these two tests two connections were examined. More details about the performed tests can be found in [14].

4. Results of experiments and the response mechanisms

4.1 Hammer-head strap connections

The main observed failure mechanism of the hammer-head strap connections, when subjected to a shear loading, is presented in Fig. 7. For the sake of simplicity, only the case where stronger (hot-rolled) channels were used is first considered. As will be explained later, the failure mechanism of the connections with cold-formed channels is somewhat different, but the overall response is, in general, similar.

At relatively low shear loads (0.5 - 1 kN), the strap rotates around the bolt, as shown in Fig. 7 (from Stage 1 to Stage 2). At a displacement of 2 - 3cm, the head of the strap becomes stuck inside the channel in the panel. Consequently, the stiffness of the connection increases significantly (Fig. 7, Stage 2). At a shear load of approximately 3 kN (Fig. 7, Stage 3), yielding of the strap occurs in the narrow part just below its head (further on in the paper this will be referred to as the neck of the strap). Finally, the connection fails due to flexural failure of the neck of the strap (Fig. 7, Stage 4).

When cold-formed channels were used instead of hot-rolled ones, neither plastic deformations nor failure occurred in the strap, but took place in the channel. However, the basic features of the response and the corresponding hysteretic loops are similar to those obtained in the case of the hot-rolled channels (see Fig. 8). More details about the cycylic response of investigated connections can be found in [13] and [14].

In some cases, the gap between the beam and panel closed before failure of the strap or of the channel. In all tests of hammer-head strap connections it was observed that due to the large rotations the strap pulled the panel more and more against the beam, whereas the relative shear displacement increased. If the gap between the panel and the beam was not large enough to accommodate the relative displacements between the panel and the beam perpendicular to the plane of the panel, the gap closed and friction between the panel and the beam was activated. Consequently, the stiffness of the connection increased. The strength of the connection was not affected since the weakest link of the connection was still either the hammer-head strap or the channel, which failed at the same force level as in the case when the gap was not closed. It is therefore suggested that the gap should be wide enough so that the displacement capacity of such connections can be utilized. For more details, see [13].

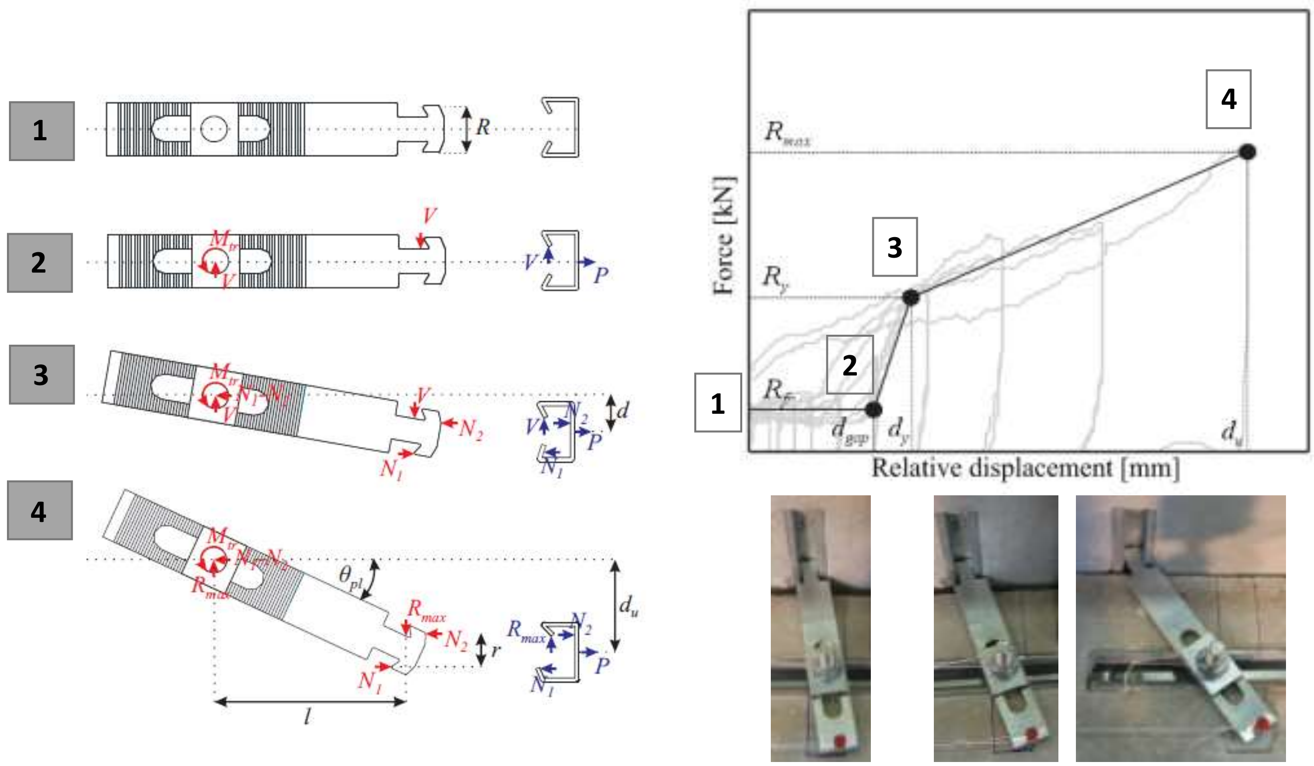


Fig. 7 The response mechanism of the hammer-head strap connections

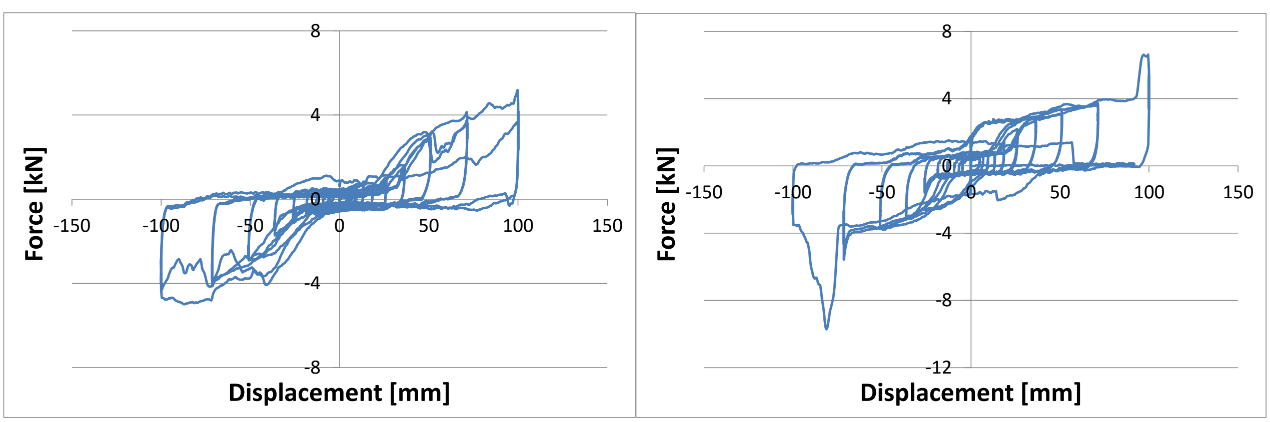
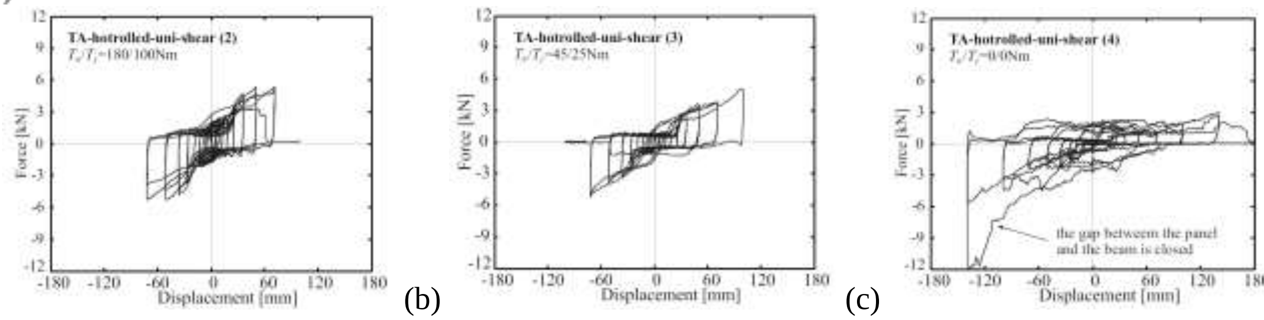


Fig. 8. Typical hysteretic response of hammer-head strap connections with: a) hot rolled channels, b) cold formed channels

The influence of the tightening torque (which is typically applied to the bolt, tightening the strap to the channel in the beam) on the global response of the investigated connections is presented in Fig. 9 by means of a comparison between three different force-displacement diagrams. In the first case (Fig. 9, left), a tightening torque of T = 45 Nm, which is typically chosen in practice, was applied, while in the second and third cases (Fig. 9, central and right) tightening torques of T = 180 Nm and T = 0 Nm were, respectively, applied.



(a) (b) (c)

Fig. 9 The influence of the tightening torque T to the response: a) T = 45 Nm, b= T = 180 Nm, c) T = 0 Nm

A comparison between the presented diagrams shows that the tightening torque is indeed an important parameter influencing the global response of such connections. If the responses of the test specimens with T = 45 Nm and T = 180 Nm is compared, it can be seen that a higher tightening torque results in more open hysteretic loops. This means that the energy dissipation efficiency is increased. However, the failure mechanism is not affected by the increased torque moment. On the other hand, when the tightening torque is very small (almost 0 Nm), then the response of the connection is somehow different to that of the test specimens where the torque was applied to the bolt. In the experiments, where the tightening torque was equal to 0 Nm, the strap was sliding along the channel inside the foundation beam. This resulted in a larger displacement capacity of the connection. This can be seen from the response hysteresis of test specimen with T = 0 Nm, which is shown in Fig. 9 (right). When the strap with the bolt hit the concrete at the end of the channel, the stiffness of the connection increased. From this point on, the failure mechanism was similar to that of the connections where the torque was appplied.

4.2 Angle connections

Response of the angle connections is less complicated than that of the hammer-head strap connections. Typical hysteretic behavior and the failure of the steel angle connections, when loaded in shear, is schematically presented in Fig. 10.

During small displacement cycles only the rotations of the angle were visible. When the displacements were increased, some deformations of the angle were noticed, however they were remained moderate throughout the experiment. Considerable cracking of the concrete and deformations of the channel, mounted in the panel, were obtained at cycles with moderate displacement levels.

Further increase of displacements resulted in an increase of the damage of the concrete around the channel, mounted in the panel, and an increase of plastic deformations of this channel. Compression forces were induced at one edge of the angle and tension forces were induced in the bolt. Rotations of the angle, around vertical as well as horizontal direction, were also more pronounced when the displacement amplitude was increased.

The failure of the connection typically occurred due to the failure of the channel mounted in the panel. The bolt was pulled out of this channel. There was no considerable sliding of the angle observed. Contrary to the connections with the hammer head straps, the considerable rotations of the angles increased the gap between the beam and the panel, pushing the panel away from the beam.

Stiffness of the steel angle connection is not negligible and it should be taken into the account in the global analysis of the structure before as well as after yielding of the connection. In the most simplified situation, steel angle connections could be considered as pinned joint.

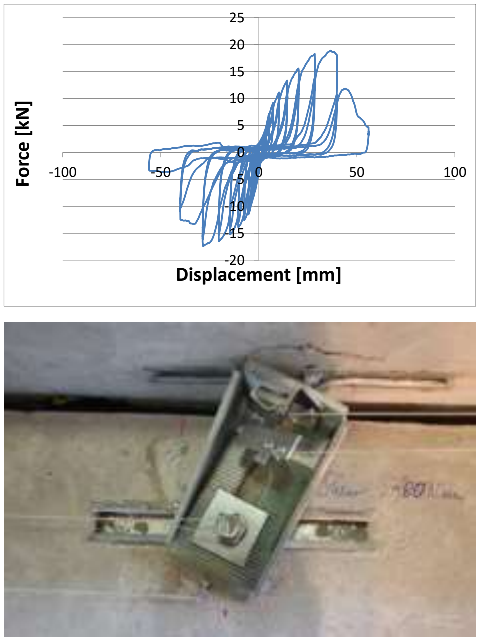


Fig. 10 Cyclic response of angle connections and the typical type of failure

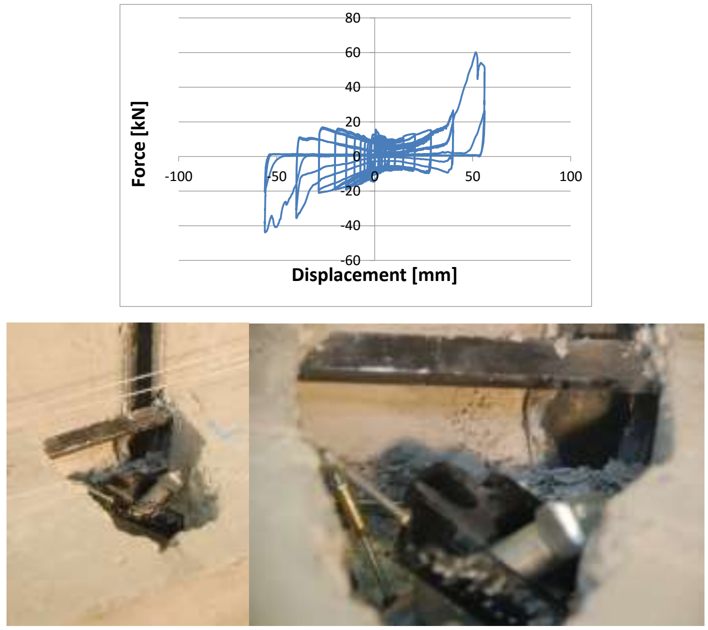


Fig. 11 Cyclic response of cantilever connections and the typical type of failure

4.3 Cantilever connections

Typical cyclic response of cantilever connections is presented in Fig. 11. At very small displacements amplitudes the sliding of the screw was initiated. Consequently, the force in the connections was increasing slowly. When the screw, together with the surrounding steel frame, reached the concrete edge, the force increased abruptly and reached the maximum values between 60 kN – 75 kN. The maximum displacements were around ±60 mm.

When the screws reached the concrete edge, deformations of the channels as well as deformations of the screw were considerably increased. In last cycles deformations of channels and screws were large. Concrete around the channels was considerably damaged. Finally, the screws were pulled out from the channels. Channels failed due to the large deformations of their lips. Contrary to the connections with the hammer head strap, this type of connection increased the gap between the beam and the panel, pushing the panel away from the beam.

5. The numerical models and design procedures

5.1 Hammer-head strap connections

Hysteretic response of a hammer-head strap connection can be modelled combining three basic models: elasto-plastic; gap and hysteretic (Fig. 12a), which are usually included in the majority of the available software for the nonlinear analysis. An example of the use of such a model is presented in Fig. 12b, where analytical response is compared to the experimental one.

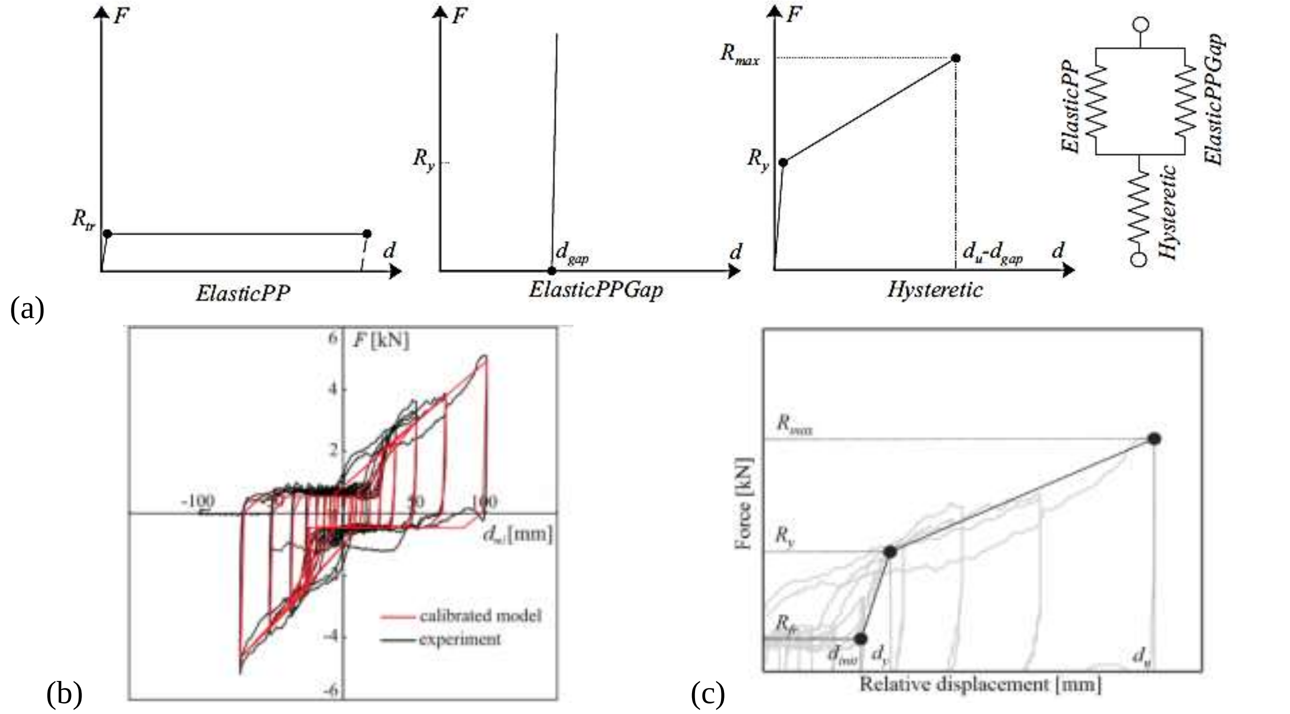


Fig. 12 Cyclic response of hammer-head strap connections: a) the numerical model, b) the hysteretic response, c) characteristic points of the envelope

Hysteretic response envelope of the shear behavior of hammer head strap connections can be idealized as suggested in Fig. 12c. The envelope is characterized by the three characteristic points. In the case of strong channels, they can be evaluated using the following expressions:

$$R_{fr} = \frac{M_{fr}}{L} \qquad d_{init} = \theta_{init}\, L \tag{1}$$

$$R_y = \frac{M_{y,N} + d_y P + M_{fr}}{\sqrt{L^2 - {d_y}^2}} \qquad d_y \approx d_{init} \tag{2}$$

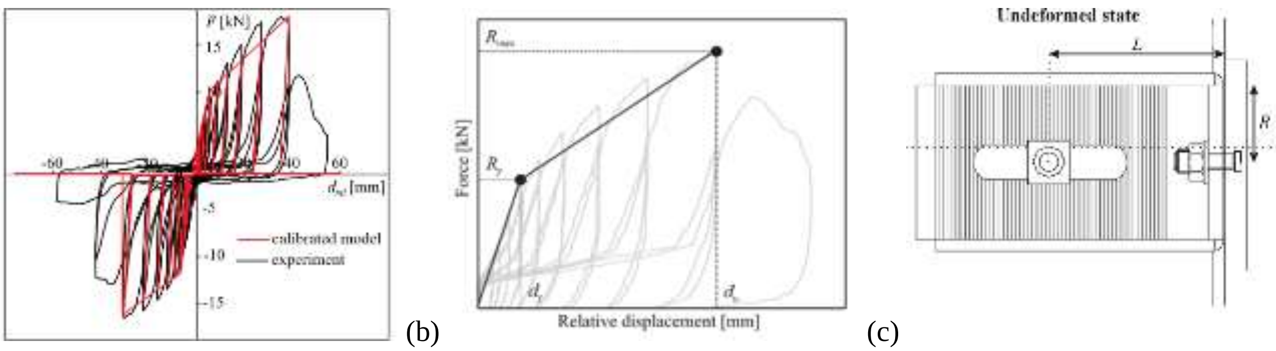
$$R_{max} = \frac{M_{pl,N} + d_u P + M_{fr}}{\sqrt{L^2 - {d_u}^2}} \qquad d_u = (\theta_{init} + \theta_{st})L \tag{3}$$

M_{fr} is the moment in the bolt (can be estimated as the tightening torque), L is the distance between the bolt and the channel mounted in the panel, $M_{y,N}$ is the flexural resistance at yield of the hammer head strap at the narrowing just under the head taking into the account axial force N, $M_{pl,N}$ is the plastic flexural resistance of the hammer head strap at the narrowing just under the head taking into the account axial force N, P is the force in the direction perpendicular to the panel plane, θ_{init} is the rotation of the strap due to the tolerances within the channel, θ_{st} is the rotation of the strap due to the flexural deformations of the strap at the narrowing just under the head, which can be estimated as ultimate curvature multiplied by the length of the narrowing

In the case of weak channels, similar expression can be used to evaluate the envelope. For more details, see [13].

5.2 Angle connections

Hysteretic response of a steel angle connection can be modelled by using a hysteretic model as shown in Fig. 13. These models are included in the majority of the programs for the nonlinear analysis. An example of a use of such model is presented in Fig. 13a, where analytical response is compared to the experimental one.



(a) (b) (c)

Fig. 13 Cyclic response of angle connections: a) the hysteretic response, b) characteristic points of the envelope, c) geometric properties, used to define the strength of the connection

The envelope of the hysteretic response can be defined using the expressions presented below.

$$R_y = \frac{\frac{1}{2}R\sqrt{1-\left(d_y/L\right)^2}\left(R_{ch,y}-P\right)+d_yP+M_{fr}}{\sqrt{L^2-{d_y}^2}} \qquad d_y = \sqrt{e_{ch,y}\left(2L-e_{ch,y}\right)} \tag{4}$$

$$R_{max} = \frac{R\sqrt{1-(d_u/L)^2}\left(R_{ch,u}-P\right)+d_uP+M_{fr}}{\sqrt{L^2-{d_y}^2}} \qquad d_y = \sqrt{e_{ch,u}\left(2L-e_{ch,u}\right)} \tag{5}$$

M_{fr} is the moment in the bolt (can be estimated as the tightening torque), R is the distance denoted in Fig. 13c, L is the distance between the bolt and the channel mounted in the panel (see also Fig. 13c), P is the force in the direction perpendicular to the panel plane, $R_{ch,y}$ is the out of plane yield resistance of the channel (typically specified by the producer), $e_{ch,y}$ is the out of plane deformation of the channel at yielding evaluated by FE analysis or experimental testing (for tested angles it was $\approx$ 5mm), $e_{ch,u}$ is the out of plane deformation of the channel at failure evaluated by FE analysis or experimental testing (for tested angles it was $\approx$ 10mm).

5.3 Cantilever connections

Similar to the hammer-head connections, the cyclic response of the cantilever connections can be modelled combining three basic models: elasto-plastic, gap and linearly elastic model, as it is illustrated in Fig. 14. An example of the response defined with such model is presented in Fig. 14 and compared with the experiment.

Characteristic points of the force-displacement response envelope of a cantilever connection with strong channels can be evaluated using the following expressions:

$$R_{fr} = P_V\,k_{fr} \qquad d_{slid} = a - D_b/2 \tag{6}$$

$$R_{max} = \frac{R_{ch,u}(r_1+r_2)}{2l_1} \qquad d_u = a - D_b/2 + e_{ch} \tag{7}$$

P_V is the axial force in the bolt due to the tightening torque, k_{fr} is the friction coefficient between panel and beam (the recommended value is 0.3), $R_{ch,u}$ is the out of plane resistance of the channel (typically provided by manufacturers), D_b is the bolt diameter, a and e_{ch} are distances denoted in Fig. 14.

$$r_1 = R_1 \sqrt{1 - \frac{P_V\, k_{fr}}{2L_2}} \tag{8}$$

$$r_2 = \frac{L_1(b_{pr} - D_b)}{2L_2} + R_2 \tag{9}$$

$$l_1 = L_1 \sqrt{1 - \frac{(b_{pr} - D_b)^2}{4{L_2}^2}} \tag{10}$$

R_1, R_2, b_{pr}, L_1, and L_2 are denoted in Fig. 14

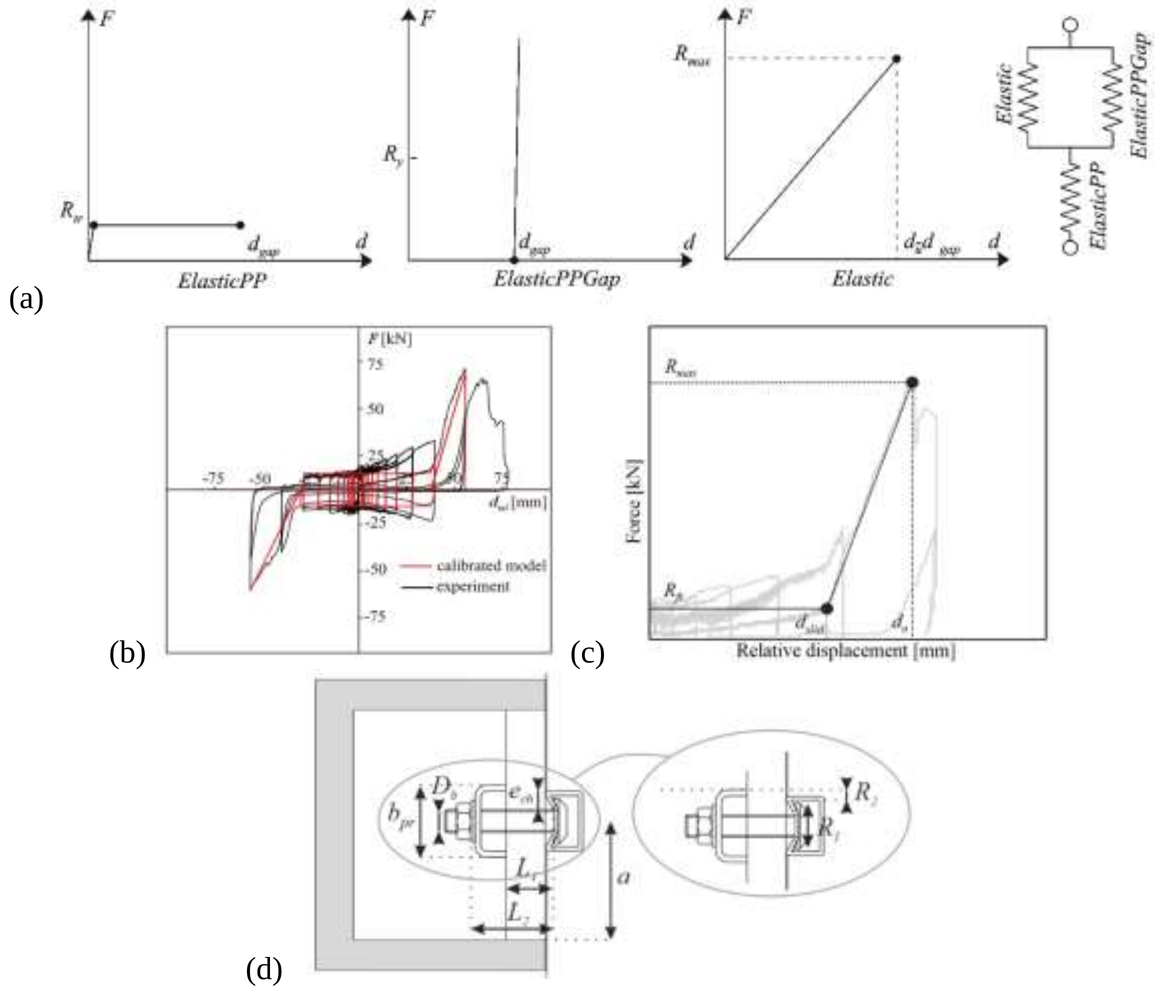


Fig. 14 Cyclic response of cantilever connections: a) the numerical model, b) the hysteretic response, c) characteristic points of the envelope, d) geometric properties, used to define the strength of the connection

6. Conclusions

Typical types of cladding panel connections and the main structure of RC precast buildings were examined experimentally and analytically within the FP7 European research project. Vertical as well as horizontal panels were addressed.

Based on the cyclic experiments of the cladding panel connections the basic mechanisms of their seismic response were identified for the first time. Contrary to the prevailing opinion that cladding panels are most vulnerable in their out-of-plane direction, it was proved that the most critical components – connections typically fail due to the seismic effects in the plane of the panels. This observation was in a good agreement with the damage observed after recent earthquakes.

The results and the observations of the experiments were used to define the appropriate numerical models for all types of examined connections. The proposed robust nonlinear engineering numerical models can be used in the majority of the available commercial and research software. The design procedures were proposed, and the expressions, which can be used to estimate the displacement and strength capacity of all examined connections subjected to the seismic load were derived.

Acknowledgements

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