The 11th Canadian Conference on Earthquake Engineering
Canadian Association for Earthquake Engineering

MONOTONIC AND CYCLIC BEHAVIOR OF REINFORCED CONCRETE CLADDING PANELS WITH INTEGRATED CONNECTIONS

Ioannis N. PSYCHARIS
Professor, National Technical University of Athens, Greece
ipsych@central.ntua.gr

Ioannis M. KALYVIOTIS
PhD Cand., National Technical University of Athens, Greece
ikalyvio@central.ntua.gr

Harris P. MOUZAKIS
Ass. Professor, National Technical University of Athens, Greece
harrismo@central.ntua.gr

ABSTRACT: Recent earthquakes have dramatically shown the importance of cladding connections on the seismic response of precast buildings. In the present work, an experimental and numerical investigation on the monotonic and cyclic behavior of fixed panel connections, referred also as “integrated” connections, is reported. An extensive experimental program has been performed at the Laboratory for Earthquake Engineering of the National Technical University of Athens, Greece, within the framework of the FP7 European project SAFECLADDING, for the investigation of the behavior of integrated connections. The experimental campaign concerns several types of connections materialized with vertical reinforcement bars referred as “rebar” connections, or steel mechanisms of two types: “wall shoe” and “steel plate”. The results show that rebar and wall shoe connections have satisfactory inelastic response, but with significant pinching and possible joint opening and slipping for large displacements. A numerical model of panels with rebar connections was constructed and calibrated against the experimental results, which can be used for non-linear analyses of buildings with integrated cladding walls.

1. Introduction

In common design practice of precast structures, RC cladding panels are not designed to contribute to the structure’s lateral stiffness but are connected to the structure with fastening devices dimensioned to bear the panels’ self-weight, wind loads and seismic loads corresponding to their mass only. However, the behavior of cladding wall systems in recent strong earthquakes, v.i.z. Gölcük, Turkey (1999); Düzce, Turkey (1999); L’Aquila, Italy (2009); Lorca, Spain (2010); Emilia, Italy (2012), dramatically showed that simple connections typically applied in practice are insufficient to resist the large forces induced to them during strong ground shaking, resulting to severe damage to the connections and collapse of the panels.

New innovative panel-to-structure connections and novel design approaches for a correct conception and dimensioning of the fastening system to guarantee good seismic performance of the structure have been investigated within the FP7 European project “SAFECLADDING: Improved fastening systems of cladding wall panels of precast buildings in seismic zones”, GA No. 314122. Part of this investigation concerned fixed panel connections, referred as “integrated”, which are designed to resist the large seismic forces that develop in them since the panels contribute to the lateral load resisting system of the structure. To this end, experimental and numerical investigation on the behavior of fixed connections and their effect on the overall response of the structure have been performed at the Laboratory for Earthquake Engineering of the National Technical University of Athens (NTUA), Greece (SAFECLADDING, 2015).

In the integrated systems, the panel connections are based on a hyperstatic arrangement of the fixed supports of each panel. Typically, vertical panels are used, connected to the beams at four points (Fig. 1a) or at three points (Fig. 1b). Horizontal panel arrangements can also be used, however they are not recommended as they transfer large forces to the columns to which they are connected. For this reason, the experimental campaign was limited to vertical panels only.

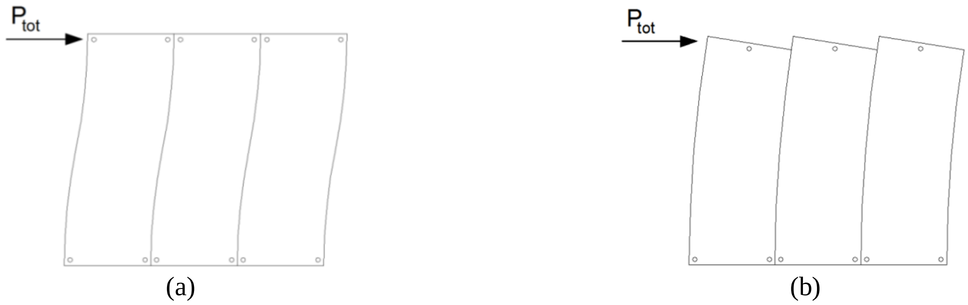


Fig. 1 – Arrangement of vertical panels with: (a) four connections; (b) three connections

Several mechanisms can be used to materialize fixed panel-to-beam connections, as simple connections made of protruding bars, which will be referred as "rebar" connections, or more sophisticated connections made of industrially produced or handmade steel mechanisms. In this paper, the experimental results concerning connections of both types are reported. A numerical model was also constructed, which was calibrated and validated against the experimental data. Application of this model allows the derivation of the constitutive law that governs the non-linear response of panel-to-beam connections of any dimensions, which can be used in non-linear analyses of precast structures with fixed cladding wall panels.

2. Experimental Program and Setup

The experimental campaign concerned a series of tests on panels connected to beams with fixed connections. Three types of connections were examined, specifically:

- Connections made of rebars (Fig. 2a), which are referred as "rebar" connections.
- Industrially manufactured wall shoes (Fig. 2b), which are referred as "wall shoe" connections.
- Connections made of bolted steel plates (Fig. 2c), which are referred as "steel plate" connections.

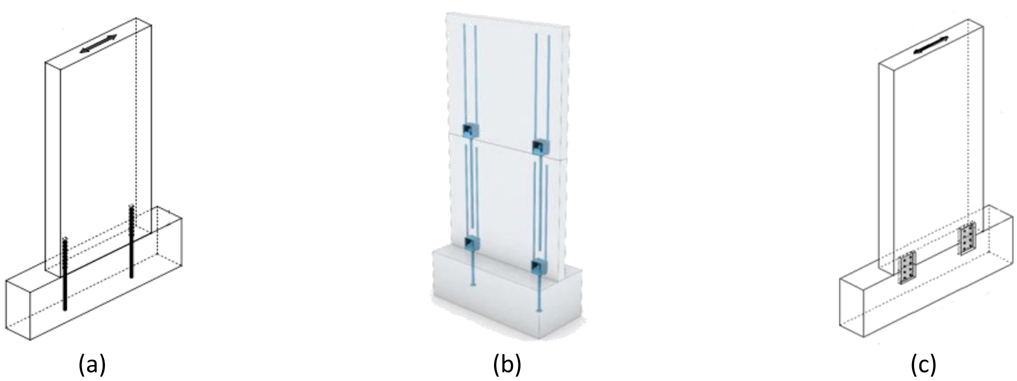


Fig. 2 – Connection types considered in the experiments: (a) rebar connections; (b) wall shoe connections; (c) steel plate connections.

The specimens consisted of a half-height panel and a beam. The panel was placed vertically, inside the front face of the beam, which had an inversed T cross section for proper fastening to the strong floor of the Lab. The horizontal loading was applied at the top of the panel using a hydraulic jack fastened to the reaction wall. The application of the force was made through a specially designed steel mechanism connected to the top of the panel through six bolts in order to obtain a uniform distribution of the load. For further security, this mechanism was restricted against sliding by vertical side plates. The experimental setup is shown in Fig. 3.

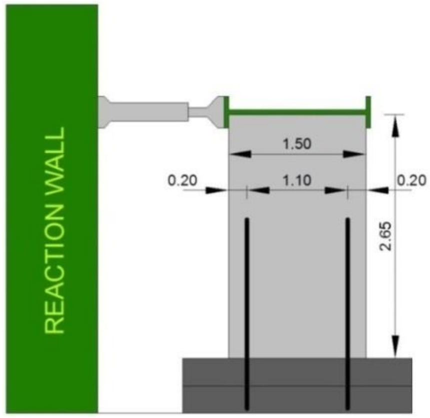


Fig. 3 – Experimental setup and photo of the specimens tested.

Panels of half-height were tested because four point connections behave as beams clamped at both ends and the zero moment location is around the mid-height. The height of the panel specimens was about 2.65 m, their width was 1.50 m, and their thickness was either 0.18 m or 0.20 m depending on the type of the connection. The beams were of cross section 0.40 m $\times$ 0.60 m.

It is noted that the width of the wall (1.50 m) is considered small for typical panels used in industrial buildings (normally in the order of 3.00 m), but it was chosen in order to reduce the size of the specimens without affecting the response of the connections. Besides, simple theoretical analysis showed that in buildings with integrated cladding walls the force induced to each panel connection is practically independent of the width of the panels. Additionally, for the single panel tested, the vertical force induced to the connections increases as the width of the panel decreases, thus, shorter walls allowed to test stronger connections without exceeding the limits of the hydraulic actuator. For this reason, a relatively small width was chosen, which, however, was long enough to guarantee that the two connections were sufficiently apart from each other to prevent interaction effects between them.

The specimens were designed for concrete grade C30/37 and steel grade B500C in accordance with EN 1992-1-1 (2004) and EN 1998-1 (2004), following the results of a finite element analysis using SAP2000.

Eleven tests were performed, five monotonic and six cyclic. In monotonic tests, reverse loading was applied after failure of the first connection (in tension). However, it was not always possible to break the second connection too.

For cyclic loading, the applied loading history (Fig. 4) was based on the protocol proposed by FEMA (2007). The loading history consisted of repeated cycles of step-wise increasing deformation amplitudes by 40%. The number of steps with different amplitudes was equal to ten, with two cycles included in each step. The final target value Δ_m was set equal to the maximum displacement achieved for monotonic loading, while the initial target value Δ_0 was determined so that Δ_m is reached at the 10th amplitude increment, i.e. $\Delta_0 = \Delta_m / 1.4^9$.

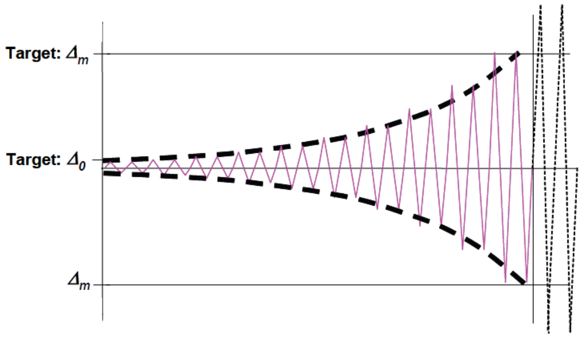


Fig. 4 – Loading protocol applied in cyclic tests.

Concerning the instrumentation, the horizontal and vertical displacements at critical positions were measured with displacement transducers while the deformation at critical parts of the connectors and of the concrete of the panel at several places along the panel-to-beam joint were measured with strain gauges. The setup of the instrumentation is shown in Fig. 5.

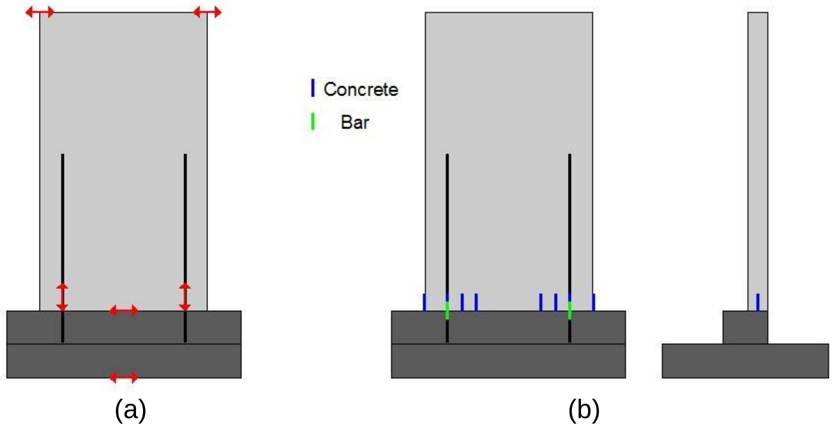


Fig. 5 – Instrumentation setup: (a) Displacement transducers; (b) strain gauges.

3. Experimental Results

3.1. Rebar connections

Two different ways of bonding the protruding bars were tested: with epoxy resins and with high-strength non-shrinking grout. In the former case, holes of about 0.30 m length were drilled in situ for the insertion of the protruding bars, which were then filled with resins. In the latter case, waiting ducts were foreseen during concreting which were filled with grout after the panel was mounted. Two cases were examined: (i) waiting ducts of sufficient length to achieve anchoring of the bars even for grout of reduced strength (about 1.30 m); (ii) industrial corrugated steel sleeves in which the bars were inserted for about 0.30 m.

During the tests with the epoxy resins, slippage of the rebar under tension was observed (Fig. 6 and Fig. 7). For this reason, it was decided to abandon this type of bonding and use high strength, non-shrinking grout instead. In the tests with the grout, no bonding problems were observed. Actually, both connections of type (i) and (ii) mentioned above showed similar response, as shown in Fig. 8: The response of the two types of connections practically coincide: the rebars broke at about the same drift in both tests and the ultimate resistance was about the same.

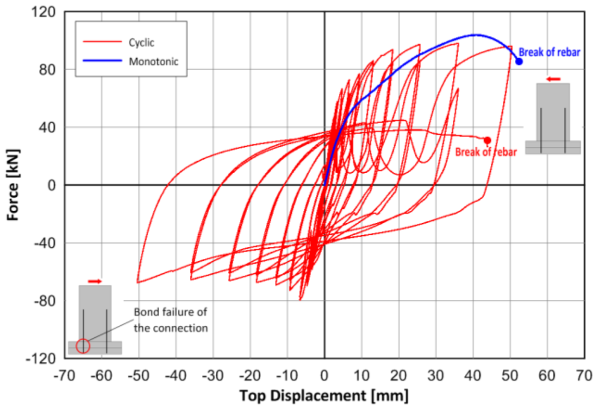


Fig. 6 – Monotonic and cyclic response of specimens with connections made of 1Ø20 rebar bonded with resins (bond failure occurred).

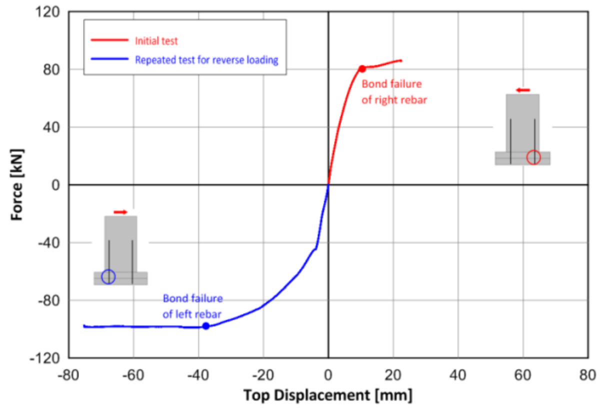


Fig. 7 – Monotonic, initial (red line) and reverse (blue line), response of specimen with connections made of 1Ø25 rebar bonded with resins (bond failure occurred).

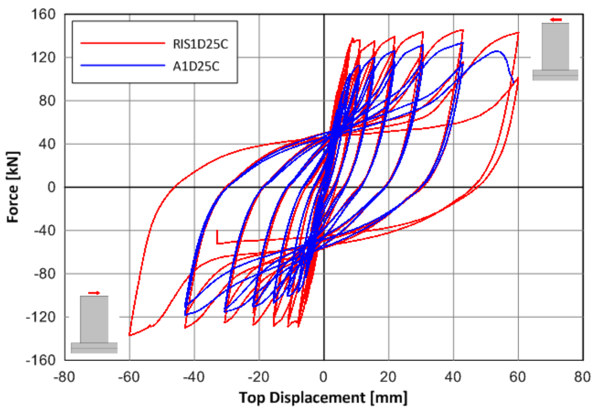


Fig. 8 – Comparison of the cyclic response of specimens with similar connections made of 1Ø25 rebar bonded with grout.

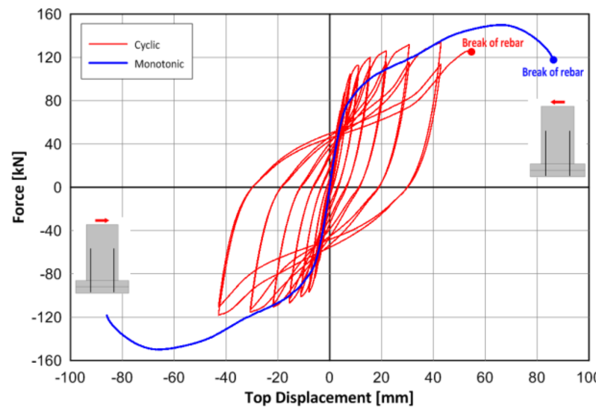


Fig. 9 – Monotonic and cyclic response of specimens with connections made of 1Ø25 rebar.

The comparison of the monotonic with the cyclic response for panels with connections made of 1Ø25 rebars is shown in Fig. 9, in which the horizontal force applied at the top of the panel versus the corresponding displacement is depicted. For monotonic loading, the maximum top displacement achieved before the breakage of the rebar under tension was 86 mm and the maximum horizontal force was 150 kN. For cyclic loading, the envelope of the response was, in general, following the capacity curve for monotonic loading, but it showed larger yield strength. It is noted that the rebar broke in significantly smaller displacement for cyclic loading than for monotonic loading; however, the achieved ductility capacity was more than 5.

The cyclic response was characterized by pinching, which is attributed to the permanent elongation of the rebars of the connections during their plastic deformation, which did not allow the re-establishment of full contact between the panel and the beam. As a result, at the late cycles of the loading history the horizontal force was undertaken solely by the dowel action of the rebars, without any participation of the friction mechanism, resulting in reduced stiffness and significant horizontal displacement at the base of the panel.

The experiments showed that the maximum force that can be attained with rebar connections is proportional to the cross section of the rebars. This is shown in Fig. 10 for monotonic loading: it is clear that the maximum force attained with one rebar ∅25 was almost 50% larger than the one for one rebar ∅20, which is close to the increase in the cross section of the rebars of the connections. Interesting to note is that the ultimate displacement at which failure occurred was also affected by the diameter of the rebars, which is attributed to the different length of bond yielding in the vicinity of the joint.

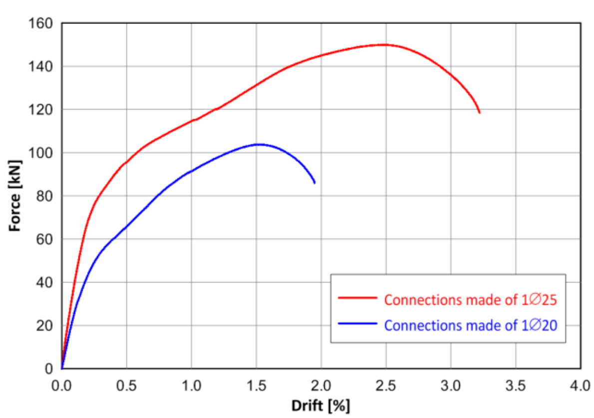


Fig. 10 – Comparison of the monotonic response of specimens with connections made of 1∅20 and 1∅25 rebar.

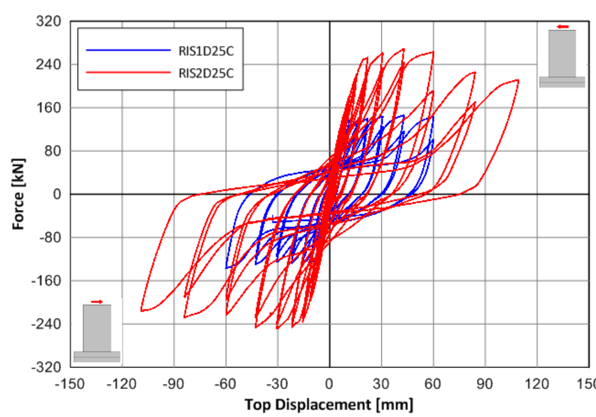


Fig. 11 – Comparison of the cyclic response of specimens with connections made of 1∅25 and 2∅25 rebars.

The linear relation between the ultimate resistance of the connection and the cross section of the rebars was also observed for cyclic response, as shown in Fig. 11, where the comparison of the cyclic response of connections with 1∅25 rebar and 2∅25 rebars is presented.

3.2. Wall shoe connections

The connections are materialized using vertical anchor bolts and a steel wall shoe (Fig. 12) embedded in the beam and the panel respectively. The shoe is anchored to the concrete with anchor bars so that the connection forces are gradually transferred to the concrete away from the connection. The bolt is connected to the shoe through a washer and a nut and, after mounting, the hole of the shoe is filled with grout.

Fig. 12 – Constitutive parts of the wall shoe connection: bolt, washer and nut (bottom left); wall shoe and anchor bars (top right).

Representative experimental results are presented in Fig. 13, in which the horizontal force versus the top displacement of the panel is shown for monotonic and cyclic loading. It is seen that, in general, the envelope of the cyclic response follows the monotonic one, with similar initial stiffness and ultimate strength. However, the ultimate displacement attained for cyclic response was considerably larger than the one for monotonic loading. This should not be attributed to different behavior of the bolt but to the significant sliding of the panel due to the loosening of the nut, caused by the plastic deformation and the elongation of the bolt (Fig. 19e). This is also the reason for the significant pinching observed during the cyclic response. The progressive loosening of the connection with the cycles of loading is shown in Fig. 14, in which the vertical joint opening is shown.

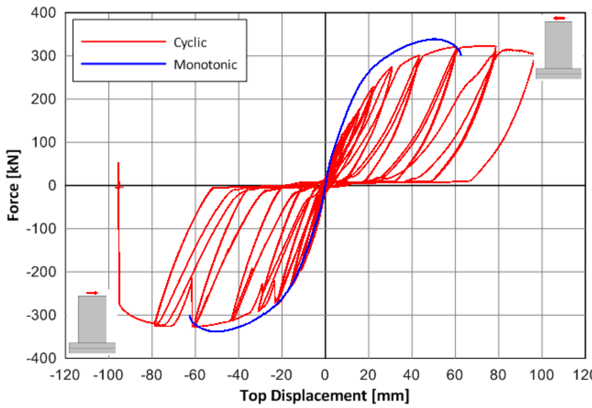


Fig. 13 – Monotonic and cyclic response of specimens with wall shoe connections.

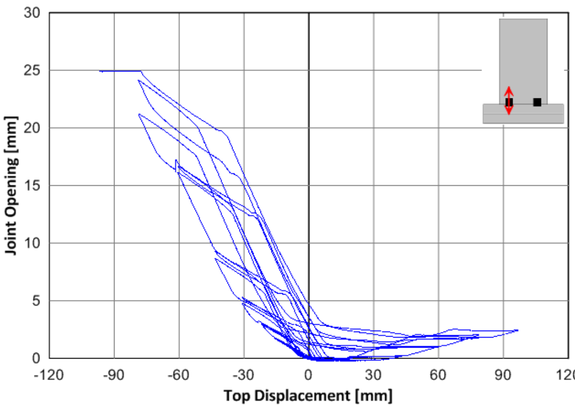
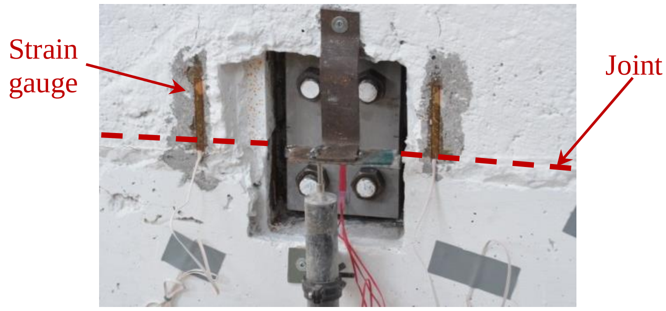


Fig. 14 – Joint opening at the left connection versus top displacement for cyclic loading.

3.3. Steel plate connections

The connections are materialized using a steel plate that is connected to the panel and the beam by an adequate number of bolts fastened to waiting nests (Fig. 15). The waiting nests are anchored to the concrete with anchor bars, so that the connection forces are gradually transferred to the concrete away from the connection.

(a)



(b)

Fig. 15 – (a) Waiting nests of the steel plate connection that are embedded in the beam and the panel; (b) completed connection after mounting the steel plate

Indicative results are presented in Fig. 16, where the monotonic and the cyclic response are compared. Similarly to the other connection types, the envelope of the cyclic response follows the monotonic one, with similar initial stiffness and ultimate strength. Again, the cyclic response showed significant pinching for large displacements, which is attributed to the loosening of the bolts and the permanent distortion of the bolts and the connecting plate (Fig. 19f). The progressive loosening of the connection with the cycles of loading is shown in Fig. 17, in which the vertical joint opening is shown.

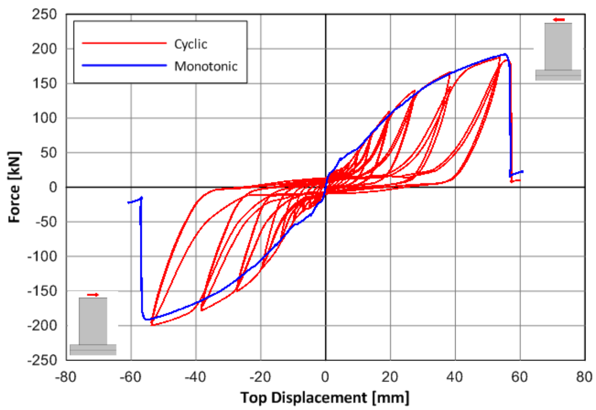


Fig. 16 – Monotonic and cyclic response of specimens with wall shoe connections.

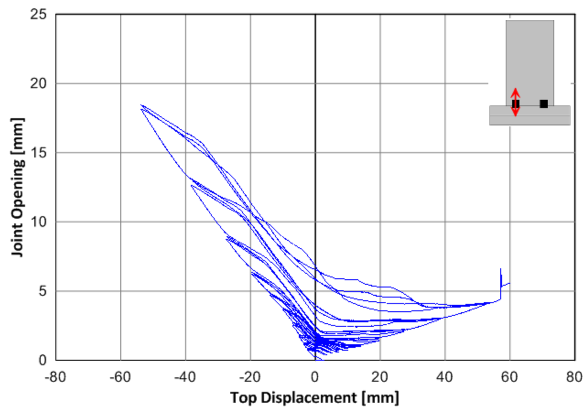


Fig. 17 – Joint opening at the left connection versus top displacement for cyclic loading.

3.4. Ductility and energy dissipation capacity

Rebar connections and wall shoe connections possess considerable ductility capacity (Fig. 11 and Fig. 13) due to the axial deformation of the rebars or the bolt. On the contrary, in steel plate connections suffer shear deformation and, as a consequence, the response is brittle and does not present any ductility capacity (Fig. 16).

Concerning the energy consumption, it is affected by the pinching observed in the behavior of each connection for cyclic loading. This pinching was larger for wall shoe and steel plate connections and less for rebar connections and as a result, the former presents small energy consumption capacity. This is shown in Fig. 18, where the specific energy consumed in each cycle is shown for rebar and wall shoe connections. The specific energy was calculated by the area corresponding to the i-th cycle divided by the one corresponding to the perfect plastic cycle. The cycle referring to failure is not included. In wall shoe connections (Fig. 18b) the specific energy is quite low and practically constant. On the contrary, in rebar connections (Fig. 18a) the energy consumption is significant, despite the fact that some pinching was also witnessed.

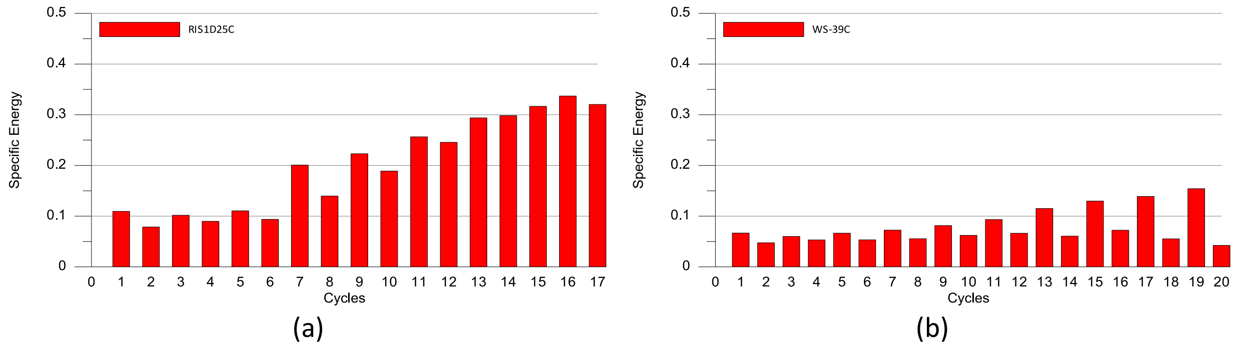


Fig. 18 – Specific energy consumption in each cycle: (a) rebar connections (1∅25); (b) wall shoe connections.

3.5. Observed damage

Concerning the damage observed during the experiments at the concrete, mainly spalling at the faces of the panel and the beam in the vicinity of the connections was observed (Fig. 19a,b). However, for "strong" connections, large compressive stresses can develop at the corners of the panel resulting in significant damage (Fig. 19c). Damage was also observed to the grout cushion (Fig. 19d), typically placed between panel and beam during erection, in case of thick cushion (more than 4 cm); fiber reinforcement, however, can mitigate this phenomenon. As mentioned above, the bolts of wall shoe connections were permanently elongated under intense loading (Fig. 19e). Finally, in the tests with steel plate connections out-of-plane bending of the steel plates occurred (Fig. 19f).

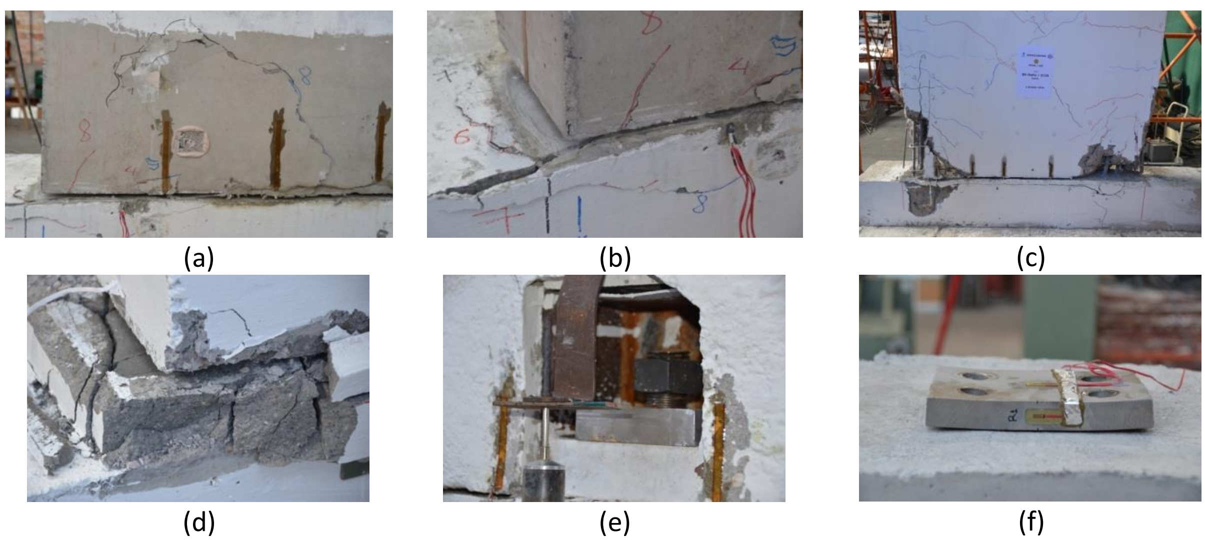


Fig. 19 – Observed damage at the end of the tests.

4. Numerical Model

The behavior of panels with rebar connections was modeled numerically using typical modelling of reinforced concrete structures (fib, 2008). Analyses were performed for panels with four and three connections (Fig. 1). In the former case, the top beam was also included in the model (Fig. 20a); in the latter, the panel was free at its top (Fig. 20b). Both the panel and the beams were modeled with shell elements, while the connection rebars were modelled with beam elements. In order to account for the possible debonding of the connection bars from the surrounding concrete, the nodes of the rebars were not connected directly to the corresponding nodes of the shell elements (modeling the panel or the beam), but through nonlinear springs, the behavior of which was elastic – perfectly plastic. Thus, the "bond" springs were yielding when the bond strength was reached, allowing local loss of the bonding along the rebars. In order to capture well the local debonding, a quite dense mesh was applied around the rebars (max node distance was 20 mm). The joint between the panel and the beam was modeled with tensionless springs (gap elements). More details on the numerical model can be found in Kalyviotis (2013).

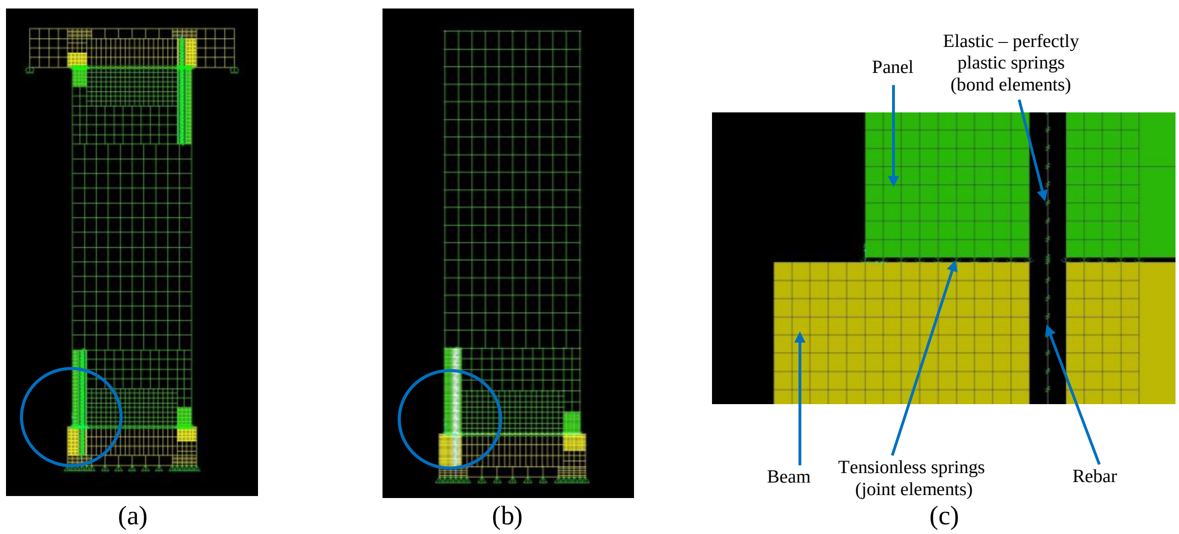


Fig. 20 – Model used for the analysis of the inelastic behavior of panel-to-beam connections: (a) panels with four-point connections; (b) panels with three-point connections; (c) detail of the connection.

The parameters of the numerical model were calibrated against the experimental data. A comparison of the numerical results with the experimental data for the finally selected parameters is shown in Fig. 21 for connections made of 1∅20 and 1∅25 rebars, in which the horizontal force at the top of the panel versus the top displacement of the panel is plotted. It can be observed that the numerical behavior captures accurately the initial stiffness and the post yield response, but the yield force is somehow overestimated. In general, the numerical model can capture quite accurately the overall behavior of the panel-to-beam connection and the distribution of the forces along the connection bars.

The verification of the numerical model allows its application for the development of capacity curves of rebar connections of any dimensions that can be used in numerical models of precast buildings with integrated panel walls for nonlinear time-history analyses.

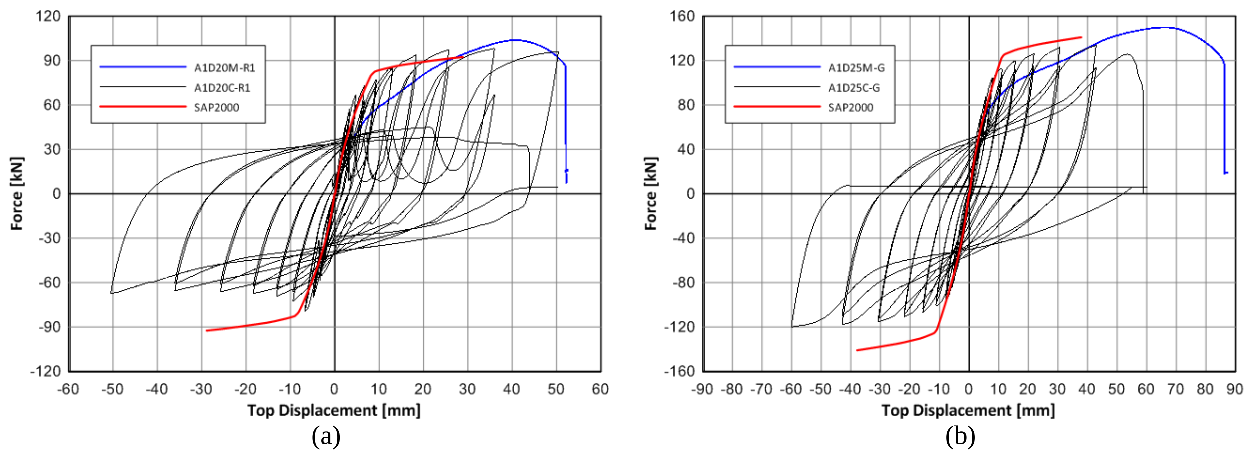


Fig. 21 – Comparison of the numerically predicted overall response (red line) of a panel with two bottom connections with experimental data for monotonic (blue line) and cyclic (black line) excitation: (a) connection made of 1∅20 rebar; (b) connection made of 1∅25 rebar.

5. Conclusions

The conclusions drawn from the performed tests and the numerical investigation on the response of the connections can be summarized as follows:

- The use of epoxy resins for the anchorage of the rebars in in-situ drilled holes proved to be insufficient, even for relatively small forces.
- Permanent opening of the joint and related pinching was observed in all three types of examined connections under cyclic loading.
- In all types of connections, the envelope of the cyclic response followed satisfactorily the monotonic response.
- The attained ductility of rebar and wall shoe connections under cyclic loading was satisfactory. However, because of the aforementioned opening of the joint, it might be problematic to design these connections for ductile response. Steel plate connections do not possess any ductility since the bolts fail in shear.
- The ultimate resistance of the connections is proportional to the cross section of the rebars or the bolts used.
- In most cases, only minor damage was observed in the panels and the beams. The damage was mainly concentrated at the corners of the panels, where large compressive stresses develop.
- In case that grout cushions of significant thickness (more than 4 – 5 cm) are applied, severe cracking can happen in the cushion if mortar without fibers is used.
- In steel plate connections, severe of out-of-plane bending of the steel plates was observed.
- The numerical model developed can capture well the monotonic and the cyclic response of panels with rebar connections and can be used for modeling the overall response of the connections in nonlinear time-history analyses.

6. Acknowledgements

The research presented herein was conducted within the project "SAFECLADDING: Improved fastening systems of cladding wall panels of precast buildings in seismic zones", which was financed by the European Commission in the framework of the FP7 under Grant Agreement no. 314122. The second author wants to thank the Onassis Foundation, for the financial support of his postgraduate studies at NTUA. Special thanks are due to Dr. Lucia Karapitta for her help in the processing of the data and to the Greek companies Interbeton S.A. and Mapei S.A. for supplying materials for the specimens' construction.

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