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DESIGN OF CLADDING PANEL WALLS WITH FIXED CONNECTIONS

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Abstract

In common design practice of precast structures, cladding panels are not designed to contribute to the structure's lateral stiffness but they are connected to the structure with fastening devices dimensioned to bear the panels' self-weight, wind loads and seismic loads corresponding to the panels' mass only. However, the behavior of cladding wall systems in recent strong earthquakes showed that cladding panels may become an integral part of the structure's lateral resisting system, resulting, in that case, to severe damage to their connections with the building.

Innovative panel-to-structure connections and novel design approaches for a correct conception and dimensioning of the fastening system were investigated within the framework of the FP7 European project SAFECLADDING. Within this project, an extensive experimental and analytical program was performed at the Laboratory for Earthquake Engineering of the National Technical University of Athens, Greece for concrete panels fixed to the structure with strong connections, capable to bear the large forces that develop during earthquakes.

In this paper, guidelines on the design of buildings with integrated arrangements of cladding panel walls, which were produced based on the results of the aforementioned experimental and analytical investigation, are reported. Issues concerning the design of the structure as a whole, as well as the design of the panels and their connections are discussed. It is noted that, due to the large stiffness of buildings with integrated panels, small story displacements are expected to occur and, thus, the prevailing energy dissipation mechanism can only be achieved through the plastic deformation of the panel connections. However, despite the fact that strong panel connections possess considerable ductility, it is questionable whether plastic deformation should be allowed to them under the design earthquake, due to reasons related to their overall behavior.

A simplified modeling of the panels and their connections is also proposed for design purposes. In this model, equivalent beam elements are proposed for the modeling of the panels, while appropriate elastic translational and rotational springs are used to capture the complex response of the panel-to-beam connections.

Finally, several types of connections, materialized with vertical reinforcement bars or industrial and hand-made steel mechanisms are presented and their behavior is discussed based on the experimental results.

Keywords: Precast concrete; Cladding panels; Panel connections; Behavior factor; Numerical model

1. Introduction

As a common practice, the panels that cover the exterior façades of precast buildings and their connections are not designed to participate in the load bearing system of the building but are dimensioned to resist only their self-weight, wind and seismic loads corresponding solely to their mass. Based on this approach, the design of precast buildings is based on a bare frame model where the cladding panels are considered only as additional masses and the energy dissipation capacity is similar to the cast in situ structures.

Although for moderate earthquakes the cladding-to-structure connections perform quite satisfactorily, experimental research conducted in the last decades ([1]-[5]) has shown that under strong excitations the panels may become an integral part of the resisting system. As a result, the overall stiffness is increased, leading to forces much higher than those calculated from the frame model. Furthermore, the seismic force reduction in frame precast structures cannot rely on energy dissipation in plastic hinges at the columns, as the connections between cladding and structure usually reach their limit well before the required drifts can develop. Despite these research conclusions and the fact that considerable damage to cladding systems has been reported since the Alaska earthquake in 1964, the inadequacy of the design of their connections was realized only recently, after the extensive damages produced during certain seismic events, like the L'Aquila, Italy (2009) and the Emilia, Italy (2012) earthquakes. It is interesting to note that in Emilia earthquake, the majority of the buildings with damages in structural elements were designed only for gravity loads (the region was not declared seismic at the time). However, the damages on non-structural elements were also significant even in case of buildings designed with seismic provisions. Information on the seismic behavior of precast buildings with panel walls in the wider Mediterranean region is rather limited. For example, in Greece precast buildings behaved relatively well during the Athens (1999) earthquake; however the relatively limited use of precast panel walls and, in general, precasting in the country does not allow the drawing of conclusions.

The investigation on the seismic response of precast structures with cladding panels has been recently complemented with the FP7 European project "SAFECLADDING: Improved fastening systems of cladding wall panels of precast buildings in seismic zones", GA No. 314122. Innovative panel-to-structure connections and novel design approaches for a correct conception and dimensioning of the fastening systems have been investigated. Specifically, the behavior of 'fixed' panel connections, designed to resist large seismic forces and forcing the panels to participate in the lateral load bearing system, has been investigated experimentally and numerically at the Laboratory for Earthquake Engineering of the National Technical University of Athens (NTUA), Greece ([6]-[8]). In such frame-panel systems, herein referred as "integrated", the connections are arranged with a hyperstatic set of fixed supports. The term 'fixed' is used here interchangeably with the term 'pinned' and denotes connections with restrained displacements, while rotations are allowed. With this arrangement of connections, the panels participate to the seismic response of the structure as in case of a wall system or wall-equivalent dual system and should, therefore, be proportioned accordingly.

2. Structural arrangement

A typical hyperstatic arrangement of panel connections is shown in Fig. 1(a) for vertical panels. Four fixed fastenings are used, one at each corner, the lower two attached to the bottom beam, the upper two attached to the top beam. With this arrangement the panel acts as a vertical beam clamped at both ends.

In order to accommodate possible thermal expansion of the panel, the two upper fixed fastenings can be replaced by two vertically sliding connections. Fig. 1(b) shows this arrangement, in which each panel acts as a vertical cantilever beam clamped at its bottom and pinned at its top. For the same horizontal top action, P_{tot}, this arrangement results to double reactions at the lower connections compared with the four-connection panel. It is noted that, for the modeling of the panel, the two upper sliding connections can be replaced by one central pinned connection. For this reason, this arrangement will be denoted as 'three-connection' panel in the ensuing.

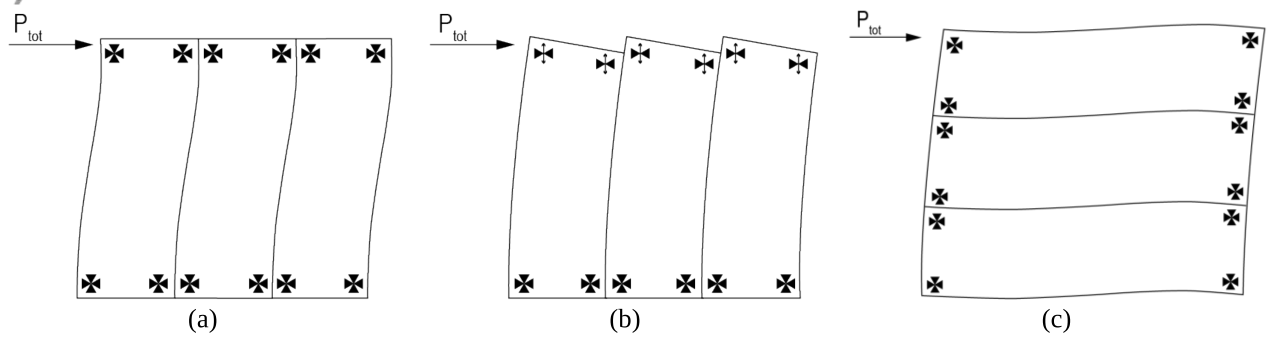


Fig. 1. Arrangement of panels: (a) Vertically with four fixed connections; (b) Vertically with two fixed and two sliding connections; (c) Horizontally with four fixed connections.

Fig. 1(c) shows a hyperstatic arrangement of connections for horizontal panels. Four fixed fastenings are used, one at each corner, attached to the contiguous columns. With this arrangement, the panels act as horizontal beams clamped at their ends. As a result, the columns' deformation is significantly affected during seismic loading, inducing high local forces at the locations of the connections, and thus horizontal panel arrangement in integrated systems is not recommended.

3. Integrated connections

Three types of connecting mechanisms, which can be used in systems with integrated cladding walls and were experimentally investigated in SAFECLADDING project [6] are presented below, specifically:

- connections with protruding rebars (Fig. 2);
- connections with bolted shoes (Fig. 4(a));
- connections with bolted plates (Fig. 6).

Independently from the type of connection, a gap is typically left during construction between the panel and the supporting beam, which is filled with high-strength, non-shrinking mortar after mounting of the panels. The purpose of this mortar bed is to form a uniform contact between the panel and the supporting beam, necessary to ensure friction and to prevent sliding. To prevent severe cracking of this embedding, its thickness should not exceed 50 mm, while the usage of fiber-reinforced cement mortar is suggested.

3.1. Connections with protruding rebars

This type of connection can be materialized using reinforcement bars protruding from the panels into the beams or vice versa. In both cases, waiting corrugated sleeves are provided in the opposite element for the insertion of the rebars, which are filled with high-strength, non-shrinking grout after erection. In Fig. 2, a typical connection in case of rebars protruding from the panel into the supporting beam is shown, whereas in Fig. 3 the cyclic response obtained experimentally for such a connection is depicted. Special holding provisions are required to ensure proper positioning of the panels and their stability until the hardening of the mortar.

No special requirements apply to the embedded part of the protruding rebars. However, proper reinforcement shall be placed around the sleeves to confine the concrete and anchor them against pull-out. The use of smooth sleeves instead of corrugated ones is not recommended, as slip of the grout on the duct s internal surface can occur during pullout.

An alternative solution would be to use mechanical couplers, as the ones applied in practice to achieve continuity of reinforcement, in order to attach the protruding part of the bars on site. Special technological provisions, depending on the adopted coupler, should be applied to ensure the required strength of the connections. If bolted bushes are used, proper provisions should be adopted to ensure that the weakening of the rebars due to threading will not jeopardize the strength of the connection. In any case, when using mechanical couplers, the coupling device must have been experimentally qualified for its effectiveness in terms of over-resistance with respect to the connected rebars.

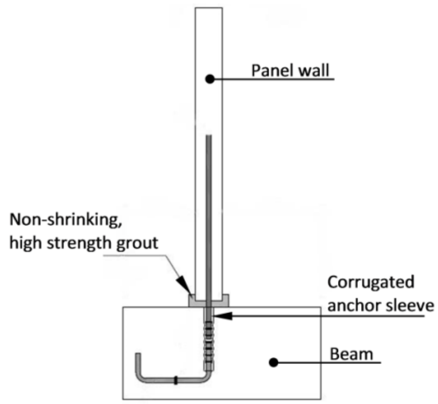


Fig. 2. Connection with protruding rebars.

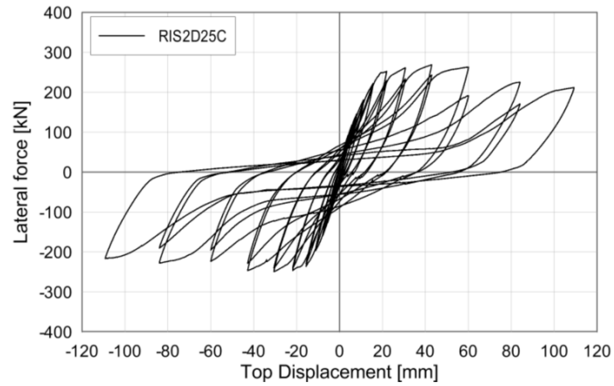


Fig. 3. Lateral force versus displacement relationship for connections with protruding rebars (from [6]).

3.2.Connections with bolted shoes

There are several companies in the market that produce devices to connect wall panels to beams or to other wall panels (Fig. 4(a)), usually called “wall shoes”. A typical wall shoe is shown in Fig. 4(b); it consists of:

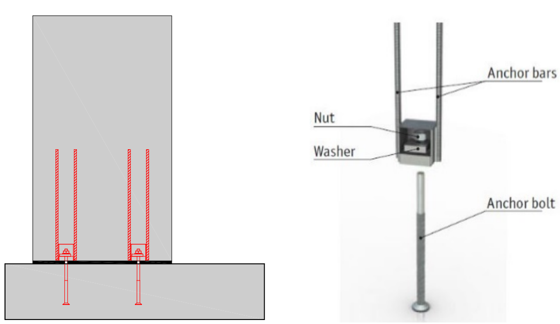
– a steel nest with a strong bottom plate which is cast into the bottom part of the upper wall and fixed with anchor rebars;
– an anchor bolt which is cast into the upper part of the lower beam;
– a washer and a nut used to fasten the bolt to the bottom plate.

There are special requirements concerning:

– the minimum concrete cover;
– the minimum thickness of the wall’s cross section;
– the minimum distance from the wall edges;
– the principal and the supplementary reinforcement.

which are usually specified by the producer. These requirements depend on the nominal force that each device can resist and vary from system to system. Proper reinforcement is required around the anchor bolts to confine the concrete around them and anchor them against pull-out.

The monotonic and cyclic experimental behavior of connections with bolted shoes is depicted in Fig. 5 in terms of lateral force.



(a) (b)

Fig. 4. Connections with bolted shoes: (a) Schematic view; (b) Wall shoe.

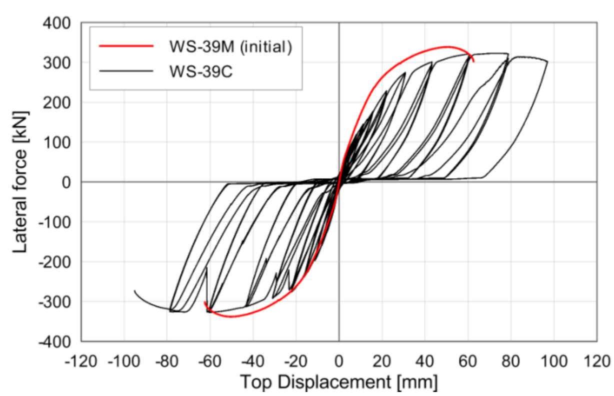


Fig. 5. Lateral force versus displacement relationship for connections with bolted shoes (from [6]).

3.3.Connections with bolted plates

This type of connection can be materialized using steel plates to connect wall panels to beams. The connections consist of waiting nests, fixed to the supporting beam and to the panel, on which a steel plate is bolted. The steel nests are embedded in the concrete and welded to reinforcing rebars so that the connection forces are gradually transferred to the concrete. A typical configuration of this connection is shown in Fig. 6.

There are special requirements concerning:

– the minimum concrete cover;
– the minimum thickness of the wall's cross section;
– the minimum distance from the wall edges;
– the principal and the supplementary reinforcement

which depend on the nominal force that each device can resist and aim at preventing local damage to the concrete.

Yielding of the steel plate is not expected and the critical failure mechanism is caused by the shear failure of the bolts (Fig. 7). The advantage of this connection is that the steel plate can be easily replaced in case of damage. It is also noted that this connection does not pose any difficulty in the installation of the panels.

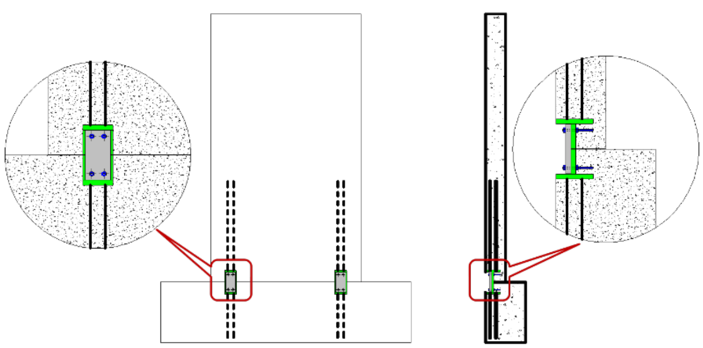


Fig. 6. Connections with bolted plates.

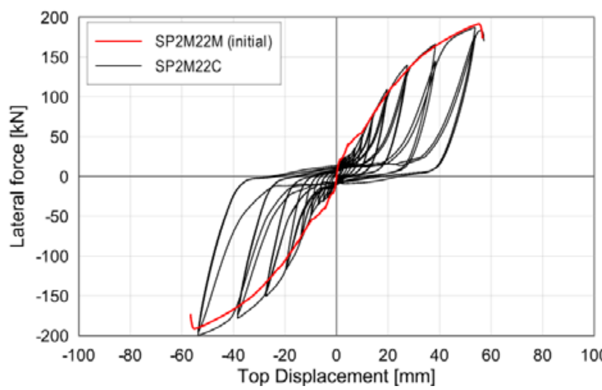


Fig. 7. Lateral force versus displacement relationship for connections with bolted plates (from [6]).

4. Seismic design

4.1.Load resisting mechanism

In Fig. 8, the resisting mechanism at the panel-to-beam joint under a seismic action is shown. Only the height of the panel up to the zero-moment point (shear length, ℓ_S) is depicted, which is equal to one half of the total height of the panel in case of four-connection panels (Fig. 1(a)) and equal to the full height of the panel in case of three-connection panels (Fig. 1(b)). The connection device provides the tensile strength in the tension zone, while the concrete provides the resistance in the compression zone. As long as there is not residual joint opening, the horizontal resistance is provided mainly by the friction between the panel and the beam and by the shear resistance of the connection.

The possible failure modes associated with the resisting mechanism in all three connection types are presented in Table 1.

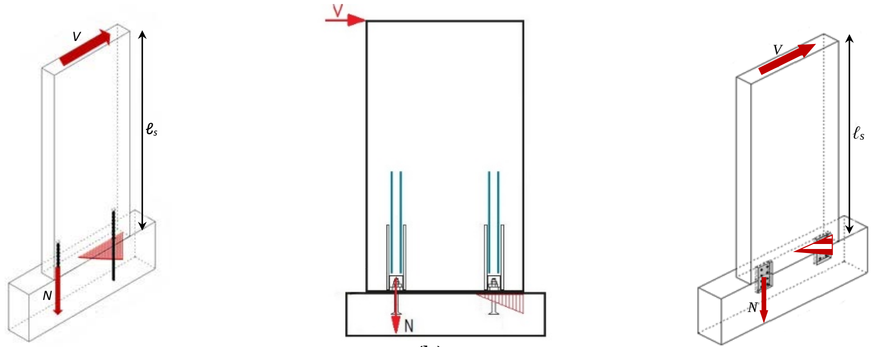


Fig. 8. Resisting mechanism for connections with: (a) Protruding rebars; (b) Bolted shoes; (c) Bolted plates.

Table 1. Modes of failure of the connections

Modes of failure	Protruding rebars	Bolted shoes	Bolted plates
Pull-out of the tension bars/anchor bolts	√	√	
Breakage of the tension bars/anchor bolts	√	√	
Shear failure of the bolts			√
Plastic elongation of the tension rebars/anchor bolts	√	√	
Failure of the concrete in the compression zone	√	√	√
Sliding shear failure of the panel	√	√	√
Warping of the washer plate		√	
Permanent distortion of the bolts and/or the plate			√
Failure of the connecting plate			√

4.2.Behavior factor

Generally, in buildings with integrated panels the lateral resistance of the panel walls is higher than 50% of the total lateral resistance. According to EC8 [9], such buildings are classified as wall systems or wall-equivalent dual systems and it is proposed to be designed for Ductility Class Medium (DCM). Therefore, the maximum allowed basic value of the behavior factor according to EC8 is q_0 = 3.0 and the overall behavior factor to be used in the seismic design is $q = q_0 \cdot k_w$, where $k_w = (1+\alpha_0)/3.0$ (less than 1.0 but not less than 0.5) and α_0 is the prevailing aspect ratio of the panel walls: $\alpha_{0i} = \Sigma h_{wi}/\Sigma \ell_{wi}$.

However, due to the large stiffness of buildings with integrated panels, small story displacements are expected to occur under seismic action. As a consequence, energy will mainly be dissipated due to the plastic deformation of the panel connections and not by the inelastic response of the columns, the behavior of which, due to the small displacements, will be rather elastic. This is not a desirable plastic mechanism, because the experimental investigation showed that it leads to permanent plastic deformations of the connecting devices with the following consequences: (i) Significant residual opening occurs at the panel-to-beam joint, resulting in sliding shear failure of the panel due to loss of the friction resistance (see Fig. 9); and (ii) The nonlinear behavior of the connections under cyclic loading is characterized by considerable pinching (see Figs. 3, 5 and 7). For this reason, it is suggested that plastic deformation should not be allowed to the design of the panel connections, despite the fact that common connecting devices possess considerable ductility (see Table 2).

Specifically, it is suggested that the connections shall be overdesigned in the sense of EC8 (clause 5.11.2.1.2) and the design action-effects should be derived on the basis of the capacity design concept. However, since capacity design is not easy to apply, it is suggested that connections materialized with protruding rebars or wall shoes shall be designed with the forces derived from a structural analysis performed with behavior factor q = 1.5, while in case of connections with bolted plates an even smaller value of the behavior factor is suggested (e.g. q = 1.0) due to the brittle type of failure. This applies only to connections and not to columns, which shall be designed with the abovementioned behavior factor, thus it does not lead to an over-conservative design.

It is noted that the numerical investigation performed within SAFECLADDING project showed that the panels' and columns' shear forces are affected by the structural regularity, the connectivity of the elements, the percentage of the perimeter covered with cladding and the openings. In several cases, columns entered the inelastic region and therefore adequate detailing is required. Bearing that in mind, strength and ductility surplus for the connections are considered as assets.

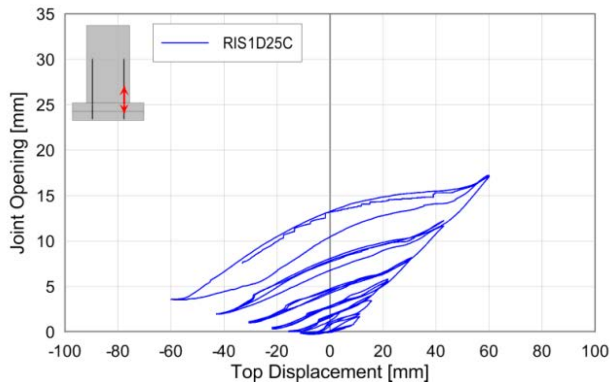


Fig. 9. Joint opening vs. top displacement for connections with protruding rebars (from [6]).

Table 2. Ductility and Stiffness degradation in SAFECLADDING experiments

No.	Test name	Connection Type	μ_Δ (a)	K_u/K_y (b) [%]	No.	Test name	Connection Type	μ_Δ (a)	K_u/K_y (b) [%]
1	A1D20M-R1	Protruding rebars	5.9	26.1	7	RIS2D25C	Protruding rebars	5.7/7.0	14.9/13.1
2	A1D20C-R1	Protruding rebars	7.6/ - (*)	17.4/ -	8	WS-39M	Bolted shoes	6.4	28.2
3	A1D25M-R2	Protruding rebars	-	-	8	WS-39C	Bolted shoes	4.4/6.3	30.1/18.9
4	A1D25M-G	Protruding rebars	8.5	14.9	10	SP2M22M	Bolted plates	-	-
5	A1D25C-G	Protruding rebars	6.4/5.4	12.4/22.7	11	SP2M22C	Bolted plates	-	-
6	RIS1D25C	Protruding rebars	6.8/7.2	15.3/15.1					

(a) Displacement ductility ratio (Δ_u/Δ_y). In monotonic tests ductility is calculated only for the initial loading.
(b) Ratio of secant stiffness at maximum displacement versus yield [$(V_u/\Delta_u)/(V_y/\Delta_y)$].
(*) After the completion of each test the response of the specimens under reverse loading was also examined. The second number represents the value obtained during the reverse loading.

Concerning the wall panels themselves, it is not necessary to be designed as ductile walls, but it is sufficient to be dimensioned following the design criteria of EC8 for Lightly Reinforced Walls. However, special care should be given to the detailing of regions where large forces develop due to rocking motion, i.e. close to the connections and the corners of the walls.

4.3. Wall panels detailing

Generally in current practice, the detailing of precast cladding panels does not fulfil the code requirements for structural shear walls, especially in what concerns their minimum thickness, since, in order to guarantee thermal insulation, common solutions use single or double thin concrete layers, both insufficient to resist the large forces induced to them during strong ground shaking in integrated systems.

Specific requirements for integrated cladding panels used in wall systems are presented here. These requirements are formulated through proper adaptation of the EC8 rules for the cast-in-situ shear walls and aim to ensure strong fastening of the connectors, able to provide adequate in plane shear resistance and sufficient ductility. Although the design criteria of EC8 for Lightly Reinforced Walls can be followed, some requirements for the DCM or DCH ductile walls should be adopted. Specifically, it is suggested that:

– Panels shall have a solid bearing layer of at least 150 mm thickness;

- A double reinforcing mesh of ductile steel shall be provided at the two faces;
- The sides of the mesh shall be not larger than 200 mm and the bars should have diameter at least 8 mm;
- A perimeter reinforcement shall be added with at least 2 longitudinal bars of diameter larger than 12 mm and edge links of diameter larger than 8 mm;
- Proper anchoring reinforcement of the inserts shall be placed at the connection points.

Panels with openings should be properly designed for the transmission of the expected in plane actions through the lateral posts of the openings. Proper reinforcement, specifically continuous steel ties, horizontal or vertical, should be provided around the openings, similar to the reinforcement placed around openings in ductile shear walls. As a minimum, these ties should satisfy clause 9.10 of EC2 [10]. Also, special reinforcement should be placed around the anchoring of the connections in order to provide adequate confinement. It is noted, however, that the experiments performed showed that, in case of industrial connections, the reinforcement specified by the manufacturer is capable to guarantee the good behavior of the connection.

5. Structural modeling

5.1. Modeling of the structure

Generally, the model of the structure that will be used in the analysis must reflect the real stiffness distribution in order to capture accurately the distribution of the forces that develop during the seismic excitation and especially the forces induced to the connections between the precast members.

For the numerical model of the structure, beam elements in combination with plate elements can be used, positioned along the axes or in the mid-planes of the corresponding structural elements. It is recommended to model the eccentricities between the members' axes at the joints using link rigid elements. Connections between structural elements should be properly modeled concerning the relaxed degrees of freedom and the corresponding stiffness. It is mentioned that, for structures with integrated arrangements of wall panels, large forces might develop not only to the panel connections but also to all other connections of precast members (roof-to-roof, roof-to-beam, beam-to-column). In order to accurately estimate these demands, accurate numerical simulation is required for the analysis.

5.2. Modeling of the wall panels and their connections

Although the use of plate elements for modeling the behavior of the panels is a better choice from a theoretical point of view, beam elements can also be used. A simple but reliable representation is shown in Fig. 10 for panels with four connections: the panel is modeled with five beam elements with the main element placed at the centerline of the panel and the remaining four elements being used for connecting the panel element with the beams.

For the panel connections, their actual deformability should be reliably represented in the numerical model in order to obtain reliable results, since, if the connections are modeled with no deformability (e.g. fixed "built in" full support or hinged support), the results of the analysis could lead to unrealistic distribution of the joint forces. For this reason, in order to account for the deformability of the connections, zero-length rotational springs are placed at the ends of the panel element in the model of Fig. 10, capturing the overall rotational response at the panel-to-beam joint at each side. It is noted that the joint response is dominated by the rocking of the panel, which leads to tension of the connection at the uplifting side and compression of the concrete at the opposite side of the panel (Fig. 8). The calculation of the rotational springs' stiffness is discussed in the next subsection.

In case that the two upper fastenings are replaced by vertically sliding connections (three-connection panels, see Fig. 1(b)), the rotational restraint at the top side of the panel is released by setting the stiffness of the top rotational spring to zero (corresponds to pinned connection between the panel element and the top connecting elements).

5.3. Stiffness of zero-length rotational spring

The stiffness K_θ of the zero-length rotational spring can be calculated assuming that the connection under tension can be simulated by a vertical linear spring of stiffness K_Z (up to the theoretical point of yielding). Let M be the bending moment at the base of the panel and θ the corresponding rotation (Fig. 11). The tensile force F_T induced to the connection is:

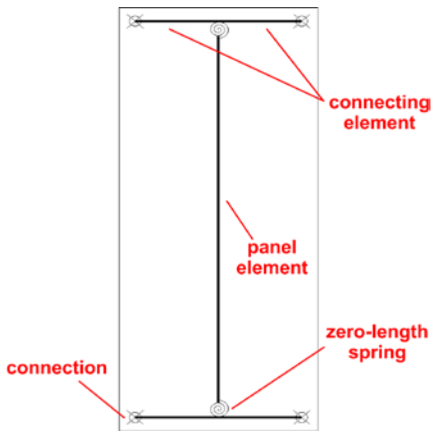


Fig. 10. Five beam model for panels with four connections.

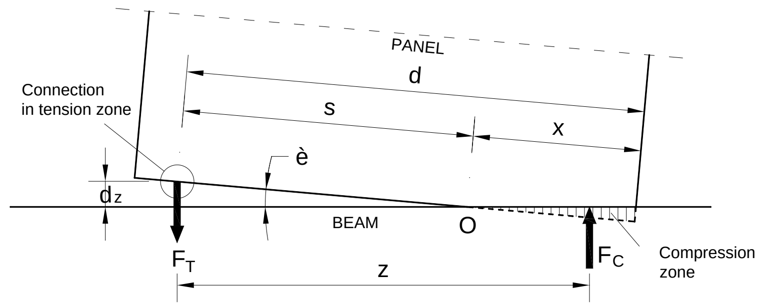


Fig. 11. Forces and rotation at the base of the panel.

$$F_T = K_Z \cdot d_Z = K_Z \cdot s \cdot \theta \tag{1}$$

$$F_T = M/z \tag{2}$$

where:

s is the distance of the centerline of the connection from the neutral axis O;
z is the inner lever arm of the forces that develop at the base of the panel.

Combining Eqs.(1) and (2), the following relation can be derived for the stiffness K_θ of the zero-length rotational spring:

$$K_\theta = M/\theta = K_Z \cdot z \cdot s \tag{3}$$

The values of z and s can be estimated following standard approximations usually made for reinforced concrete sections (e.g. EC2):

$$\begin{gathered} z = 0.90 \cdot d \\ x = 0.25 \cdot d \\ s = d - x = 0.75 \cdot d \end{gathered} \tag{4}$$

where:

d is the effective depth of the cross section;
x is the length of the compression zone.

In what concerns the value of K_Z, for connections with protruding rebars and wall shoes (details about the connection types are given in section 3), one can write:

$$K_Z = E \cdot A/L_{eff} \tag{5}$$

where:

E is the Young's modulus of the steel of the rebars/anchor bolts;

A is the stressed area of the rebars/ anchor bolts;
L_{eff} is the equivalent length of the spring, denoting the effective length in which the elongation of the rebars/ anchor bolts takes place.

Based on the data obtained from the experimental investigation performed within SAFECLADDING project [6], L_{eff} is related with the diameter of the rebar/anchor bolt according to the following approximate relation:

$$L_{eff} \cong 15 \cdot \emptyset \tag{6}$$

It is noted, however, that, due to the limited number of the available experimental data, this approximation needs further verification from additional experimental and numerical investigations. For this reason, if K_θ is determined from Eq. (4), it is suggested that two structural analyses are performed: one with double the value of K_θ, from which the maximum forces in the connections shall be determined, and one with half the value of K_θ, from which the maximum displacements shall be determined.

For connections with bolted steel plates, the calculation of K_Z is not easy, since it is affected by a number of factors which cannot be easily modelled, as the shear deformation of the bolts, the elongation of the steel plate, the distortion of the holes and the plate itself, etc. Further investigation is needed in that case.

5.4. Pre-dimensioning of panel wall connections

In order to calculate the rotational stiffness K_θ of the panel model using the aforementioned formulae, the cross section of the rebars/anchor bolts is needed. Therefore, an initial evaluation of the forces expected to develop in the connections is necessary. This pre-dimensioning of the connections can be based on simplified assumptions concerning the distribution of the lateral forces, as the one presented in the following.

Let us assume that there are *n* vertical panels at each side of the building along the direction of the seismic action and that each panel is pinned to the top and the bottom beam by two connectors at each side. Each panel has dimensions $L_{panel} \times H_{panel}$, while L and H are the horizontal and the vertical distance between the connections (Fig. 12). Then, one can define the coefficient C_1:

$$C_1 = L/L_{panel} \tag{7}$$

which measures the insertion length of the connections from the panel edges.

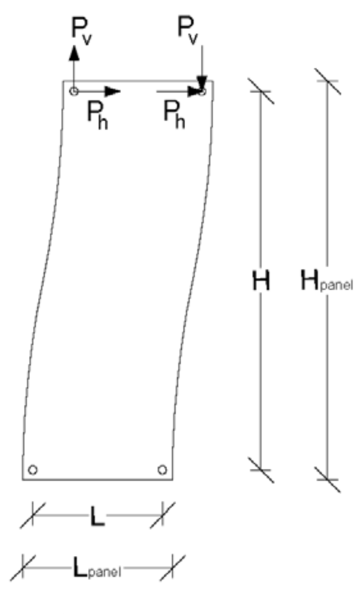


Fig. 12. Distribution of forces at a panel with four connections.

In general, the total length, L_{tot}, of the building sides is not fully covered with panels. The 'coverage' of each side with panels is measured with the coefficient C_2 which is defined by:

$$C_2 = n \cdot L_{panel}/L_{tot} \tag{8}$$

For a symmetric building and cladding wall panels placed at the two external sides, and accounting for the large stiffness of the panels compared to the stiffness of the precast frame, it can be assumed that the base shear due to the earthquake load, P_{base}, is undertaken solely by the panels. Then, it can be proven that the horizontal force, $P_{i,h}$, and the vertical force, $P_{i,v}$, that are induced to each panel's connection (Fig. 12) are given by the following expressions:

$$P_{i,h} \cong P_{base}/(4 \cdot n) \tag{9}$$

$$P_{i,v} \cong P_{i,h} \cdot H/L \tag{10}$$

The total force induced to each connection is equal to $P_i = \sqrt{{P_{i,h}}^2 + {P_{i,v}}^2}$. Using the above equations, one can write:

$$\frac{P_i}{P_{base}} = \frac{1}{4}\sqrt{\frac{1}{n^2} + (\frac{H}{C_1 \cdot C_2 \cdot L_{tot}})^2} \tag{11}$$

In general, for values of n larger than 4, the term $1/n^2$ is much smaller than the second term under the square root and can be neglected. Then, Eq.(10) turns into:

$$\frac{P_i}{P_{base}} \cong \frac{H}{4 \cdot C_1 \cdot C_2 \cdot L_{tot}} \tag{12}$$

In case of panels with sliding top connections, rotation at the top side of the panel is allowed, which is equivalent to one pinned top connection as mentioned above (three-connection panels). In that case, the forces induced to the bottom connectors are double than those for panels with four connections, i.e.:

$$\frac{P_i}{P_{base}} = \frac{H}{2 \cdot C_1 \cdot C_2 \cdot L_{tot}} \tag{13}$$

Eqs (12) and (13) imply that the force induced to each connection is independent of the width of the panels but is greatly affected by the 'coverage' of the external sides by panels (coefficient C_2) and the height of the story H. Moreover, it is obvious that the major component of P_i is in the vertical direction.

It must be emphasized that the above analysis gives an estimation of the average forces expected to develop in the panel connections. Rigorous analytical investigations ([6], [11]) have shown that the forces might change significantly from place to place, depending on the position of each panel in the load-resisting structural system. Therefore, this analysis can only be used for the pre-dimensioning of the connections, while the final verification of the connections shall be based on the actual forces derived from the structural analysis of the whole system, performed using the model of Fig. 10. In case this verification shows that some connections need to be modified, the structural analysis shall be repeated.

6. Conclusions

As the current practice, according to which cladding panel walls and their connections are not designed to participate in the load bearing system of the building, has proven to be insufficient for strong earthquakes in which panels may become part of the loading bearing system, an alternative with integrated arrangements of panel walls is proposed. In the present paper, guidelines on the design of buildings with fixed cladding panel walls and the design of the strong panels' connections are presented. The main issues discussed are:

- In buildings with integrated cladding panel walls, plastic deformation of the panel connections occurs for large lateral displacements. However, the experimental investigation showed that this is typically accompanied by horizontal slippage at the panel-to-beam joint and by considerable pinching during the cyclic response.
- Due to the above-mentioned unwanted effects, it is suggested that the panels' connections should be overdesigned applying the capacity design concept. To this end, it is suggested that the over-proportioning of the connections shall be based on the forces derived from a structural analysis performed with behavior factor q not larger than 1.5.
- For the design of the columns, the behavior factor that corresponds to wall or wall-equivalent dual systems should be applied.
- A linear model consisted of five beam elements is proposed for the model of the panels, in which rotational springs are included, associated with the moment-rotation law at the panel-to-beam joint.
- The pre-dimensioning of the connections can be based on simplified assumptions concerning the distribution of the lateral forces, while the final verification of the connections shall be based on the actual forces derived from the structural analysis performed using the full model of the structure.
- Large forces might also develop to other connections of precast members (such as roof-to-roof, roof-to-beam etc.). In order to accurately estimate these demands, accurate numerical simulation is required.

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