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EVALUATION OF MAIN BEAM-TO-COLUMN HINGED CONNECTIONS USED FREQUENTLY TO PRECAST GROUND FLOOR STRUCTURES

BY

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Abstract. This paper aims to evaluate numerically to what extent some connection joints currently used respects the definition of hinged connection.

Within the design of connections, as design engineers, we must consider two main aspects:

– Hinged connection, as per definition, is a connection allowing rotation between two components, while all displacement of any of these two components one from another is not allowed;

– Importance of the type of connection used ought to get as close as possible to its theoretical hypothesis as only in this way the designed structures will be able to generate effects according to predictable and controllable expectations.

According to experience gained, three types of joint connection were selected for a numerical evaluation: the results lead to the conclusion that it is necessary to make efforts in order to adopt clear norms regarding the design and calculus of the hinged connections between precast elements.

There is a strong argument as for the necessity of adopting norms in this field – the design workshops do not have nor research funds or the know-how, while the state does. In the absence of norms, hinged connection will be further made on the acceptance principle of a certain solution by the local/regional designers’ community.

Keywords: precast; connection; beam; column; design.

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1. Introduction

Connections between precast concrete elements were always a challenge for the design engineers in view of buildings stability. In many cases these connections were successful, therefore they were taken over and used by the design engineers' community.

But, unfortunately, there were cases when, due to the use of inappropriate connections, the result was the collapse of some prefabricated elements.

Such situation took place during the 2012 Emilia earthquake where most of precast structures were affected causing a direct loss of 1 billion euros and an indirect one of 5 billion euros (Magliulo *et al.*, 2014), resulting from the interruption of the industrial activity. An important aspect to be mentioned is that a part of the structures was made by beams to columns friction connections, precisely by simple contact beams, but some structures were provided with joint connections between beams and columns. Fig. 1 is presenting the way that these connections failed.

a *b*

Fig. 1 – Failed precast connections (Magliulo *et al.*, 2014); *a* – Dowel beam-to-column connection failure and *b* – consequent loss of support of the beam from column.

The importance of the connections is in the same measure as of the precast elements that are connected, without these connections it is not possible to speak of a building as a whole because the connection has the role to take over the loads which are acting on the building and to transmit through the precast elements to the foundation.

Over the time, a great attention was paid to the connections between precast elements of residential and commercial buildings, parking areas, offices, all multi storey buildings with fixed connection. As for the industrial, logistic, zootechnical buildings, that is in case of all ground-floor buildings, connections were not so highly considered as in the case of the above-mentioned buildings. In case of ground-floor buildings, the connections have not evolved spectacularly over time and are made in the same way, with small differences.

Ground floor buildings are an indeterminate one-degree static structure, where columns are base fixed and main beams and purlins are hinged connected to the columns. This simple static scheme speeds up the assembling of this kind of buildings, mostly due to the hinged connections of the roof elements. This static scheme requires a standard solution (modular design, precast elements standardization, standard design of precast elements connections),solution that leads to repeatability and accumulation of experience in assembly on one hand, to lower cost and quality assurance on the other hand.

As company product standardization is already a reality, modular design represents a matter of time, the remaining part is connections standardization.

2. Numerical Evaluation

In the following paragraph three connection types will be evaluated agreed to be articulated by one or another design engineers' community. All three connections are at roof level between roof main beam and column.

All three types of connections are being evaluated as follows:

– Connection response to the roof elements and snow load to the main beam;

– Connection response to the temperature difference request;

– Connection capacity.

Thus, we considered the following hypotheses:

– Beam height in the contact area is 100 cm;

– Beam deformation in the middle from roof elements and snow load is 4 cm;

– Length of the main beam was considered to 2400 cm;

– It was considered an assembly temperature of 50°C and an operating temperature of 200°C;

– Concrete strength is 50 MPa;

– Yielding limit is 500 MPa.

In case that the main beam would be simply supported, a rotation and displacement would be happening like in Figs. 2 and 3.

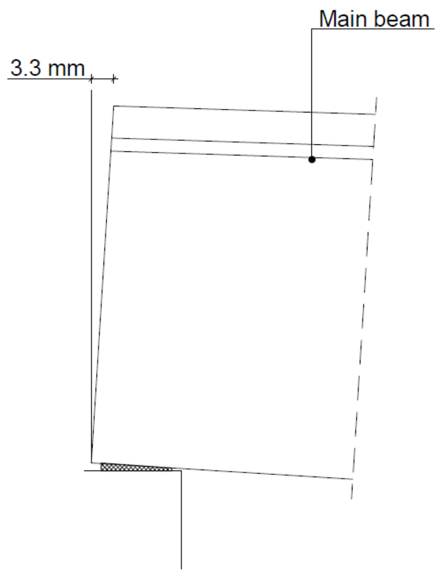

Fig. 2 – End-beam rotation as a result of vertical loads.

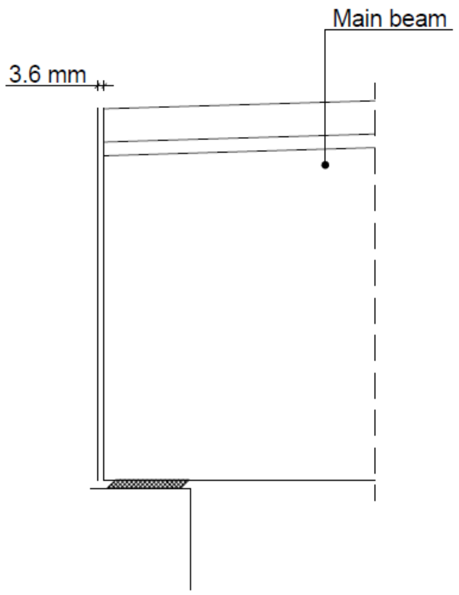

Fig. 3 – End beam displacement due to temperature difference.

2.1. Connection Type 1 (CT1)

CT1 connection, from Fig. 4, consists of one or two horizontal bolts, introduced through holes at the end of the main beam and at the end of the column. The main beam is seated on the column through a neoprene pad or a steel plate.

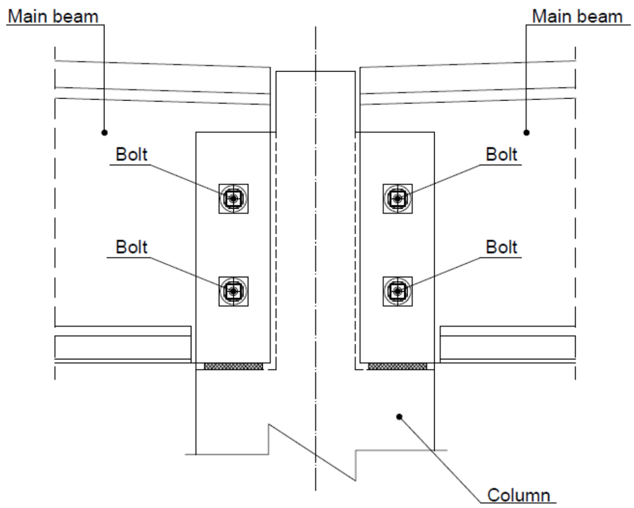

Fig. 4 – CT1 connection.

The gap through which the bolts pass is 1-2 cm in diameter larger than the diameter of the bolts, and the space between the beam and the column is approximately 1-2 cm, due to mounting tolerance conditions as can be seen in Fig. 5. The space between the beam and column are not filled with mortar.

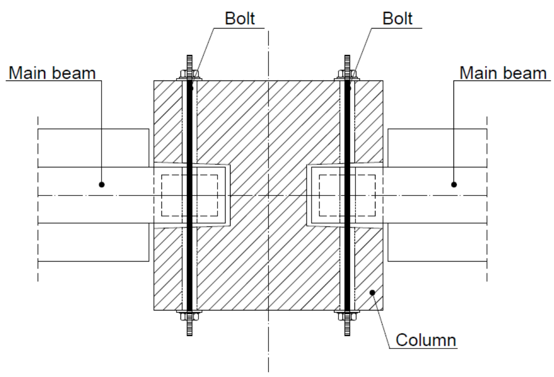

Fig. 5 – CT1 connection.

As it can be seen, CT1 connection is not subjected to vertical load efforts and temperature difference, but contrarily, this is a simple supported beam connection till the space between bolts and precast elements is consumed. The space is consumed in case of a horizontal demand, one of the effects being the ram type connection. The response of this connection can be seen in Figs. 6-8 considering a monotonic load.

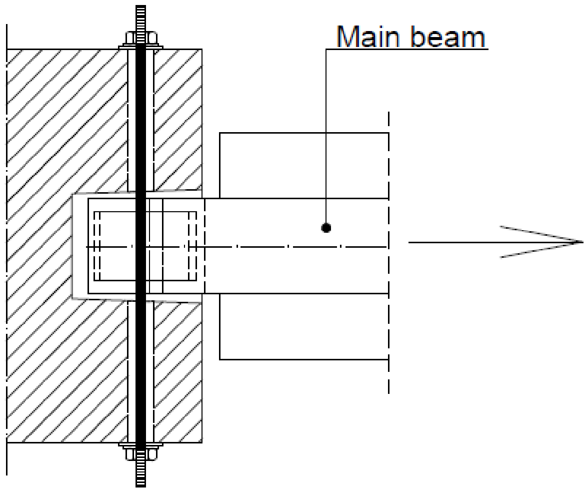

Fig. 6 – Due to beam displacement, the beam is in contact with the bolt.

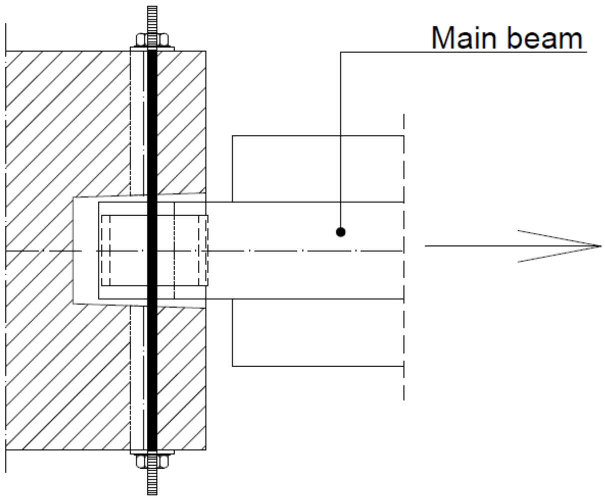

Fig. 7 – Due to the fact that the beam continues to displace,
the bolt contacts the column.

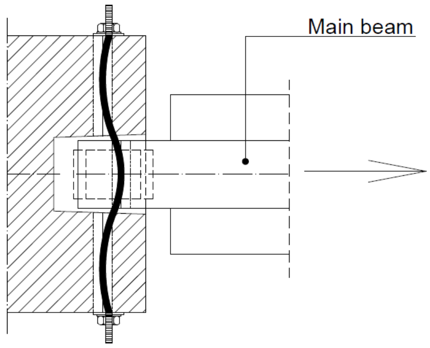

Fig. 8 – Due to the fact that the beam continues to displace, the bolt begins deforming.

The CT1 connection becomes a hinged connection after 2-4 cm beam's displacement, depending on the space between bolts and precast elements. Therefore, until connection starts working, entire horizontal load is overtaken by the column on the other side of the beam, after diminishing the space between the beam and the column with 1-2 cm, resulting here also a hammer effect on the connection and on the column.

This is when we can approximate bolts connection capacity to horizontal monotonic efforts as follows:

– calculate the maximum rotation of the bolt section at the yielding limit;

$$\theta = \frac{\varepsilon}{\Phi}$$

where Φ is bolt's diameter [m] and ε = yielding/elasticity modulus resistance.

– if θ permits deformation from Fig. 8 and the precast elements hole are created by embedding metal pipe with a considerable web thickness, then the bolt's static scheme is represented in Fig. 9.

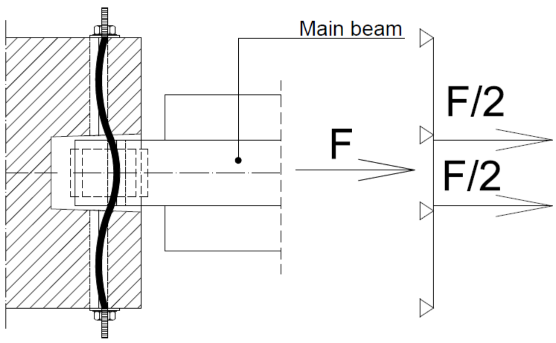

Fig. 9 – Bolt's static scheme.

– if the angle θ allows deformation of Fig. 10 and the precast elements hole are created by embedding metal pipe with a considerable web thickness, then the bolt’s static scheme is represented by Fig. 10.

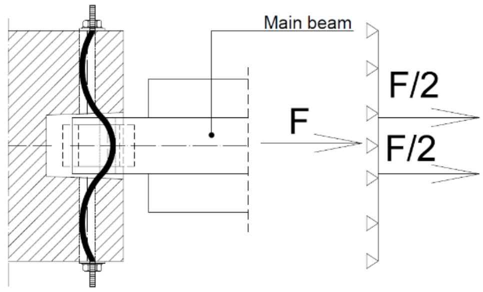

Fig. 10 – Bolt static scheme.

From the static schemes below can be determinate bolt’s bearing capacity, steel washer and/or thread net area bearing capacity and concrete section splitting capacity, the minimum value representing connection bearing capacity to a static load.

2.2. Connection Type 2 (CT2)

CT2 connection, represented in Fig. 11, consists of two column embedded vertical dowels introduced into the sleeves at the end main beam. The main beam is seated on the column through a neoprene pad or a steel plate. After the positioning of the main beam the sleeves are filled with mortar. Mortar's resistance will be at least as the minimum resistance from beams and columns concrete resistance.

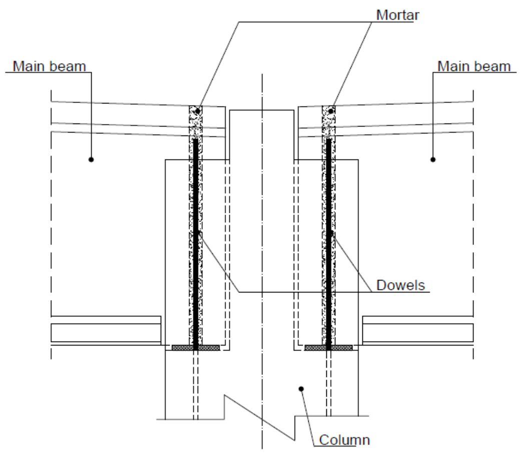

Fig. 11 – CT2 connection.

The sleeves are 1-2 cm larger than dowels diameter, and the beam-to-column space is approximately 1-2 cm, for reasons at assembling. The beam-to-column space is recommended to be filled with mortar. The connection capacity can be determined according to fib Bulletin 48.

In the first phase we determine the connection response to vertical loads (Fig. 12).

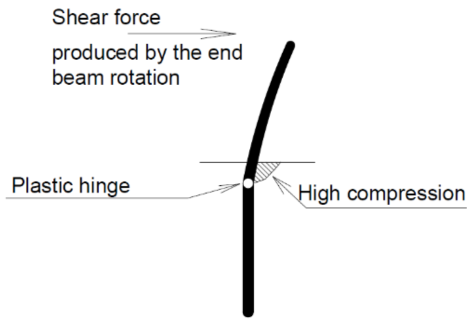

Fig. 12 – CT2 connection behaviour represented in fib Bulletin 43.

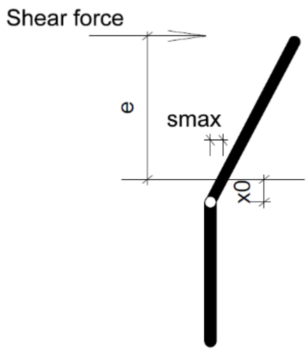

Fig. 13 – Model to determine concrete embedded dowel deformation capacity represented in fib Bulletin 43.

According to calculus method presented in fib Bulletin 43, in case of beam ends rotation due to gravitational loads, the $s_{\max}$ displacement necessary to capable shear effort mobilization is:

$$s_{\max} = 0.05\Phi$$

where Φ is the dowel diameter. For a diameter 25 mm results $s_{\max}$ = 1.25 mm. In other words, for a bolt of 25 mm diameter and a deformation of 1.25 mm a plastic hinge will form according to Fig. 12.

Fib Bulletin 43 is also presenting an alternative calculus proposal of the $s_{\max}$ displacement necessary to mobilization of capable shear effort:

$$s_{\max} = \theta crit \cdot x_0;$$
$$\theta crit = \frac{k_r \cdot \varepsilon_{sy}}{\phi};$$

The k_r coefficient reflects the bend distribution.

$$x_0 = \frac{F_{vR}}{3 \cdot \alpha_0 \cdot f_{cd} \cdot \phi};$$

$$\alpha_0 = 1, simplifying;$$

$$f_{cd} = \frac{50 \text{ MPa}}{1.5} = 33 \text{ MPa};$$

$$f_{yd} = \frac{500 \text{ MPa}}{1.15} = 434.7 \text{ MPa};$$

$$\phi = 25 \text{ mm};$$

$$F_{vR} = \alpha_0 \cdot \alpha_e \cdot \phi^2 \cdot \sqrt{}(f_{cd} \cdot f_{yd});$$

$$\alpha_e = \sqrt{1 + (\varepsilon \cdot \alpha_0)^2} - \varepsilon \cdot \alpha_0;$$

$$\varepsilon = 3 \cdot e \cdot \sqrt{}(\frac{f_{cd}}{f_{yd}});$$

e = 100 cm, beam height in the contact area;
Thus, it results:

$$\varepsilon = 3 \cdot 1 \cdot \sqrt{\frac{33}{437.7}} = 0.827;$$

$$\alpha_e = \sqrt{1 + (0.827 \cdot 1)^2} - 0.827 \cdot 1 = 0.47;$$

$$F_{vR} = 1 \cdot 0.47 \cdot 0.025^2 \cdot \sqrt{33 \cdot 434.7 \cdot 10^{12}} = 35182.74 \text{ N};$$

$$x_0 = \frac{35182{,}74}{3 \cdot 1 \cdot 33 \cdot 0.025 \cdot 10^6} = 0.014 \text{ m} = 14 \text{ mm};$$

$$\theta crit = \frac{1 \cdot \frac{500 \cdot 10^6}{200 \cdot 10^9}}{0.025} = 0.1 \text{ rad};$$

$$s_{\max} = 0.1 \cdot 14 = 1.4 \text{ mm} = 0.056 \cdot \Phi;$$

A first remark shows that in case of an end beam rotation due to vertical loads, we can state that the bolt stress is little and it can be neglected (Fig. 14).

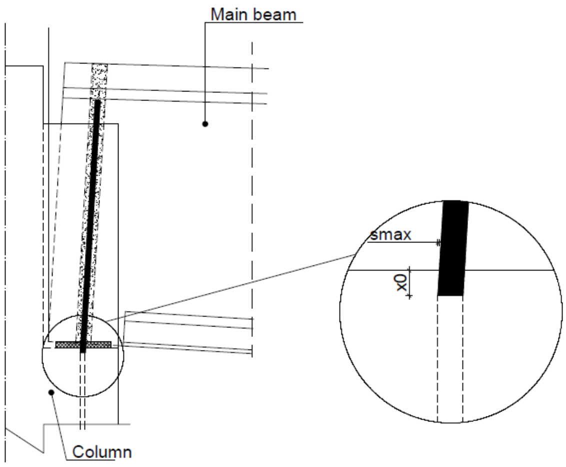

Fig. 14 – Maximum displacement where dowels resistance is below the yielding limit.

During next phase we determine the connection response to temperature difference loads (Fig. 15).

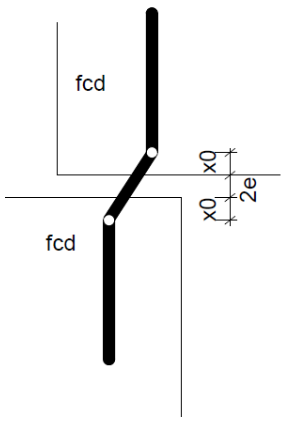

Fig. 15 – CT2 connection behaviour represented in fib Bulletin 43.

According to calculus method represented fib Bulletin 43, in case of beam shortage due to temperature difference, $s_{\max}$ displacement necessary to capable shear effort mobilization is:

$s_{\max}$ = 0.10 Φ, where Φ is the dowel diameter. For a 25 mm diameter it results $s_{\max}$ = 2.50 mm. In other words, for a 25 mm bolt diameter and a 2.50 mm deformation a plastic hinge will form according to Fig. 16. In this case we notice that the maximum dowel deformation (2.50 mm) is smaller than the end beam displacement resulting from temperature difference (3.60 mm - Fig. 3), so the connection bearing capacity could be exceeded.

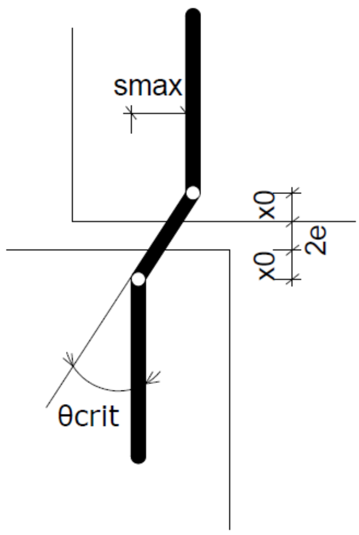

Fig. 16 – maximum displacement presented fib Bulletin 43.

The fib Bulletin 43 also presents an alternative calculus proposal of $s_{\max}$ displacement necessary to capable shear effort mobilization:

$$s_{\max} = \theta crit \cdot 2 \cdot x_0;$$

$$\theta crit = \frac{k_r \cdot \varepsilon_{sy}}{\phi};$$

The k_r coefficient reflects the bend distribution.

$$x_0 = \frac{F_{vR}}{3 \cdot \alpha_0 \cdot f_{cd} \cdot \phi};$$

$$F_{vR} = \alpha_0 \cdot \alpha_e \cdot \phi^2 \cdot \sqrt{(f_{cd} \cdot f_{yd})};$$

e = 10 mm, half of the neoprene pad or steel plate thickness;

Therefore, it results:

$$\varepsilon = 3 \cdot 0.01 \cdot \sqrt{\frac{33}{437.7}} = 0.008;$$

$$\alpha_e = \sqrt{1 + (0.008 \cdot 1)^2} - 0.008 \cdot 1 = 0.99;$$

$$F_{vR} = 1 \cdot 0.99 \cdot 0.025^2 \cdot \sqrt{33 \cdot 434.7 \cdot 10^{12}} = 74260 \text{ N};$$

$$x_0 = \frac{74260}{3 \cdot 1 \cdot 33 \cdot 0.025 \cdot 10^6} = 0.03 \text{ m} = 30 \text{ mm};$$

$$\theta crit = \frac{1 \cdot \frac{500 \cdot 10^6}{200 \cdot 10^9}}{0.025} = 0.1 \text{ rad};$$

$$s_{\max} = 0.1 \cdot 30 \cdot 2 = 6 \text{ mm} = 0.24 \cdot \Phi;$$

In this case it can be noticed that, according to alternative calculus proposal, the admitted deformation of the dowels have not been reached due to the temperature difference, but we must consider that 50% from the admitted deformation was reached.

After the above checking the shear resistance of the connection can be estimated according to “Design Guidelines for Connections of Precast Structures under Seismic Actions” (Negro and Toniolo, 2012) which underlines the fact that tensile stress due to other possible contemporary effects on the dowels should be taken in consideration. We also consider that according to “Design Guidelines for Connections of Precast Structures under Seismic Actions” (Negro and Toniolo, 2012) there are calculus formulas only for shear forces:

a) dowels bearing capacity;

b) spalling of the concrete edge of the beam;

c) spalling of the concrete edge of the column.

As in case a minimum distance is respected between dowels and ends and column and beam ends are being reinforced accordingly, then the failure is taking place mostly in the dowels (Negro and Toniolo, 2012).

Thus, we have the following formula:

$$R_d = 0.90 \cdot n \cdot \Phi^2 \sqrt{f_{cd} \cdot f_{yd} \cdot (1 - \alpha^2)},$$

Where:

– Φ is the vertical bolt diameter, in this case 25 mm;

– n is the number of bolts, in this case 2;

– α is the ratio σ / f_{yk}, σ being the initial bolts effort state.

In case there are not efforts due to temperature difference, then:

$$R_d = 0.90 \cdot 2 \cdot 0.025^2 \sqrt{33 \cdot 434.7 \cdot 10^{12} \cdot (1 - 0^2)} = 134742 \text{ N} = 13.47\ to;$$

$$R_{Rd} = \frac{R_d}{\gamma_R} = \frac{13.47\ to}{1.2} = 11.22\ to,$$

where $\gamma_R = 1.2$ for DCH, and R_{Rd} is dowels shear capacity.

In case of stress due to temperature difference, then it is necessary dowels initial state calculus and we can proceed as follows:

– if the end beam shortage is 3.6 mm, results $s = 3.6$ mm. For $s = 3.6$ mm (60% of $s_{\max}$), we can approximate σ = 260.82 MPa, thus $\alpha = 0.52$ resulting:

$$R_d = 0.90 \cdot 2 \cdot 0.025^2 \sqrt{33 \cdot 434.7 \cdot 10^{12} \cdot (1 - 0.52^2)} = 115092\ N = 11.51\ to;$$

$$R_{Rd} = \frac{R_d}{\gamma_R} = \frac{11.51\ to}{1.2} = 9.60\ to.$$

Dowels shear capacity is compared to the shear resistance of the critical section of the column (Negro and Toniolo, 2012), but we must consider that plastic hinge formation to the column's base is not the effect of a diminished horizontal effort, but represents columns post-elastic deformation (empirically established), therefore it is recommended that the dowels shear capacity should be compared to the shear resistance of the critical section of the column multiplied with the behavior factor q.

An important aspect related to this connection represents the bearing pad which can be made through a neoprene pad or a steel plate. In case of a neoprene pad, the end beam can rotate due to neoprene deformability property, but due also to this property the transfer of vertical loads can be happen through monolithization. In case of a steel plate bearing pad, due to steel rigid property, the vertical loads transfer cannot be realized through monolithization, but instead it is possible to form a fix connection that can weaken more the dowels (Fig. 17).

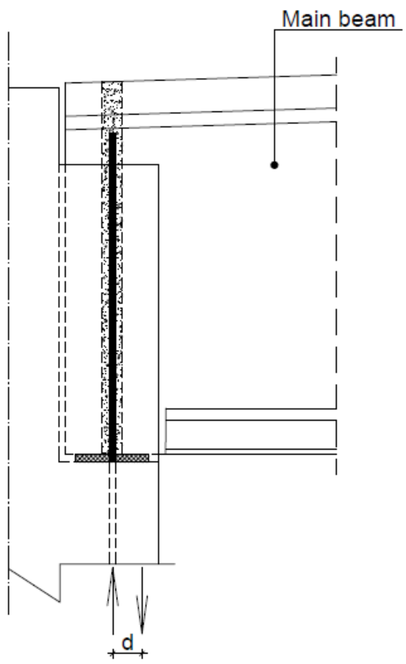

Fig. 17 – Semi – rigid connection.

When evaluating this connection type bearing capacity, we must consider that the earthquake has as an effect a top displacement of the column due to base rotation (Fig. 18).

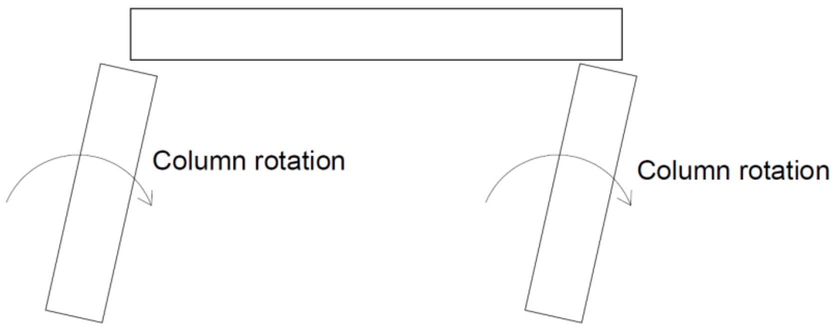

Fig. 18 – Deformed frame.

This displacement at the top results in the rotation of the support (Fig. 19), a rotation that can produce unforeseen effects in the dowels, like tension.

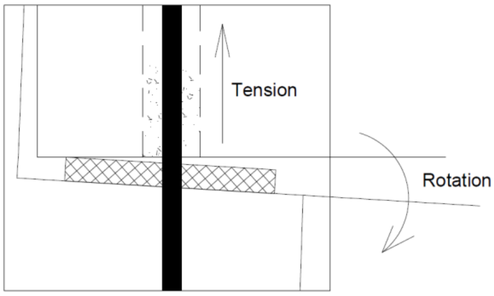

Fig. 19 – Unforeseen effects in the dowels.

Because in the state before the earthquake a part of the elastic deformation was consumed, the limitation of the rotation of the support, implicitly of the movement at the top of the structure, becomes a necessity.

2.3. Connection Type 3 (CT3)

The CT3 connection, Figs. 20-22, consists of two vertical dowels embedded in the column which are inserted through some gaps at the end of the main beam. In addition, there are one or two horizontal bolts that are inserted through some gaps at the end of the main beam and at the end of the column. The main beam rests on the column by neoprene or steel pad. After positioning the beam, the vertical and horizontal bolts and the space between the beam and the column are monolithized with mortar. The strength of the mortar will be at least as low as the minimum strength of the concrete in the beams and columns.

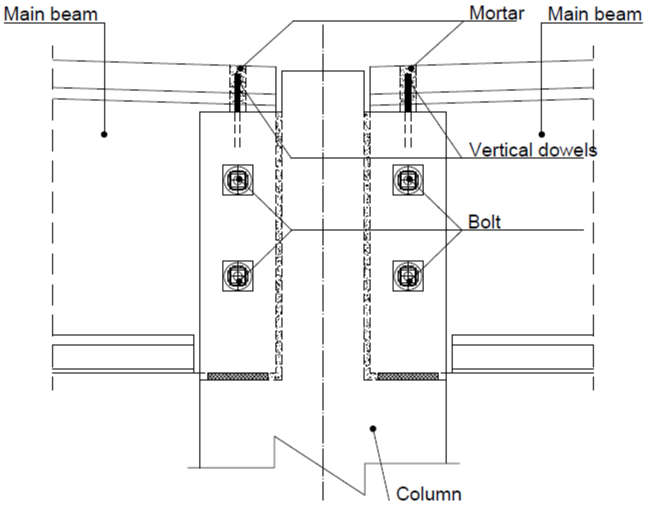

Fig. 20 – CT3 connection.

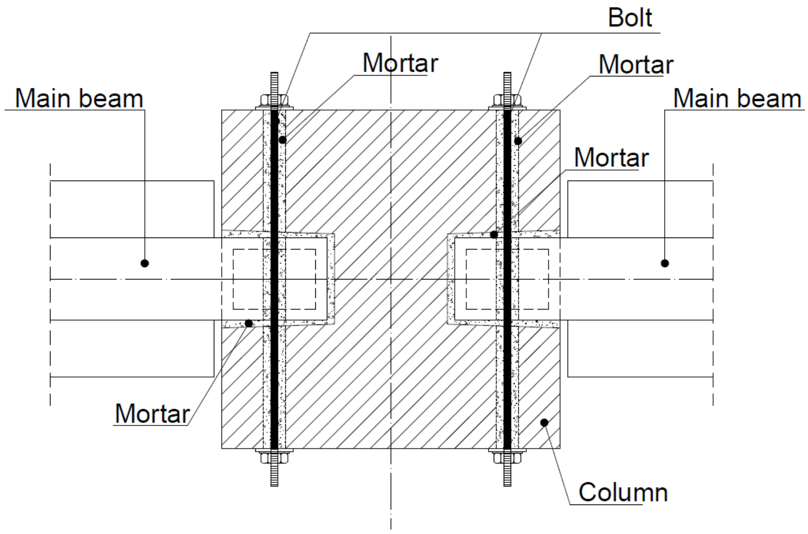

Fig. 21 – CT3 connection.

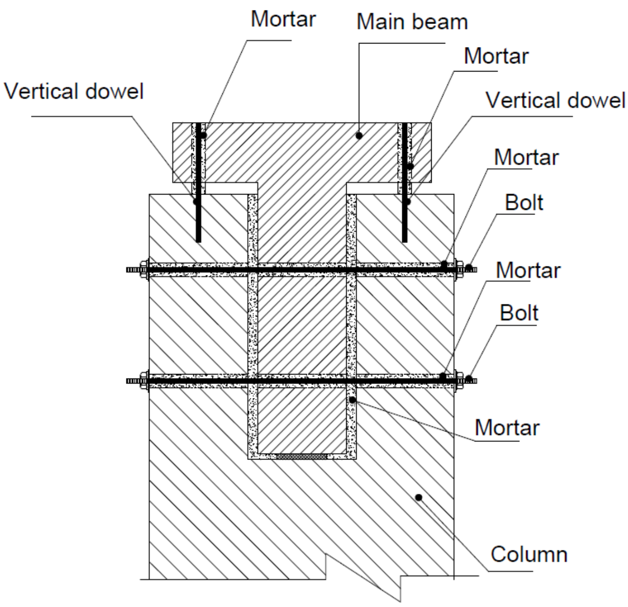

Fig. 22 – CT3 connection.

The CT3 connection by way of realization can be considered a semi-fixed connection for vertical loads, because the arrangement of the vertical dowels and the monolithization of the space between the beam and the column prevented the rotation of the end of the beam when the vertical dowels were deformed enough to activate the shear capacity. As the shear capacity of the vertical dowels is small and the bending moment is considerable, it is estimated that these bolts do not make a significant contribution to the bearing capacity of this type of connection at all.

Also, if the mortar does not penetrate all the gaps between the bolts and the prefabricated elements, this connection is transformed into connection type 1 (CT1).

If it is certain that the mortar has penetrated all the holes, then it is verified how much of the load-bearing capacity of the horizontal bolts embedded in the mortar was consumed by the temperature difference according to the above calculations, and the result thus obtained is compared with the seismic effort.

The calculation can be done according to Fib Bulletin 43 and Fig. 23.

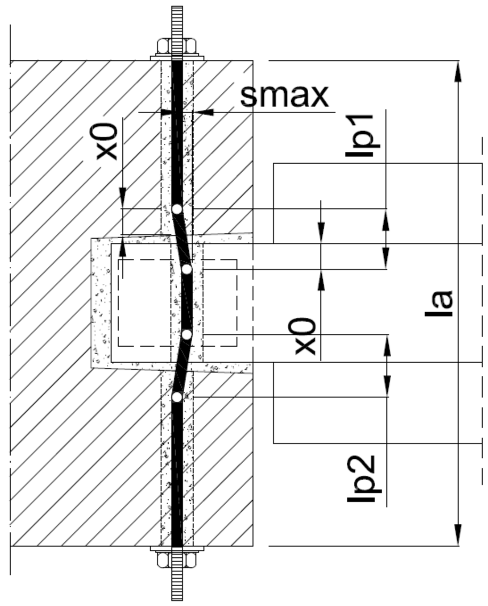

Fig. 23 – Maximum displacement of the horizontal bolt.

In this particular case the presence of washers and nuts having the effect of developing a tensioning effort must be taken into account.

On the one hand they have a beneficial effect by developing a friction between the reinforced concrete elements, and on the other hand they have a negative effect because the bolt is subjected both shear and tension.

The maximum elongation over which the bolt fails occurs is:

$$n = \sqrt{l_p^2 \quad + \left(\theta_{crit} \cdot l_p\right)} - l_p \;;$$
$$l_p = l_{p2} + l_{p2} = 4 \cdot x_0 + 2 \cdot e;$$

where e is the distance between the two precast elements.

$$x_0 = 30 \text{ mm}, l_p = 4 \cdot 30 + 30 = 150 \text{ mm}, n = 7.48e^{-4}\text{m};$$

If the elongation is supposed to be uniformly distributed between the washers, then:

$$\sigma = \frac{n \cdot E}{l_a} = \frac{7.48e^{-4} \text{ m} \cdot 200 \cdot 10^{-3}}{0.90 \text{ m}} = 166 \text{ MPa};$$

where l_a is considered to be 90 cm.

The horizontal bolt shear capacity is:

$$F_{vR} = \alpha_0 \cdot n \cdot \phi^2 \cdot \sqrt{f_{cd} \cdot f_{yd\,1}} + \mu \cdot \sigma \cdot A_S;$$

$$f_{yd\,1} = f_{yd} - \sigma;$$

$$F_{vR} = 1 \cdot 2 \cdot 0.025^2 \cdot \sqrt{33 \cdot (434.7 - 166) \cdot 10^{12}} + 0.7 \cdot 166 \cdot 10^6 \cdot 4.91 \cdot 10^{-4} = 117706 \text{ N} + 81506 \text{ N} = 19.92 to;$$

In order to establish the capacity of the connection, the following observations are required:

– from the rotation of the end of the beam due to the gravitational loads there is a displacement of approximately 2.5 mm at the second row of bolt, from bottom to top, to which is added a displacement of 3.6 mm from the temperature difference. Because the $s_{\max}$ displacement is 6 mm, the second row of bolt is plasticized.

– the first row of the bolt, if there is a temperature difference, a deformation s = 3.6 mm occurs, approximately 50% of the maximum deformation ($s_{\max}$), finally resulting the shear capacity of approximately 10to.

If, simplistically (recommended in the design), $s_{\max} = 0.1\Phi = 2.5$ mm is considered, then all the horizontal bolts are plasticized.

A final aspect should be taken in consideration, the column displacement.

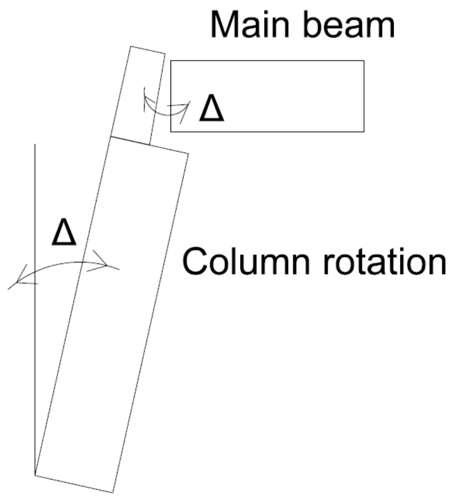

Fig. 24 – Additional shear effort in the first row of bolt.

As can be seen in Fig. 24 the column rotation introduce an additional shear effort in the first row of bolt due to the presents of the mortar poured between the beam and the column, so a fixed connection is forming when the shear capacity of the bolts is reached and/or the s_{max} = 6 mm displacement is reached (Fig. 23). In other words, in order to have a connection, the maximum displacement s_{max} should be limited to 6 mm.

If we consider the distance from the bolt to the center of the compressed area to be 600 mm and there is a temperature differences, then a maximum value of tgΔ is 0.004 or the maximum displacement of the top of the column should be limited to 0.004H, where H is the length of the column. If there are not temperature differences, then the maximum displacement of the top of the column should be limited to 0.01H.

If, simplistically (recommended in the design), $s_{max} = 0.1\Phi = 2.5$ mm is considered and have no temperature differences, then the maximum displacement of the top of the column should be limited to 0.004H.

3. Conclusions

Choosing and designing the connections between the prefabricated elements is a decision that involves a great responsibility.

At the same time, it should be noted that design engineers face an acute lack of rules to facilitate this, on the one hand, and on the other hand are under pressure to maintain the simplicity of prefabricated ground floor structures.

That is why there has been a uniformity of the solutions regarding the realization of the connections between the prefabricated elements at the ground floor type structures, but the latest events have shown their importance.

Solutions to increase the bearing capacity of CT2 and CT3 connections would be the following:

– reducing the rotation of the end of the main beam by increasing its section and by corresponding reinforcement with strands;

– either avoiding large temperature differences between installation and operation, or reducing the length of the main beams;

– limiting the column displacement to 1% of its height or even less;

– monolithization of the space between the beam and the column at the CT2 type connection;

– making control holes to ensure that the mortar penetrates all the holes in the case of the CT3 connection, especially in the first row of horizontal bolt;

– performing monolithization in case of CT3 connection with qualified personnel and under strict supervision;

– in the case of CT2 and CT3 type connections, it should be considered in the dimensioning of the column in which the beam pushes, the fact that the maximum mobilization of the connection, only from the point of view of the bolt, takes place after consuming its maximum deformation;

– Proper reinforcement of the gaps through which the bolts are inserted.

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EVALUAREA UNOR CONEXIUNI ARTICULATE
UTILIZATE ÎN MOD FRECVENT LA
CONSTRUCŢIILE PREFABRICATE DE TIP PARTER

(Rezumat)

Lucrarea prezintă o evaluare a trei tipuri de conexiuni între grinzile principale prefabricate și stâlpii prefabricați utilizați în mod obișnuit în România considerate a fi articulate.

Evaluarea celor trei tipuri de conexiuni se bazează pe definiția teoretică a unei conexiuni articulate și pe rolul inginerului proiectant de a stabili soluții, care se apropie foarte mult de ipotezele lor teoretice, pentru a oferi o predictibilitate ridicată și o controlabilitate a structurilor prefabricate.