The Seismic Behavior of An Asymmetric Exterior Precast Beam-Column Connection

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SUMMARY:
Two types of asymmetric precast beam to column exterior connections have been tested in the Structural and Earthquake Engineering Laboratory of Istanbul Technical University (ITU) and a complementary theoretical work has been carried out to generalize the results achieved experimentally, all within the framework of SAFECAST Project which is an FP7 project supported by European Commission[1]. *Four* 1/2 scale so called *industrial type* beam to column wet connections have been exposed to monotonic and cyclic displacement increments in the first stage of the research program together with other *four* 1/1 scale so called *residential type* beam to column connections. This paper consists of the experimental and theoretical findings pertinent mainly to the *industrial type connections*.

Keywords: precast, asymmetry, exterior beam to column connection, earthquake, macro elements.

1. SPECIMENS

Half scale *industrial type* beam to column connections have been scaled down from a real beam–column connection of an industrial building designed so that it satisfies all the requirements of Turkish design codes. Since the main girder in that building was a pre-stressed precast girder, pre-stressing used in the specimens' girders as well. Smallest size hollow core slab elements which can be found in the market were suitable for the ½ scale specimens.

The lower and the upper part columns of *industrial type* connection are cast at the same time leaving an indentation at the middle where the top main reinforcement bars of semi-cast girder with protruded stirrups, will be placed later on in two layers. In this type of connection the built in steel plate at the bottom edge of the girder will be welded on the steel plate placed properly on the corbel standing out from lower column, Fig.1.1.

Two small transversal beam pieces have been used in the connection zone in order to have better representation of the real beam to column connections since it is known that there exists a certain amount of confinement effect on the critical zone provided by the transversal reinforcements coming from transversal beams, Fig.1.2.

After having placed the hollow core slab elements near by the sides of main girder, wire mesh is used on top, before pouring the topping concrete. Wire meshes are effective in the critical zone to a certain extent.

[1] The genaral title of the FP7 Project is *Performance of Innovative Mechanical Connections in Precast Building Structures Under Seismic Conditions. Grant agreement number: 218417, Project Coordinator: Dr Antonella Colombo – ASSOBETON.*

Figure 1.1. Industrial type beam to column connection

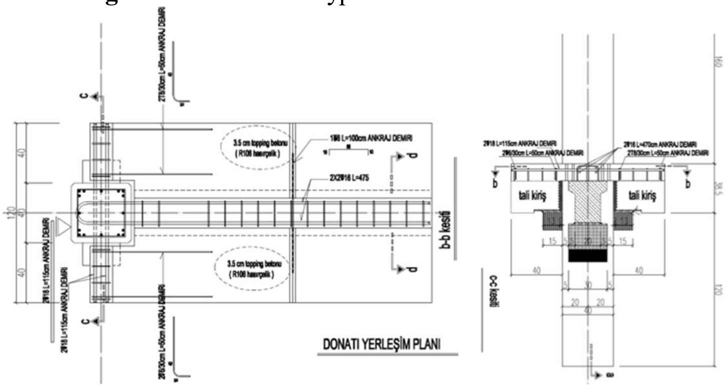
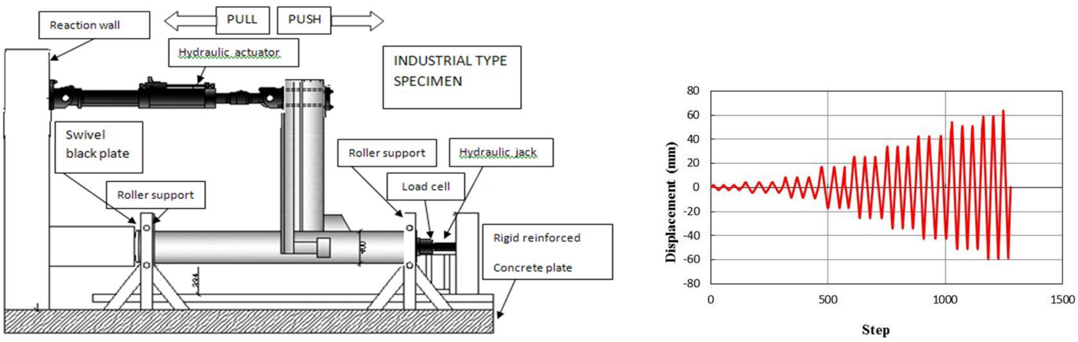


Figure 1.2. Top view and a section of industrial type specimens

2. TESTING SET-UP

The specimens are tested after they have been rotated 90 degrees in counter clockwise direction. In other words the columns of specimen will be in lateral position and will be loaded axially by the hydraulic jack shown in Fig. 2.1. The applied axial force to the columns was 250 kN which is in the range of 5-10% of the axial load capacity of columns. An MTS hydraulic actuator is connected to the beam which is in upright position .The supports of columns and the swivel head of the actuator are providing hinge connections to the specimen ends. Hydraulic jack is controlled in displacement mode following the proposed displacement protocol given in Fig. 2.1. One side view of instrumentation is presented in the same figure.



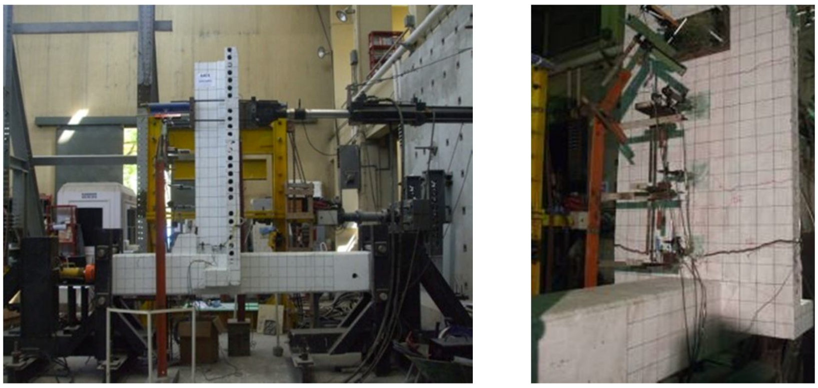


Figure 2.1. Testing setup and displacement protocol for cycle excitation

3. TEST RESULTS

Because of the asymmetry in beam sections two monotonic tests have been carried out in two opposite directions. These independent tests have been finished turning back to complete the cycle. The other two specimens have been tested in a cyclic way following up the displacement protocol given in Fig. 2.1. The results achieved at the end of four tests are given in Fig. 3.1. Plastic deformations have been accumulated in the critical section of the beam just in front of the corbel as they are expected. Stirrups welded on the plate at the edge of the beam were broken as it can be seen in Fig. 3.2. One of the important observations was the penetration of big cracks into the beam-column connection zone.

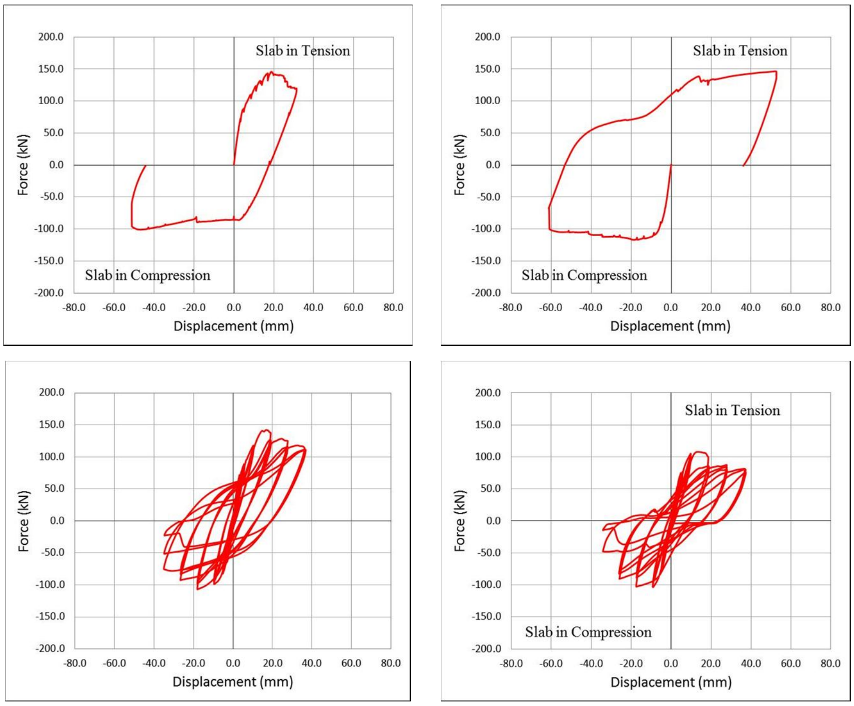


Figure 3.1. Monotonic and cyclic test results

After having completed these four tests two more specimens have been prepared and tested to see the efficiency of the modification done to improve the performance of the connections. Modifications in the details of connections have been based on the observed deficiencies, aiming to prevent premature failures and having more stable hysteresis loops with higher strength and ductility. For that reasons, *i*-Stirrup diameters have been increased from 8 to 10 mm, *ii*-Stirrups have been welded by sides to a vertical plate which is welded on the steel plate in perpendicular direction, Fig. 3.3, *iii*-Thickness of the welding has been limited by 0.3ϕ, where ϕ is the diameter of reinforcement bar, *iv*-Equivalent carbon content has been checked to see whether it is lower than the acceptable limit or not before fabrication starts.

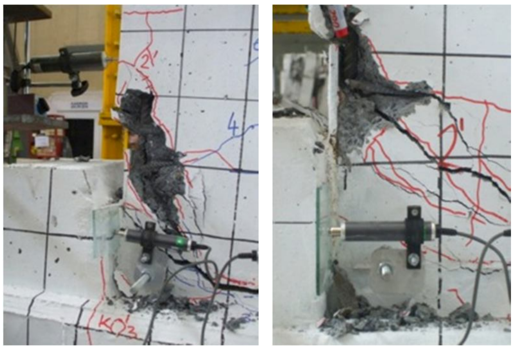


Figure 3.2. Plastic deformations penetrating into the connection zone and failure of stirrup weldings

Figure 3.3. Original and modified plate used for stirrup and longitudinal bar connections

Two improved specimens prepared accordingly have been tested for cyclic excitation controlling the lateral displacements. The force displacement relation and the damages shown in Fig. 3.4 have been observed without having cracks in beam column connection zone and without having early failure of stirrup welding.

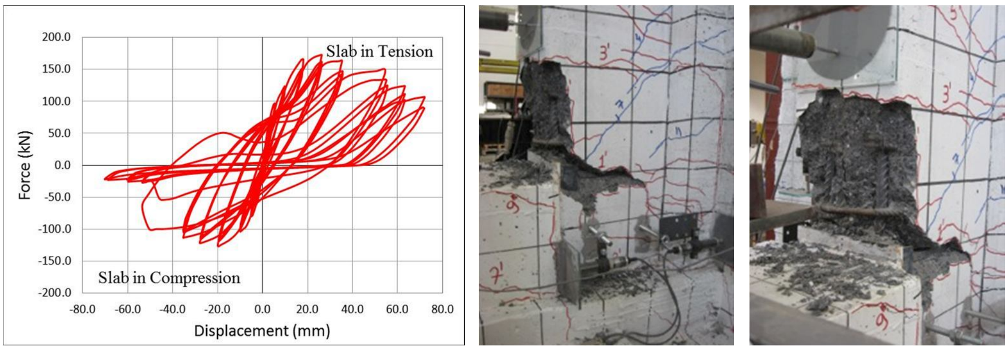


Figure 3.4. Typical failure modes of improved specimens

4. TEHORETICAL WORKS

Both OpenSees and SAP2000 softwares, which are very well known computer programs, as well as DOC2D, which is a locally developed software, have been utilized first to obtain theoretical responses which are matching best with the experimental findings. The generalized results obtained have been employed to achieve the nonlinear response of a precast multistory frame subjected to lateral load increments.

4.1. Monotonic Loading

The computer program DOC2D has been employed for the simplified analyses of beam to column connection taking into consideration the material nonlinearity. The structural model, which consists of frame elements, is shown in Fig. 4.1. Theoretical and the corresponding experimental findings are displayed in Fig. 4.2 on the same graphics. It can be concluded that the experimentally obtained results can be reproduced by means of theoretical works introducing 1D nonlinear simple beam elements. The size of beam elements should be small enough to represent the expected behavior.

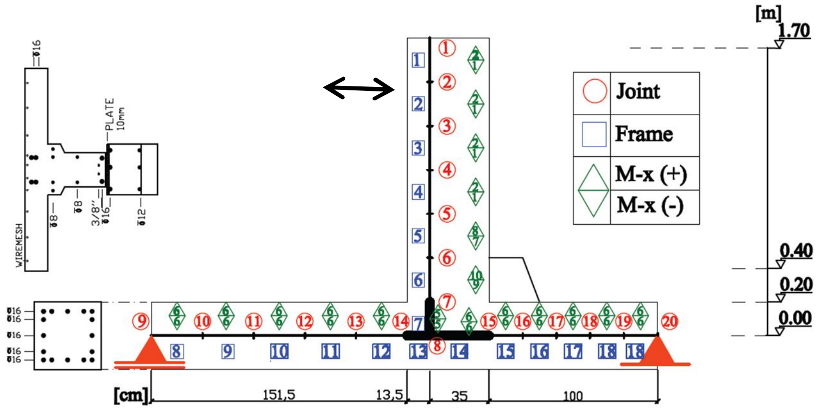


Figure 4.1. Structural model consists of 1D nonlinear beam elements

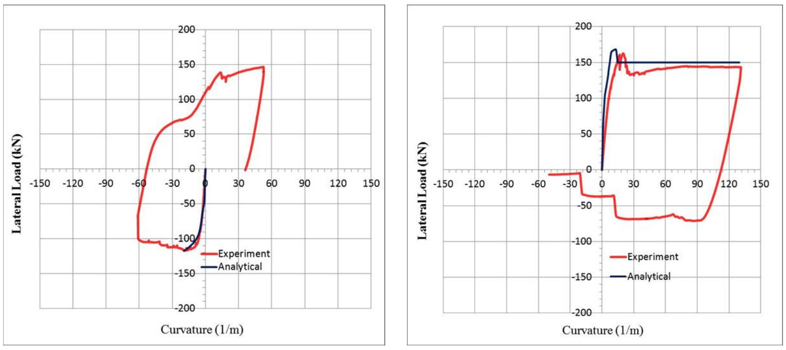


Figure 4.2. Comparision of theoretical and experimental findings for monotonic loading

4.2. Cyclic Loading

Two different approaches have been employed to achieve the cyclic response of the connection, namely *i*-Calibration efforts, *ii*- OPENSEES Hysteresis Loops and Pivot Models.

The following factors have been taken into consideration during the calibration efforts; i- Welding has diverse effect on the strength of deformed reinforcement bars when carbon content is relatively high, ii- The bars that do not have enough development lengths, iii- Hollows of hollow core slab elements in the critical section, iv- Pinched stress–strain diagrams of reinforcement bars, v- The effective thickness of unconfined concrete covers, vi- Force based elements versus displacement based elements. The final loop after tuning and the cumulative energy over the pseudo testing time are presented in Fig. 4.3.

After the calibration of the modeling option for further use in conducting numerical experiments, the next step was to find a set of plastic hinge properties to represent the force-deformation cycles accurately enough. The *Hysteretic Material*, as shown in Fig. 4.4, available in OpenSees is used for this purpose. The match between the plastic hinge behavior and specimen responses is also achieved by using the *Pivot Model* available in SAP2000 as shown in Fig. 4.4. The pivot model parameters used are $\alpha_1=\alpha_2=4$, $\beta_1=\beta_2=0.5$ and $\eta=0$.

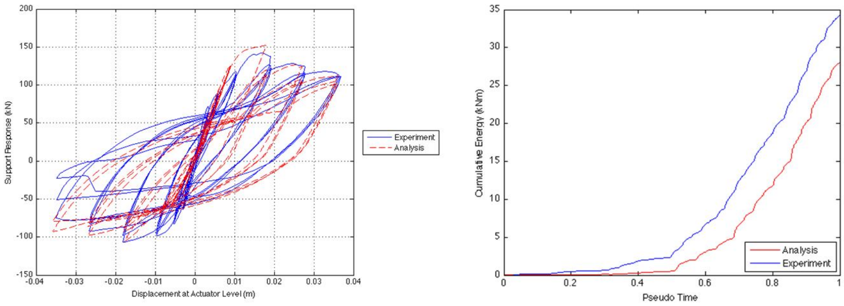


Figure 4.3. The test analysis results obtained by OpenSees (left) and cumulative energy comparison (right)

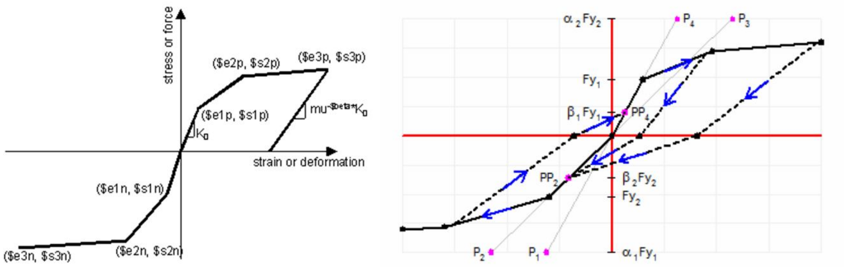


Figure 4.4. The *Hysteretic Material* in OpenSees on the left and the *Pivot Model* in SAP200 on the left

The results of the plastic hinge analyses for two programs are given together with the experimental results, Fig. 4.5.

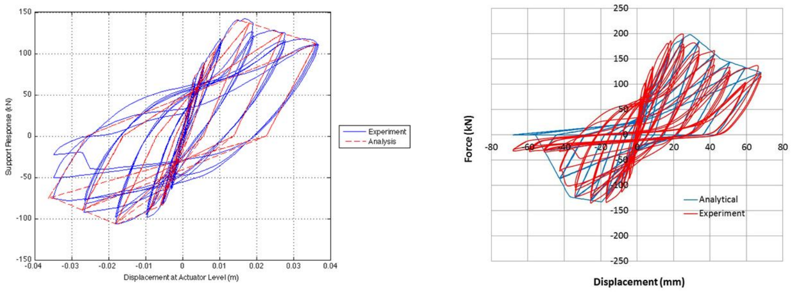


Figure 4.5. Matching the tests and the plastic hinge representations by Hysteretic Material in OpenSees on the left and pivot model SAP2000 on the right

The comparison of monotonic and cyclic behavior of the connection is presented in Fig. 4.6. There exists substantial difference in the behavior especially slab in compression case.

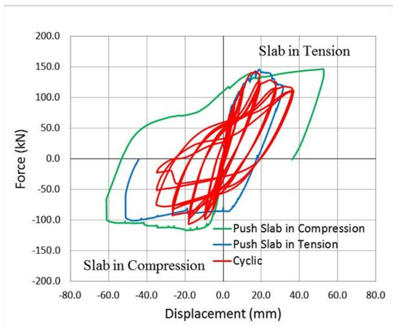


Figure 4.6. Comparisons of monotonic and cyclic behaviors of industrial type connection

The model shown in Fig. 4.7 is created by placing plastic hinges at the beams ends and at the bottom level of the columns, whilst the elements between the hinges are modeled as elastic members. The columns are 40×40 with 2% vertical reinforcement. The empirical parameters required to construct the hysteresis loops for the columns, such as the post-yield stiffness ratio, unloading and reloading stiffness values, are taken from (Yuce 2009). The beams of the frame have 48.5 cm depth, 20 cm width and 120 cm slab widths. In order to create an asymmetry to better represent the possible real design cases, the right end of the longer beam has been modeled to have the same behavior as the tests conducted at ITU while the beam ends in the middle of the frame have 70% of the strength of the tested specimen. Finally, the left end of the smaller beam has 50% of the strength of the tested specimen. In the hysteretic material properties for the beam ends, the beta unloading coefficient is 1.00, the stiffness and strength degradation coefficient, which are called "damage", are 1.0 meaning no degradation, and finally the pinching parameters are zero meaning no pinching. The ultimate moment capacity of the tested beam section is modeled as around 265 kNm in slab-in-tension and 175 kNm in slab-in-compression. The yield rotation is 1% in slab-in-tension and 0.6% in slab-in-compression.

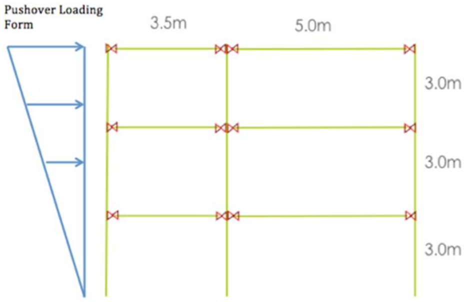
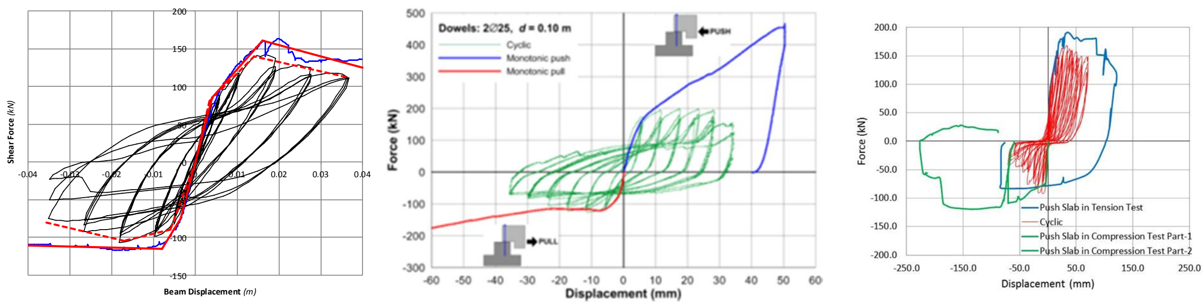


Figure 4.7. Two-bays two-story asymmetric planar frame

There are two purposes of this example. The first one is to underline the effects of running the pushover in different directions. Some researchers, among which is Smyrou 2008, report that the asymmetry in two different loading directions of a system is rather problematic in terms of design or assessment. The other purpose is to observe whether the effects of using monotonic or cyclic behavior to create a backbone curve have a significant effect on the overall behavior. The pushover plots presented in Fig. 4.8 exhibit an asymmetric behavior in positive and negative directions indicating a need to consider the issue of polarity in design. There exist also a significant difference between the two capacity curved based on different element backbone curves constructed on monotonic or cyclic test results, which is another issue that requires utmost attention.



Industrial type connection *Dowel connection* *Residential type connection*

Figure 4.8. Monotonic versus cyclic asymmetric behaviors of other type connections

The first yield of a member in the frame shown in Fig. 4.7 occurs at 7.6 cm top displacement, thus the overall ductility seen in Fig. 4.9 is in the order of 5. The ductility demand on each plastic hinge used to model the beams has been observed when the overall ductility has reached 5. The member ductility in different positions for different analysis options are given in Table 1. A back calculation performed shows that the maximum overall ductility that can be reached without any beam-end reaching its collapse limit state is only 4.2 for the examined example. This finding suggests that there is a requirement of a backward-control of the model by checking the ductility demands in each section after the analysis is completed and the target displacement is reached.

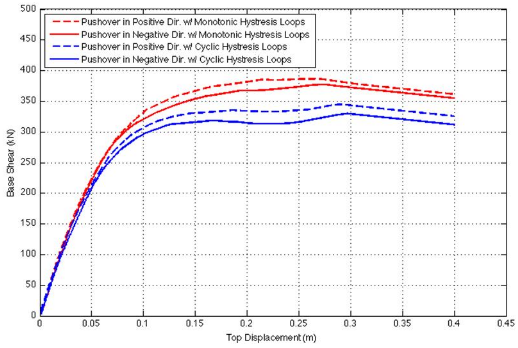


Figure 4.9. Push over curves in two opposite directions and pushover curves where the backbone curve is based on monotonic or cyclic tests

Table 4.1. Beam plastic ductility demands when the overall ductility is 5.0. Please note that the elements reaching collapse limit state are red highlighted.

		Left Beam Left End	Left Beam Right End	Right Beam Left End	Rigt Beam Right End
3rd Floor	Monotonic Props (+)	3.6	3.1	3.0	3.6
	Monotonic Props (-)	3.6	2.9	3.1	3.2
	Cyclic Props (+)	3.9	3.4	3.3	3.8
	Cyclic Props (-)	3.8	3.3	3.5	3.5
2nd Floor	Monotonic Props (+)	4.1	4.0	3.9	4.2
	Monotonic Props (-)	4.0	3.7	3.9	3.7
	Cyclic Props (+)	4.2	4.1	4.0	4.3
	Cyclic Props (-)	4.2	3.9	4.1	3.9
1st Floor	Monotonic Props (+)	4.5	4.1	4.1	4.5
	Monotonic Props (-)	4.3	3.8	4.0	4.0
	Cyclic Props (+)	4.4	4.0	4.0	4.3
	Cyclic Props (-)	4.2	3.8	4.0	3.9

5. BEHAVIOR MODELS

In both stages of hinged and fixed support, the shear force coming from the beam is assumed to go entirely on the corbel standing out from the column. The local detailing and design calculation of the corbel shall be made following the provisions of EC2. After the hardening of the cast-in-situ concrete in slab, provides a moment resisting support between the parts, with a dissymmetrical behavior for positive and negative moments. The loads applied after the hardening of the slab take their action on this moment resisting connection. The self-weight of the beam, included the upper concrete slab, acts on a simple hinged support arrangement.

Tests carried out indicate that monotonic and cyclic behavior of these connections is not only asymmetric as they are expected, but they are different as far as strength, strength degradation, stiffness, stiffness deterioration, pinching features are concerned. They are better be assumed different type semi-rigid connections in any kind of structural analyses such as modal superposition, linear and nonlinear time history or any equivalent way of analysis.

The following important observations done during the testing campaign will be the bases of design rules summarized below: *i.* Hollow core slabs are very well integrated to the subassembledge by topping and by protruded stirrups of beams, *ii.* No shear failure penetration into the connection zone has been observed for improved specimens, *iii.* Plasticization zones due to flexure are all in the critical sections of beams, namely after the corbel.

5.1. Failure Modes

The following failure modes can be recognized when the connection is subjected to *negative moments*;

- *a- Flexural failure of the connection referred to the yielding of the longitudinal upper bars*
- *b- Bond failure of the anchorage of the upper bars*
- *c- Longitudinal shear failure at the interface between precast beam and cast-in-situ slab*
- d- *Failure of the bottom connection between the girder and the supporting corbel.*

The following failure modes can be recognized when the connection is subjected to *positive moments*;

- *a- Flexural failure of the connection referred to the rupture of the bottom connection;*
- b- *Longitudinal shear failure of the interface between precast beam and cast-in-situ slab*;

5.2. Calculation Formulae

Ultimate resisting *negative moment* can be calculated by $M_{Rd}=A_{st} f_{yd} z'$ where A_{st} and f_{yd} are the total sectional area of the longitudinal upper bars and $f_{yd}= f_{yk} / \gamma_s$, respectively where f_{yk} is the characteristic yield strength of steel.

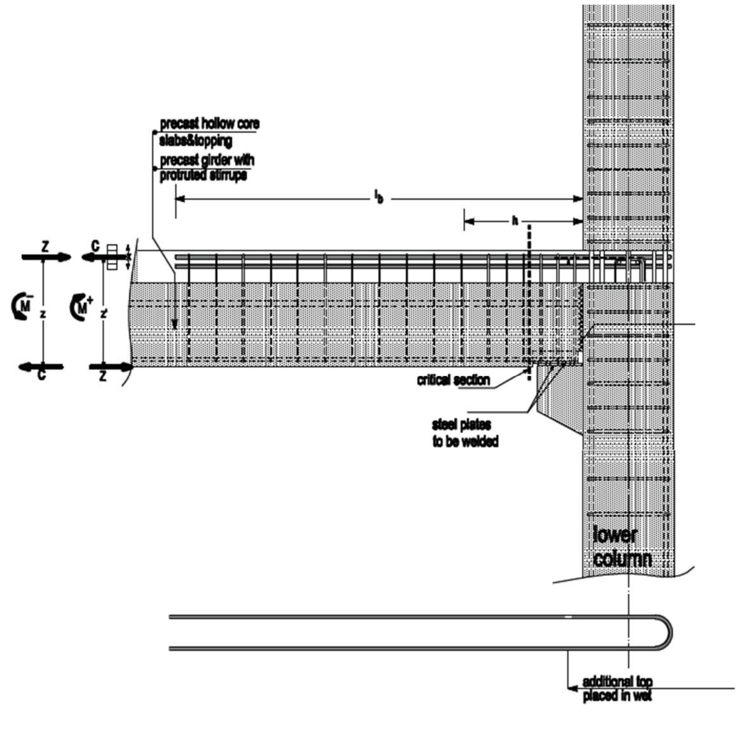


Figure 5.1. Industrial type beam column connection

Flexure: This inequality should be satisfied; $M_{Rd} \geq M_{Ed}$ where M_{Ed} is the design moment coming from structural analysis. M_{Rd} will be utilized in the capacity design provisions for the frame resisting system under seismic action.

Bar anchorage: The following inequality should be satisfied for each bar; $l_b\ u\ f_{bd} \geq \gamma_R A_s f_{ym}$ where l_b, A_s, u, f_{ctd} are the anchorage length of the bar in the upper slab, its sectional area, its perimeter and the design tensile strength of the cast-in-situ concrete respectively. The ultimate bond strength f_{bd} and the mean yielding stress of the steel f_{ym} can be calculated by $f_{bd}=2.25 f_{ctd}$ and $f_{ym}=1.08 f_{yk}$.

Longitudinal shear: This inequality should be satisfied $A_{ss} f_{yd} \geq \gamma_R A_{st} f_{ym}$ where A_{ss} is the total sectional area of protruded stirrups available in the end segment which has the length *h*, the height of the beam.

Bottom connection: The previous verifications shall be applied referring to an acting $R=\gamma_R A_{st} f_{ym}$.

Ultimate resisting *positive moment* can be calculated by $M_{Rd} = R_R\, z$ where z is $z = h - x / 2 \leq 0.96h$ where x is $x = R_R / (f_{cd}\, b)$ where R_R and b are the minimum resistance of the bottom connection calculated from all the failure modes, collaborating width of the upper slab, respectively.
Flexure: The following inequality should be satisfied $M_{Rd} \geq \gamma_R\, M_{Ed}$ where M_{Ed} is the design value coming from structural analysis.
Longitudinal shear: The following inequality should be satisfied $A_{ss}\, f_{yd} \geq \gamma_R\, R_R$ where A_{ss} is the total sectional area of the protruding stirrup available in the end segment of the beam equal to the height of the beam.

γ_R can be taken as 1.2 in above given equations.

6. CONCLUSION

The following conclusions can be extracted from the research summarized above: *i.* The lateral load capacities achieved in two directions are different for all specimens as it is expected. *ii.* The concrete quality used in the wet connection zone and the mortar used to fill the gaps are extremely effective on the general performance of the beam-column connections. *iii.* Precast slab elements are in a monolithic behavior till almost the end of the tests. *iv.* Improved specimens displayed better behavior than the initial specimens. The improvement in the response is seen clearly in the case of slab in compression. *v.* The envelope curve of the cyclic behavior might be different from the monotonic loading curve especially when it is dealt with the descending branch. *vi.* Asymmetrical load – displacement hysteresis obtained by experiments can be better represented by *OPENSEES Hysteretic Model or Pivot models* provided by commercial computer programs. *vii.* The necessary parameters have already been extracted through experimental works carried out within the scope of this project. *viii.* Beam to column connections can be modeled by simple beam elements considering the material nonlinearity using any code including the one developed in ITU, DOC2D, in addition to the codes commercially available for nonlinear analyses. *ix.* Whenever it is encountered by asymmetrical cyclic behavior in sections or connections of asymmetrical framed buildings two directional push analyses may be needed to find out the most unfavorable one.

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REFERENCES

DOC2D, Yüksel, E. (1998). Bazı Düzensizlikler İçeren Üç Boyutlu Büyük Yapı Sistemlerinin Doğrusal Olmayan Çözümlemesi. Ph.D. Thesis submitted to Graduate School of ITU.
Eurocode 2: Design of Concrete Structures, 1992.
Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., Medina, R. (2001). Development of a Testing Protocol for Woodframe Structures, Department of Civil and Environmental Engineering, Stanford University.
OpenSees (2012). The Open System for Earthquake Engineering Simulation. Pacific Earthquake Engineering Research Center.
SAP2000 v15.1.0. Static and Dynamic Finite Element Analysis of Structures. Berkeley: Computer and Structures, Inc.
Smyrou, E. (2008). Seismic design of T-shaped RC walls, PhD Thesis, University of Pavia, Rose School.
Yüce, S.Z. (2009). Basitleştirilmiş Betonarme Sistemlerin Deprem Hesabı için Yerdeğiştirmeleri Esas Alan Bir Yöntem, Ph.D. Thesis submitted to Graduate School of ITU.