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NUMERICAL AND EXPERIMENTAL SEISMIC PERFORMANCE EVALUATION OF THE NEW CONCEPT OF PRECAST CONCRETE CONECTIONS

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ABSTRACT

This paper aimed to introduce a new concept of precast concrete connection for the purpose of mass construction for low-rise buildings. Performance of connection was assessed based on FEMA 356 and the level of ductility has been calculated. Also to find the practical guideline for using of this connection, numerical evaluation has been carried out and compared with full scale tests results. Results shows that connection can be used for ordinary level of ductility based on ASCE7-2010 and to predict the realistic elastic deformation the moment of inertia modifier should be applied. Also special safety factor to achieve the expected performance shall be considered for some parts of the connection. Conclusion shows a good correlation with previous researches and confidence for the use of current design procedures with proposed modifications in this paper.

INTRODUCTION

Growing of demand for the residential housing in Middle East is experiencing the new challenge for the use of new competitive models of precast concrete structures with a proper design to the existing condition. Some part of region categorized as the medium to high seismic zone. This grow in use is attributed again to increasing studies by both designers and researchers to find alternatives for existing models of precast concrete system. Similar to the California, the major problem for that region is to have a clear idea about the seismic performance of the connections.

A new model of beam-column connection introduced a mixed up dry and wet connection systems (Figure 1).

Figure 1, Structural assembling process

Using a steel connector segment as a part of precast beam helps the system for simplicity in dry connection without using post tensioning such as available experiences (Giok 1993, Saqan, 1995, Kulkarni et al, 2008, Maya, et al, 2012) (Figure 2).

Figure 2, Structural assembling process

Connection to foundation is also designed by using base plate and hollow core system has used for flooring system. A layer of cast in situ concrete filled on slabs and on top of precast beams as well and it will create the structural integrity. It would be considered as the wet part of the beam column connection (Figure 3 & 4). This system has been designed to provide for rapid field assembly without using of concrete corbels.

Figure 3, Base connection and flooring system

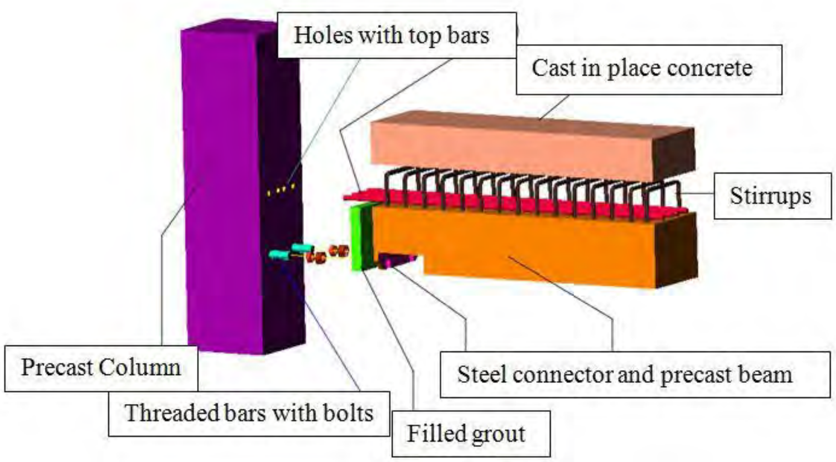


Figure 4, Assembly scheme

A pair of full scale specimens subjects to a cyclic inelastic loading were tested for this reaserch. Tests were initiated by Building and Housing Research Center (BHRC) in Iran. The main aim for these experiments was to evaluate the perfomance and develop a guideline for analysis and design for the users and designers. Performance of connection has been evaluated and compared by FEMA 356.

TEST SET UP

A rigid test frame with 2.5m height were set up for testing of specimens. Maximum allowable loading for this test frame was given equal to 600 KN while the maximum applied load on specimens was 250 KN. A data logger with 24 chanel has been connected to LVDT sensors for gathering the displacements in diffenenent sensors (Figure 5&6). Three Concrete cubic samples with 150*150*150 mm were used to find the strength on the day of tests.

Two specimens (PCF-1 and PCF-2) with 2.5m height of column and 2m length precast beams were tested. The column base was pinned and all other connections were roller supported. Sections of columns and beams were designed based on standard procedures for concrete elements using ACI 318 practice code (Figure 7).

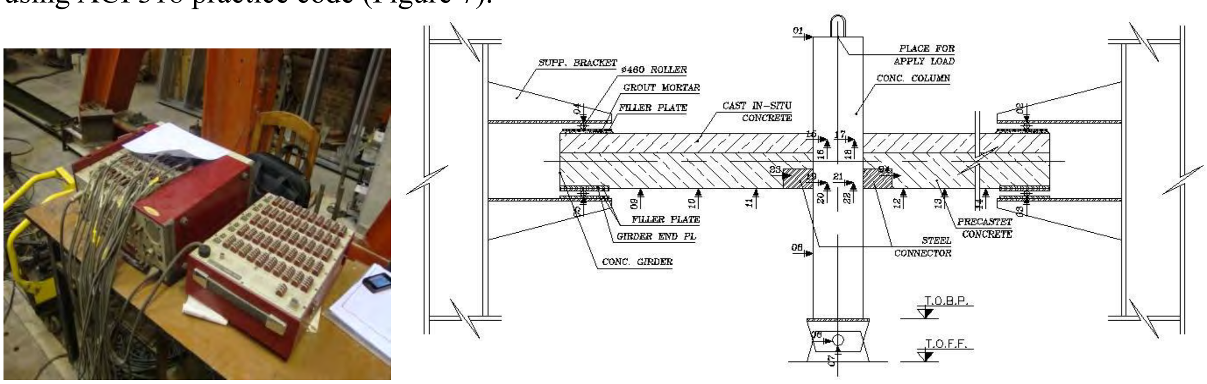


Figure 5, Data logger and LVDT sensor placement scheme

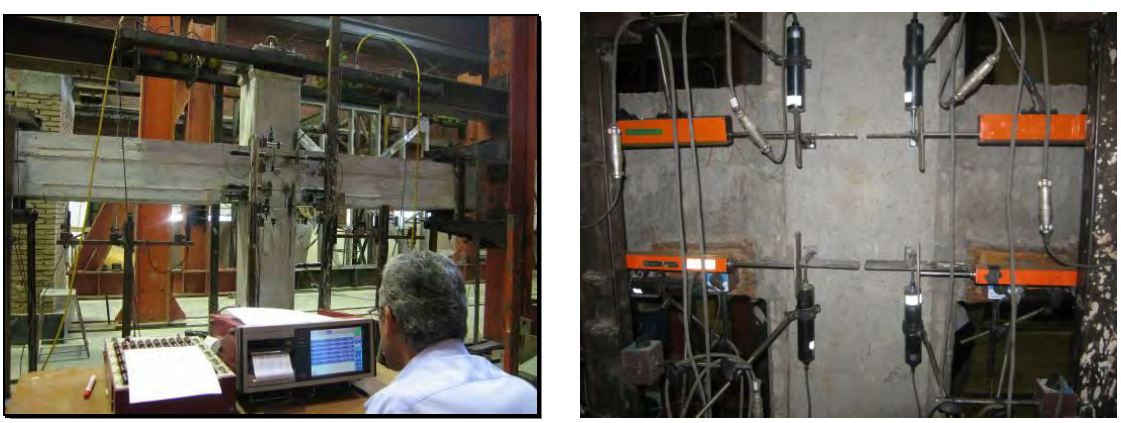
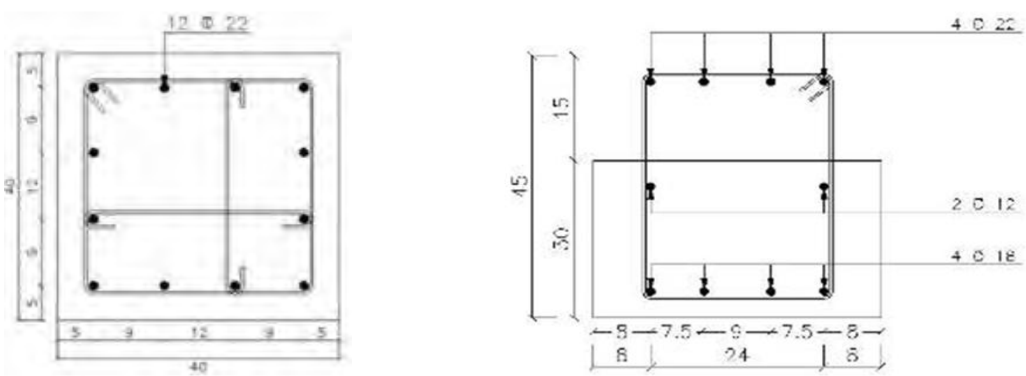


Figure 6, Sensor placement and test setup

Designed sections for both specimens are obtained from a real design for a five stories building to represent a non scaled model for the behavior and mechanical properties. The steel connector element as the innovative part for this beam was designed based on AISC-ASD 89 code and HEB240

German section was used for this segment. This element was connected to column by 2 No. 32mm bars and bolted properly. The only difference between 2 specimens is to use 2 nuts for this connection in PCF-2 instead on using 1 nut in PCF-1. The column laterally loaded on top head. Loading started from 6 KN. Tests setup, loading arrangement and LVDTs arrangement for measurements are used based on the FEMA 461 loading protocol. Lateral deflection and deformations in connection measured at each step of static cyclic loading by increasing rate equal to 1.4 (FEMA 461, 2007). Tests were continued to find first yield displacement and occurrence of plastic hinging which represents failure mechanism. Related displacement to the failure has been described in FEMA 356 for the precast connections as a qualitative description. The 80% drop in forces from the second yielding point or failure displacement equal to 2 times than yielding displacement are also other criteria (Geok et al, 1993). Ductility was calculated given equal to the ratio of maximum displacement to the yield displacement. Therefore, the performance evaluation can clearly give a guideline and modifiers for using the code criteria for the precast system.



Column Beam

Figure 7, Sensor placement and test setup

MATERIAL SPECIFICATION

Characteristic strength of concrete (f'c) is equal to 30MPa and steel grades are S400 with Fy=400 MPa. Master flow 515 grout mortar was used for filling the joint between column and beams. The steel connector element was made of German steel type (ST37) steel with Fy= 240 MPa.

TESTS' RESULTS

The hysteresis and load deflection curves for column head of both specimens (PCF1 & PCF2) show clearly the elastic, Elasto-plastic and plastic behavior which represent the normal behavior of a concrete structure (Figure 8&9). The hysteresis loops shows pinching that were because of inelastic behavior of the steel connector element and its deformation at the stage of 19 however in PCF-1 some failures occurred in nuts at connection which shows more pinching (e.g. PCF-1 had 1 nut and PCF-2 had a pair of nuts to connect steel connector to bars). The history of load-deflection for PCF-2 shows linear behavior up to point 15. There is no significant change in behavior and the structure shows an elastic behavior. First yield occurred at the loading cycle 16 and cycle 17 shows the 2nd yielding clearly. For assuring of these observations other hysteresis loops gathered by sensors in different points, also have reviewed precisely and results show confidence on achievements.

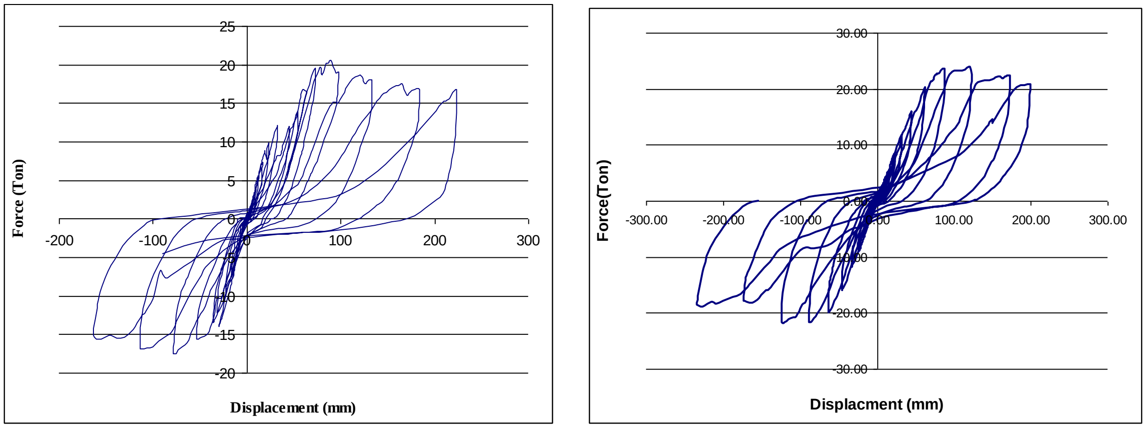


Figure 8, Hysteresis curves, Left PCF-1, Right PCF-2

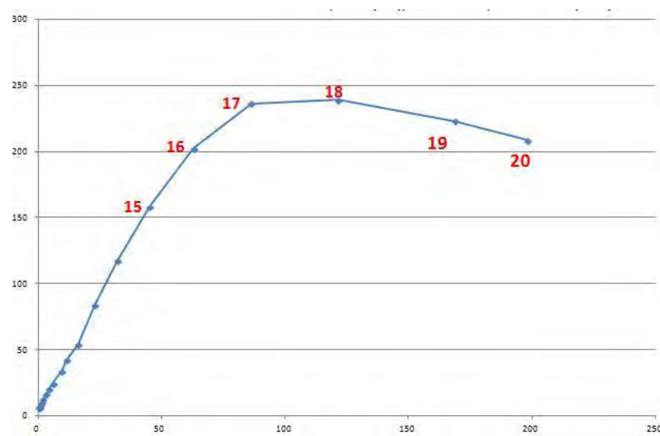


Figure 9, Hysteresis and Load Deflection of the column head

VERIFICATION OF FINITE ELEMENT MODEL

Finite element model (FEM) evaluated and verified with tests' results. This will support to understand the behavior and to have a proper model for study on steel connector segment. Solid and Truss type finite elements functions were used for the FEM using ABAQUS software to represent concrete and steel bars respectively. The same loading history applied for both model and test samples (Figure 10). Attempted has been made to evaluate the model for a close consistency between theoretical and experimental hysteresis loops (Figure 11).

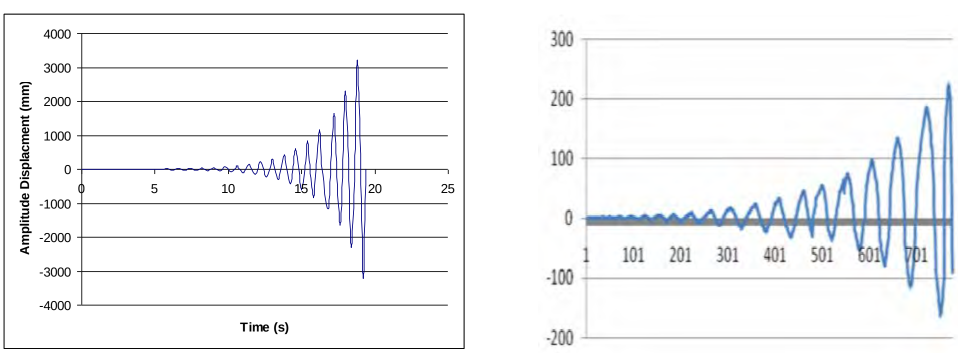


Figure 10, Loading history, Left; Numerical model, Right; Experimental Model

Both studies show pinching in hysteresis loops and the behavior of steel connector for both models are similar and this conveys that the real behavior of precast beam and column are predictable and consistent to theories however, the connection behavior shows deviation from the cast in place concrete (Figures 12,13).

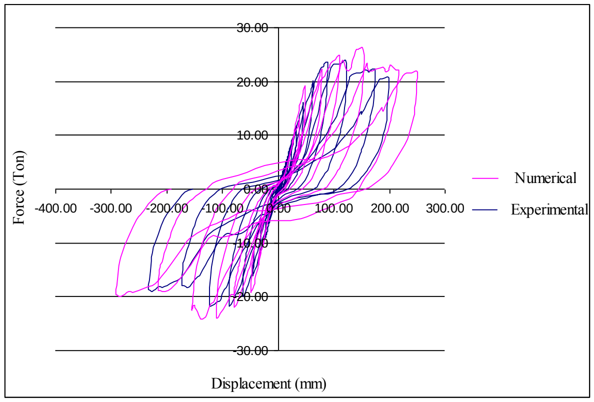


Figure 11, Comparison on Theoretical and experimental hysteresis loops

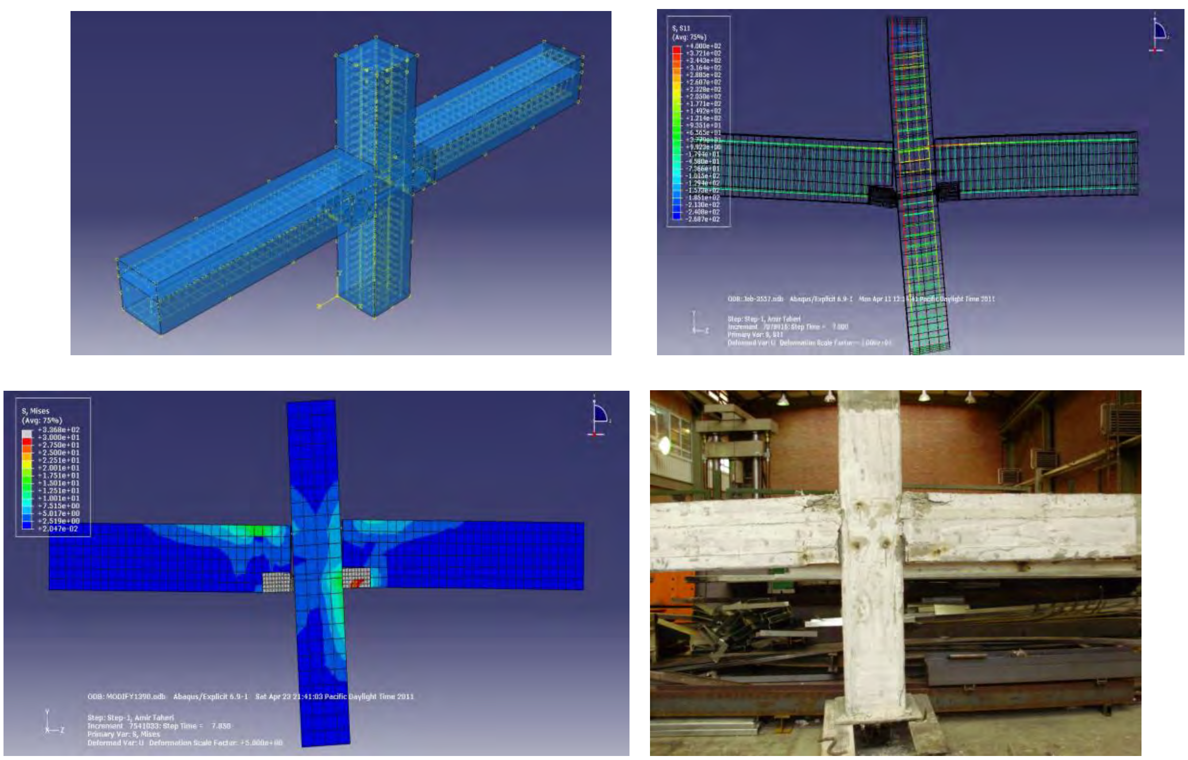


Figure 12, Comparison on Theoretical and experimental models at the final stage of loading

CALIBRATION OF THE FRAME TYPE ELEMENT FOR THE USE IN FEM

As the main purpose of this paper is to find an applied solution for using this connection, the finite element model has shown that the elements are behaving on the basis of mechanics of material without deviation in behavior; however ductility of connection and the effect of steel connector element will have overall effect of the behavior of structure. For modeling of structures normally the frame type element with static and pushover analysis are used. Therefore for making a guideline for the use of this connection and have realistic results from normal FEM modeling with frame elements some modifications might be needed to represent the effect of pinching and loss of strength.

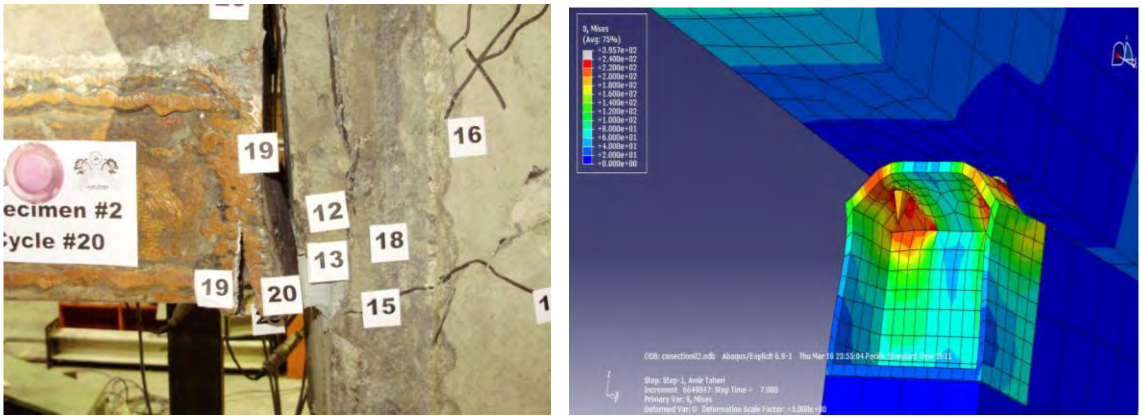
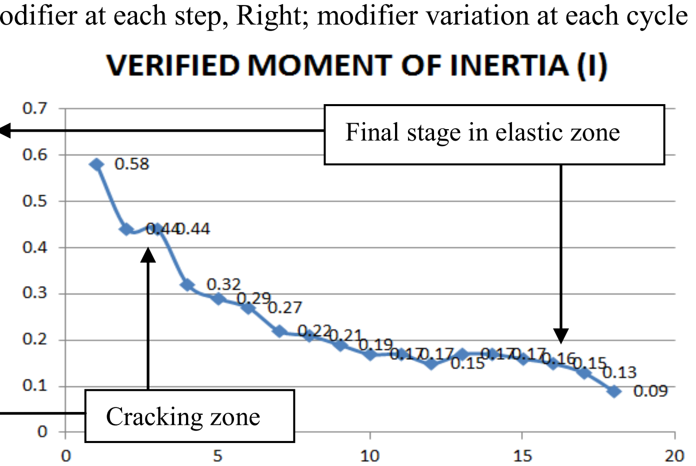


Figure 13, Comparison on Theoretical and experimental behavior of steel connector at the final stage of loading

SAP 2000 has been used for this study. A series of static linear analysis on the frame type model (Figure 14) were carried out to find the first yield point and compare the theoretical and experimental elastic deformations. The effect of precast connection and flexibility of steel connector shows more drift for the structure. This comparison shows that a multiplier on moment of inertia is circa 16%. This value is half of the proposed value for the cracked reinforced concrete beams (e.g. 35% I). Cycle 17 and 18 shows a significant change in modifier which is prorating to the load deflection curve so, cycle 17 is considered as yield point. The trend of deflections clearly shows the point of concrete cracking and the cycle number for the first yield as well.(e.g. cycle number 16 = Δy) (Table 1).

Table 1, Results of calibration of model and tests' displacements
Left; required moment of inertia modifier at each step, Right; modifier variation at each cycle

Load Cycle	Load(KN)	δ Theory(mm)	δ Test (mm)	Modifier
18	238.81	31.999866	121.58	0.09
17	236.138	31.641825	86.18	0.13
16	202.07	27.07681	63.05	0.15
15	158.316	21.213897	45.2	0.16
14	117.568	15.75378	32.28	0.17
13	83.834	11.233519	23	0.17
12	54.442	7.295074	16.43	0.15
11	42.418	5.683892	11.83	0.17
10	34.068	4.565016	9.45	0.17
9	24.382	3.267119	6.03	0.19
8	20.708	2.774814	4.58	0.21
7	16.366	2.192998	3.43	0.22
6	12.358	1.655937	2.13	0.27
5	10.354	1.387407	1.65	0.29
4	8.35	1.118876	1.23	0.32
3	7.014	0.939856	0.75	0.44
2	6.346	0.850346	0.68	0.44
1	6.68	0.895101	0.55	0.58



SEISMIC PERFORMANCE OF CONNECTION

Seismic performance of the connection was analyzed by Pushover Static Analysis Method (PSAM) from the cycle 17 as yield point (Figure 9). The levels of immediate occupancy (IO), Life safety (LS) and collapse prevention (CP) have been considered based on FEMA 356, table C1-3. There are no quantified values exists for precast connections in the mentioned code, so those qualitative description for precast connections has been used to find those levels based on test observations. Observations show that cycle No. 20 was the last step and would be considered as CP, Cycle 19 was LS and cycle 17 was behaved as IO condition (table 2).

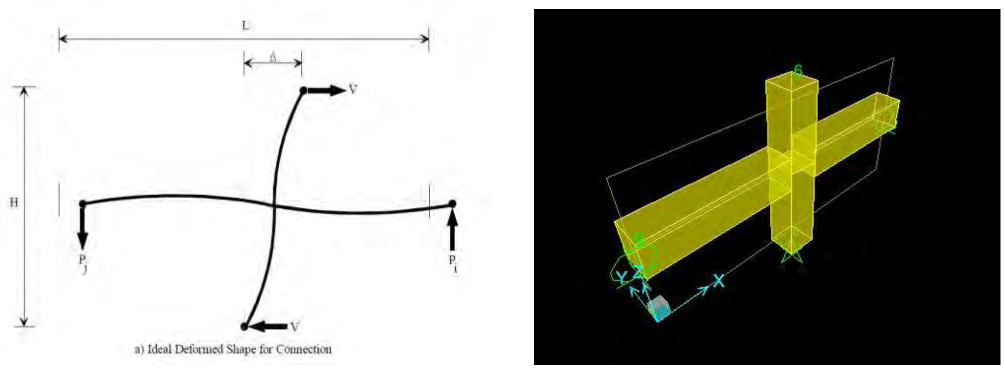


Figure 14, Second numerical model for study on elastic and plastic behaviors

Table 2, Relation between connection condition at each cycle and FEMA 356, table C1-3

		Structural Performance Levels		
Elements	Type	Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Precast Concrete Connections	Primary	Some connection failures but no elements dislodged.	Local crushing and spalling at connections, but no gross failure of connections.	Minor working at connections; cracks <1/16" width at connections.



CP-Cycle 20 LS-Cycle 19 IO-Cycle 17

A series of PSAM analysis were carried out based on the standard load deflection curve in FEMA 356 (Figure 15). In order to understand the relation between real performance and FEMA descriptions by using an iterative approach, the relation between each performance level, the applied load and deflection were studied. Deflections and loads has been extracted based on computer output on for each performance level prorata (Sample in figure 16).

Results have been summarized in figure 17 and this comparison shows that using of FEMA code for IO and LS levels is conservative and clearly the numerical model shows more deflection than the test. CP condition in test has more deformation that was coming from the plastic deformations of steel connector and it was also the cause of pinching of hysteresis loops. Comparison between displacements in CP condition shows around -10% lack of confidence. Difference between performances in IO show +24.5% more strength. Therefore, to maintain this amount of strength at

CP condition, the steel connector shall be designed for circa 35% more safety factor on usual AISC-ASD89 design criteria.

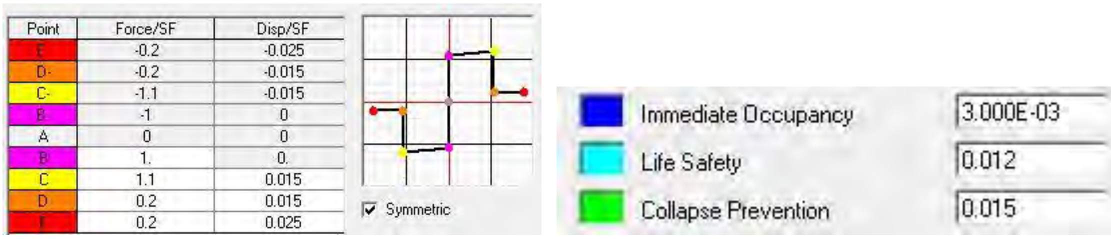


Figure 15, FEMA 356 standard load-deformation curve and performance levels

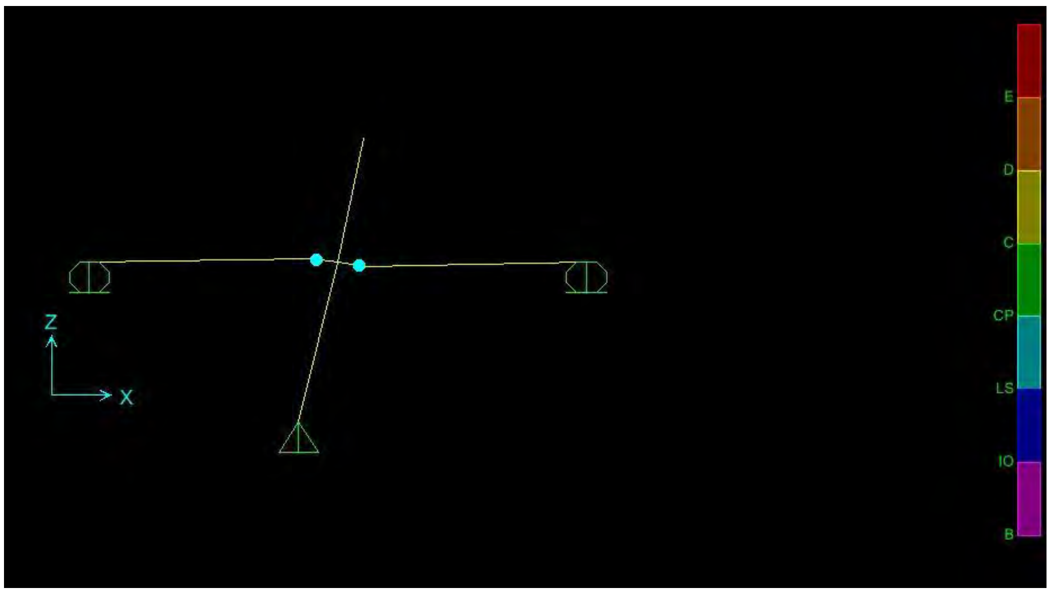
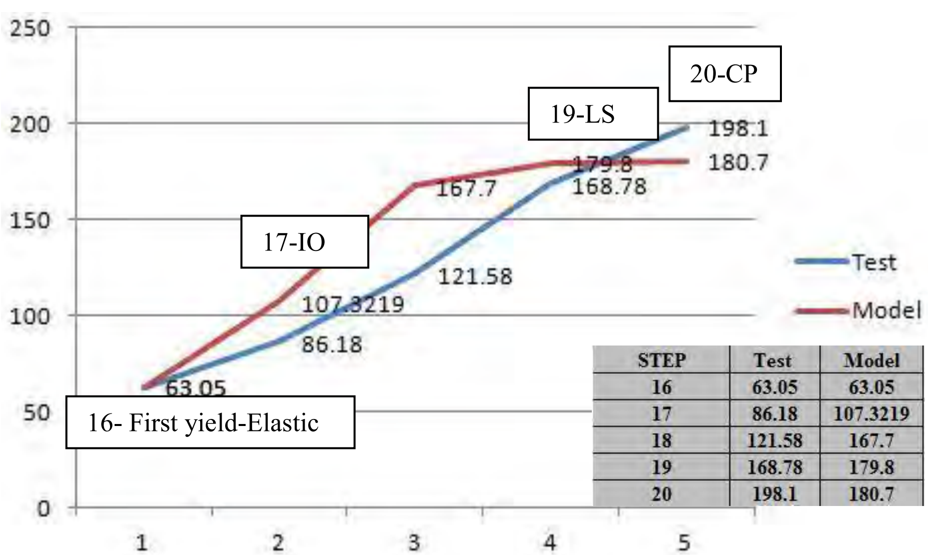


Figure 16, A sample of push over nonlinear analysis for finding the performance level



STEP	Test	Model
16	63.05	63.05
17	86.18	107.3219
18	121.58	167.7
19	168.78	179.8
20	198.1	180.7

Figure 17, inelastic deformations and performance levels of precast connection

Calculation of response modification factor (Rμ) carried out by finding the displacement ductility (μ=Δmax (cycle 20)/ Δy (cycle 17)) by using different available methodologies. Strength factor (R_S) was calculated based on the ratio of Δy (cycle 17)/ Δe (cycle 16) and redundancy factor (R_R) for this case considered as 1. Rμ can be taken equal to μ. R_R . R_S (Naeim F., 2004). Results for different approaches show more ductility for the tests' results in compare with numerical model (Table 3).

Table 3, comparison of response modification factor for test and numerical model based on FEMA 356 assumptions

THEORY	R TEST	R MODEL
YOUNG	4.713	4.299
NEW MARK & HALL(T<0.5s)	3.889	3.929
NEW MARK & HALL(T<1s)	4.713	4.299
KRAWINKLER & NASSAR (T=0.5)	4.511	4.218
KRAWINKLER & NASSAR (T=1)	4.816	4.341
MIRANDA & BERTERO (Rock, T=0.5s)	4.308	4.06
MIRANDA & BERTERO (Alluvium T=0.5)	4.599	4.238
MIRANDA & BERTERO (Rock, T=1s)	5.226	4.66
MIRANDA & BERTERO (Alluvium T=0.5)	5.675	4.953

For an applicable proposal on response modification factor a simple probability analysis were carried out. Gauss (Normal) distribution factor with 2% & 5% probability of failure was used to find the Rμ from different approaches. Therefore, having Rμ(Average) = 4.717 and Rμ(Standard Deviation) = 0.512 yields to find Rμ(for 2% risk of failure) = 3.51 and Rμ(for 5% risk of failure) = 3.87.

Comparing the result with the ASCE7-2010 for the response modification factor, clearly shows that the structure can be classified as the frame with ordinary ductility and the suggested response modification factor for this class is given equal to R=3.5.

CONCLUSION

This paper introduced and has given the guideline for design of a new type of precast concrete connection which might be used for the purpose of mass construction for low-rise buildings. Performance of connection assessed based on FEMA 356 and the level of ductility has been calculated. Also to find the practical guideline for using of this connection, numerical evaluation has been carried out and compared with tests' results. Results show:

- Installation procedure of this connection is much easier in compare with other existing solutions such as post tensioned connections.
- Comparing of tests' results and finite Element analysis shows consistency between structural behavior with current theories and the use of existing codes design procedures will be possible. Also this analysis confirmed that it will be possible to use FRAME type element for FE analysis of structure.
- Both Finite element analysis and tests results show the weakness of steel connector element at the stage of plastic hinging and the need for some modification in its design procedure.
- Using of Elastic linear analysis method cosidering FRAME elements for calibration of displacements in the elastic zone shows that the actual moment of inertia should

be taken equal to 16% of gross one. This shows more reduction in compare with current suggestions by code (i.e. 35% for beams).

- Nonlinear static analysis shows that the performance of structure in IO and LS condition will be conservative by using the FEMA 356 criteria, however, 10% lack of strength has been observed for the CP condition.
- A safety factor of 1.35 for design of steel connector based on AISC-ASD89 has been suggested to maintain the performance for CP, LS and IO prorata. However this paper suggests to do further research for improving the performance of the steel connector.
- This research shows ordinary ductility of this structure for designing in the seismic areas; however, based on current codes, this structure for the purpose of mass construction of residential buildings can be used for Middle East countries in high seismic risk areas.
- The level of obtained ductility for this structure is given equal to 3.5 and has classified similar to the concrete frame system with ordinary ductility in ASCE7-2010.

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