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BEHAVIOUR OF STRUCTURE WITH COMMON CONNECTIONS OF PREFABRICATED INDUSTRIAL HALLS UNDER STRONG EARTHQUAKES

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Abstract:
In most recent earthquakes, heavy damages and total collapses of prefabricated industrial halls have been observed widely in the world. Therefore, specific new research work has to be conducted in order to define the most suitable systems for earthquake protection of large prefabricated industrial hall structures. In the first part of this paper presented are original results from the conducted seismic response analysis of the selected representative common prototype structure (commonly used in practice) of industrial hall building. With application of the formulated advanced 3D nonlinear analytical model, successfully are defined critical connections, their actual hysteretic response, as well as, nonlinear response characteristics of the integral structure under simulated real strong earthquakes. Under mentorship of the third author, originated was development of new upgraded seismically safe system (USS-system) of prefabricated industrial halls. This development is made possible thanks to realized extensive innovative experimental research project leaded by the second and the forth author, providing highly valuable original experimental results (Ristic J. et all.). The obtained initial test results are mentioned in the second part of the paper.

Keywords:
Industrial hall, prefabricated structure, nonlinear response, critical connection, seismic safety

1. INTRODUCTION

The main industrial facilities, representing large industrial halls, are in recent years rapidly constructed in the region of South East Europe (SEE) and wider, applying various precast RC systems. High seismic risk of precast industrial halls, including heavy damages and total collapses, was commonly observed in many past earthquakes widely in the world. It clearly points out the urgent need to seriously treat this problem in regard of providing essential structural safety, sustainable economic and social development and general seismic security in seismically active regions. The created specific seismic risk of this type of structures has not been well quantified to

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this date and sound seismic risk mitigation concepts are not available. So, this study is basically focused on development of advanced structural design strategies through conducted extensive experimental and theoretical research. Using results from the original experimental tests, new specific innovative end-products (deliverables) are created representing highly important novel upgraded seismically safe system (USS-system) of prefabricated industrial halls. In the first part of this paper presented are some selected original results from the conducted seismic response analysis of the analysed representative ***common prototype*** structure (commonly used in practice) of industrial hall building. With application of the formulated advanced 3D nonlinear analytical model, successfully are defined critical connections, their actual hysteretic response, as well as, nonlinear response characteristics of the integral structure under simulated real strong earthquakes. In the second part of the paper presented are some initial test results obtained from completed experimental test of the basic and important connection of precast RC column with precast RC footing.

2. PROTOTYPE OF PREFABRICATED STRUCTURE

The structure represents precast frame system formed by installed fourteen (14) three span frames in transverse direction (span Lt=20.00m), and four (4) frames in longitudinal direction having 13 spans (span Ll=12.00m). Having 4 columns in transverse direction and 14 columns in longitudinal direction, the hall structure is integrating in total 56 columns supported by 56 individual precast foundations with variable dimensions 400x400cm and 300x300cm, depending on actual vertical load on column.

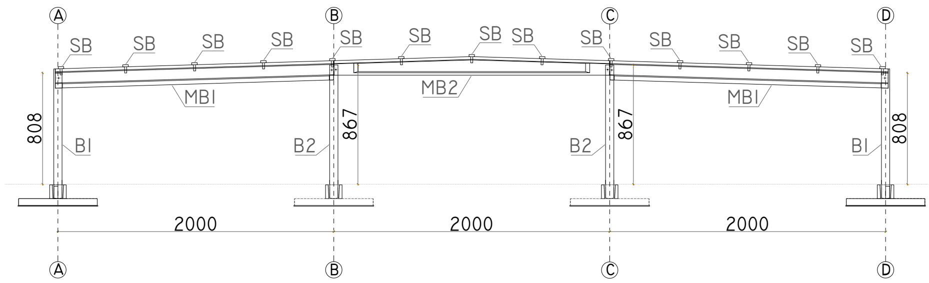


Figure 1. Transversal cross-section of industrial hall prototype structure

In x and y direction, the structure dimensions in plan are L_x=13x12.00=156.00m and L_y=3x20.00=60.00m. The columns are designed with the same cross-sections 60x60cm (cast with concrete C40) and two different reinforcements.

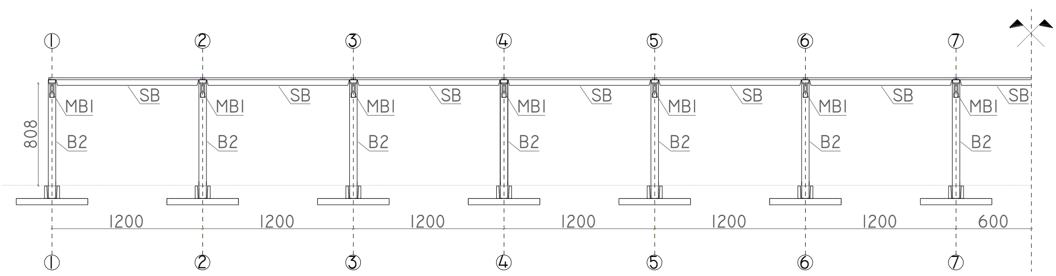


Figure 2. Longitudinal cross-section of industrial hall prototype structure

Column B1 and column B2 are reinforced respectively with longitudinal bars 12ϕ20mm (ties ϕ8/15cm+ϕ10/15cm) and 12ϕ25mm (ties ϕ8/15cm+ϕ10/15cm). The height of central and side columns are H_c=8.67m and H_s=8.08m, respectively, Fig. 1 and Fig. 2. The roof structure is formed with precast roof I-beams with h=140cm and span L=20.17m, Fig. 2, and roof beams with h=134.35m and L=20.00m. The other roof members are secondary elements. In the structure project shown are all details and geometry data of considered prefabricated system.

3. NONLINEAR MODEL AND GLOBAL STRUCTURAL SEISMIC RESPONSE

3.1. Formulated advanced nonlinear 3D analytical model

Formulation of advanced nonlinear 3D analytical model is based on structural geometry, material properties and characteristics of prefabricated members and their connections. The selected hall prototype structure consists of prefabricated concrete elements such are columns, main beams and secondary beams, which together form a 3D structure. The main frames are designed at every 12m forming in total 13 spans. The mainframe exists of three equal spans of 20m, respectively.

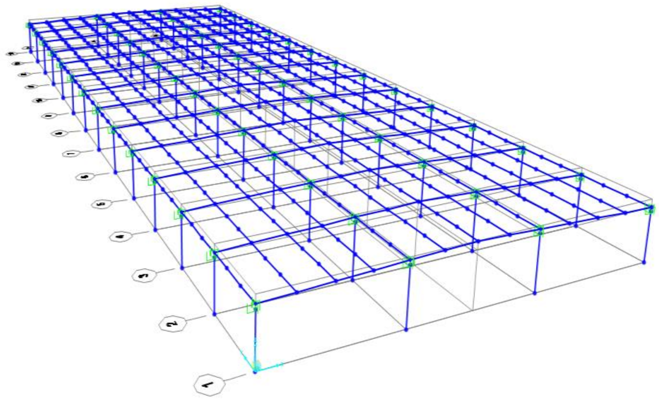
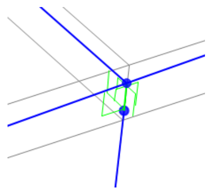


Figure 3. Formulated nonlinear 3D analytical model of industrial hall prototype structure

NONLINEAR MODEL OF CONNECTIONS



Link-L7 & Link-L-10

Figure 4. Nonlinear 3D link model of connection

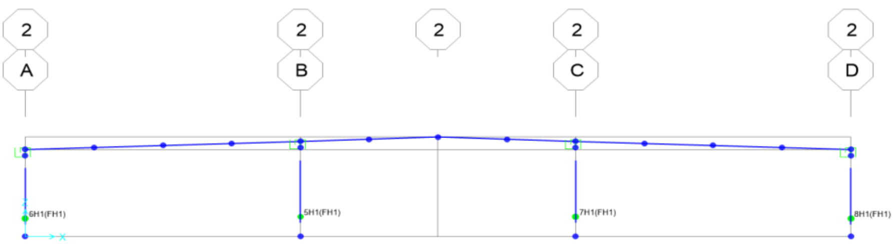


Figure 5. View of single frame of formulated nonlinear 3D analytical model

All concrete structural prefabricated elements (columns, main beams, and secondary beams) are made of C30/37 concrete. Columns (56 in total, 28 side and 28 inner) and main beams (56 in total), as well as, secondary beams (169 in total) are connected on the top of the columns with hinge connections. Actual connections of the main beams and columns are modeled with corresponding nonlinear link elements in x and y direction, using analytically defined parameters of used nonlinear models of link elements. Structure columns are considered in the model as fixed to the prefabricated foundation, considering fixing point at distance of dh=0.1m of the actual height of the columns. Advanced fiber modeling concept was used for nonlinear behavior modelling of all prefabricated columns. All adopted cross-sections of the columns are modeled considering in total 488 fibers, involving types of confined concrete fibers, unconfined concrete fibers and steel fibers, respectively. Using fiber models, advanced simulation of plastic hinges of columns was provided. Side columns are designed with height H1=6.5m, while the height of inner columns is H2=7.15m. Using realistic nonlinear behaviour characteristics of the implemented structural members and connections, in SAP2000 formulated was nonlinear analytical model of integral full-scale precast prototype industrial hall structure, Fig. 3, Fig. 4 and Fig. 5. The formulated model existed of 632 effective joints and 56 fixed boundary joints, and was used to study seismic response performances of the structure under the effect of strong earthquakes.

3.2. Dynamic characteristics of the structure

From the analysis of the dynamic characteristics of the initial state of the structure defined were the first twelve vibration modes and periods (Frequencies), Table 1. As dominant detected are vibration periods of the first three modes with periods: T_1=1.276431s, T_2=1.211326s and T_3=1.167516s, exposed dominantly in x-direction, y-direction and in torsion mode direction, respectively. The next higher modes are with closed period values and mainly express global-local characteristics of the structure.

Table 1. Computed dynamic characteristics of the structure

Mode	**Period** (sec)	**Frequency** (cyc/sec)	**Mode**	**Period** (sec)	**Frequency** (cyc/sec)
1	1.276431	0.783434	7	0.582971	1.715351
2	1.211326	0.825541	8	0.577706	1.730983
3	1.167516	0.856519	9	0.574829	1.739648
4	0.592302	1.688328	10	0.565856	1.767234
5	0.587990	1.700709	11	0.565785	1.767455
6	0.586765	1.704260	12	0.563391	1.774964

3.3. Displacement time-history response under earthquakes action in direction-x

Seismic response of the integral structure has been analysed for the case of simulated two selected representative strong earthquake effects acting separately in x and separately in y direction.

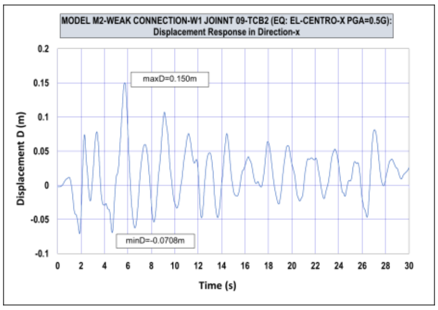


Figure 6. Displacement history of J-09; EQ: El-Centro-x (PGA=0.50G)

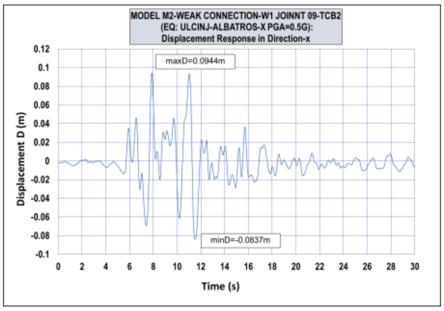


Figure 7. Displacement history of J-09; EQ: Ulcinj-Albatros-x (PGA=0.50G)

The considered two seismic ground motions in this study, actually represent El-Centro earthquake record and Ulcinj-Albatros earthquake record and both were scaled to the very high intensity represented by peak ground acceleration PGA=0.50g. The actual seismic response of the integral structure is characterized by computed response characteristics of various physical parameters. The most important results are processed and briefly presented in several sections. The computed displacement time-history response under action of two earthquakes in direction-x with PGA=0.50g is shown in two representative figures. In Fig. 6 shown is computed displacement response of node J-09 (top of column) for simulated El-Centro earthquake, while Fig. 7 shows computed displacement response of the same node J-09 (top of column) for the case of simulated Ulcinj-Albatros earthquake.

3.4. Displacement time-history response under earthquakes action in direction-y

The computed displacement time-history response under action of two earthquakes in direction-y with PGA=0.50g are of similar magnitudes. It is observed from presented the computed displacement response of node J-09 (top of column) for simulated El-Centro earthquake and the computed displacement response of the same node J-09 (top of column) for the case of simulated Ulcinj-Albatros earthquake.

3.5. Velocity time-history response under earthquakes action in direction-x

The computed velocity time-history response under action of two earthquakes in direction-x with PGA=0.50g is shown in two representative figures. In Fig. 8 shown is computed velocity response of node J-09 (top of column) for simulated El-Centro earthquake, while Fig. 9 shows computed velocity response of the same node J-09 (top of column) for the case of simulated Ulcinj-Albatros earthquake.

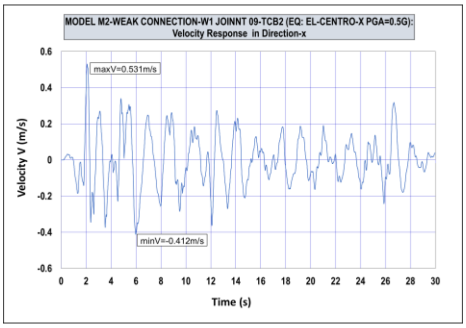


Figure 8. Velocity history of J-09; EQ: El-Centro-x (PGA=0.50G)

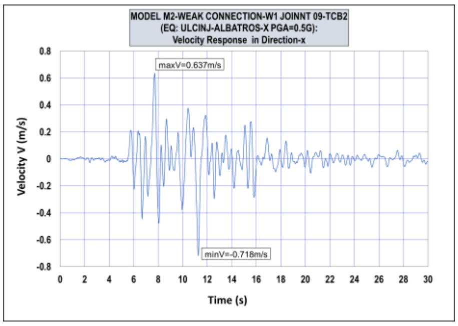


Figure 9. Velocity history of J-09; EQ: Ulcinj-Albatros-x (PGA=0.50G)

3.6. Acceleration time-history response under earthquakes action in direction-x

The computed acceleration time-history response under action of two earthquakes in direction-x with PGA=0.50g is shown in two representative figures.

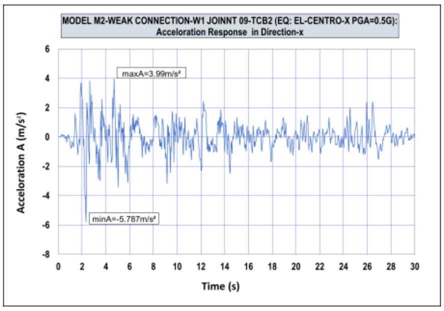


Figure 10. Acceleration history of J-09; EQ: El-Centro-x (PGA=0.50G)

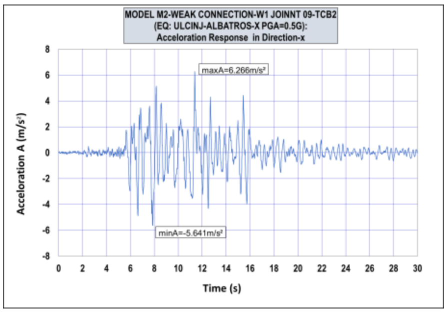


Figure 11. Acceleration history of J-09; EQ: Ulcinj-Albatros-x (PGA=0.50G)

In Fig. 10 shown is computed acceleration response of node J-09 (top of column) for simulated El-Centro earthquake, while Fig. 11 shows computed acceleration response of the same node J-09 (top of column) for the case of simulated Ulcinj-Albatros earthquake.

4. NONLINEAR HYSTERETIC RESPONSE OF CRITICAL CONNECTIONS

From the computed time-history seismic response of the integral structure for the case of simulated two representative strong earthquake effects acting separately in x and separately in y direction, scaled to the PGA=0.50g, the two main critical connections were identified: (1) Fixed section of columns showing hysteretic response and (2) Connections of main beams exposed to critical nonlinear response. Representative results are shown in next sections.

4.1. Hysteretic response of columns under earthquakes action in direction-x

The computed hysteretic response of column B1 and column B2 under action of two earthquakes in direction-x with PGA=0.50g is shown in four representative figures. In Fig. 12 and Fig. 13 shown are computed hysteretic responses of column B1 and column B2 for simulated El-Centro earthquake, while Fig. 14 and Fig. 15 shows computed hysteretic response of column B1 and column B2 for the case of simulated Ulcinj-Albatros earthquake.

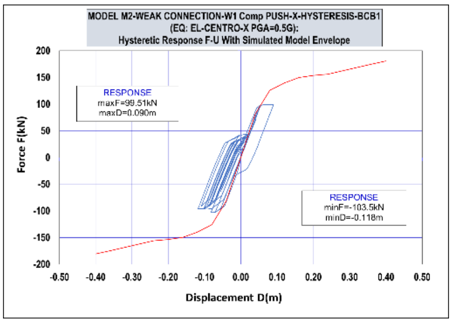


Figure 12. Hysteretic response of column B1; EQ: El-Centro-x (PGA=0.50G)

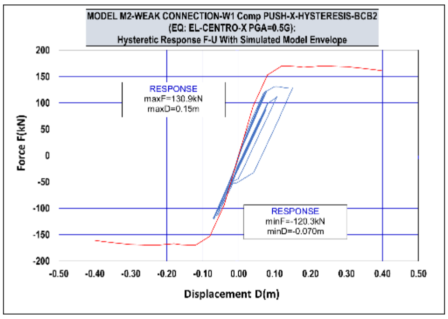


Figure 13. Hysteretic response of column B2; EQ: El-Centro-x (PGA=0.50G)

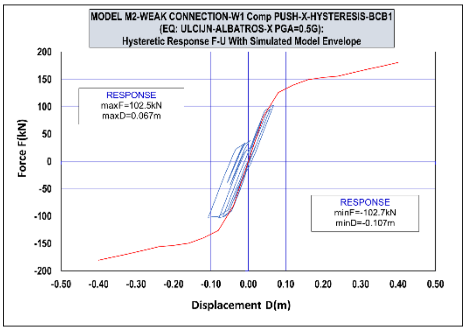


Figure 14. Hysteretic response of column B1; EQ: Ulcinj-Albatros-x (PGA=0.50G)

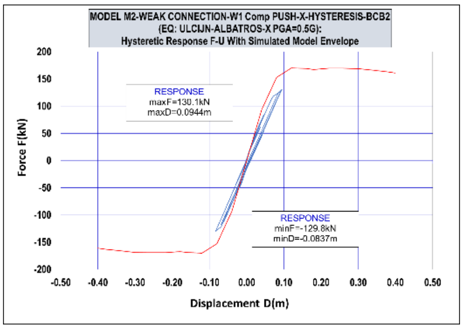


Figure 15. Hysteretic response of column B2; EQ: Ulcinj-Albatros-x (PGA=0.50G)

4.2. Hysteretic response of columns under earthquakes action in direction-y

The computed hysteretic response of column B1 and column B2 under action of two earthquakes in direction-y with PGA=0.50g was observed also from four representative figures. The computed hysteretic responses of column B1 and column B2 for simulated El-Centro earthquake and the computed hysteretic response of column B1 and column B2 for the case of simulated Ulcinj-Albatros earthquake are similar.

4.3. Hysteretic response of connections under earthquakes action in direction-x

The computed hysteretic response of connection L7 and connection L10 under action of El-Centro earthquake in direction-x with PGA=0.50g are shown in two representative figures. In Fig. 16 and Fig. 17 shown are computed hysteretic responses of connections L7 and L10 for simulated El-Centro earthquake. The computed hysteretic response of connection L7 and L10 for the case of simulated Ulcinj-Albatros earthquake are similar.

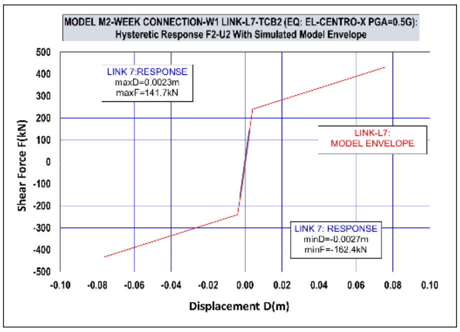


Figure 16. Hysteretic response of link-L7;
EQ: El-Centro-x (PGA=0.50G)

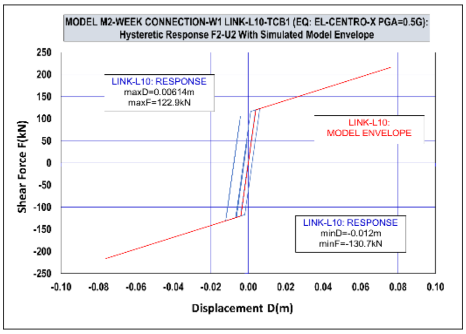


Figure 17. Hysteretic response of link-L10;
EQ: El-Centro-x (PGA=0.50G)

4.4. Hysteretic response of connections under earthquakes action in direction-y

The computed hysteretic response of connection L7 and connection L10 under action of two earthquakes in direction-y with PGA=0.50g were shown also in four representative figures. The computed hysteretic responses of connections L7 and L10 for simulated El-Centro earthquake and the computed hysteretic response of connection L7 and L10 for the case of simulated Ulcinj-Albatros earthquake are similar.

5. TESTING OF MODEL OF COLUMN-FOOTING CONNECTION: MODEL-M1

Providing safety and controlled behaviour of connection between precast RC column and precast RC footing is very important condition for assuring seismic stability of the integral precast structural system. To get full evidence in real nonlinear behaviour characteristics of this critical connection type, performed was detailed experimental test model design, then common production and then laboratory testing up-to failure of representative prototype model–M1 in the scale M=1:2 (Ristic J. et all.). Experimental test was realized on existing laboratory testing frame under simulated constant vertical load and horizontal cyclic displacement with increasing amplitude up-to deep nonlinearity. Model test set-up is shown in Figure 18. Experimental model–M1 is composed of precast RC footing and precast RC column with cross-section dimensions 30x30cm and its total length of L=165.0cm. Column length l_1=50cm was installed in footing box and the remaining column`s length of l_2=115cm was used for application of simultaneous vertical and horizontal cyclic load. Longitudinal reinforcement consisted of 12ϕ10mm steel bars and special confining ties of ϕ= 6mm spaced at distance of e=10cm. Footing RC base plate dimensions are d=25 cm and a/b=120x100cm, reinforced with steel bars ± 9ϕ12mm and ± 7ϕ12mm in both sides, was fixed to the frame base with 6 bolts with diameter of ϕ32mm. The RC footing box is with outer dimensions of 60x60cm and bottom inner dimensions of 35x35cm were used to fix RC column applying standard putinzenjering technology. The four side-walls of RC box were reinforced at both faces using 16+16=32ϕ8 steel bars as vertical reinforcement and the horizontal reinforcement existed of 6ϕ8 and 6ϕ8 steel bars at outer and inner wall faces, respectively. The recorded hysteretic curve from the performed experimental test, Figure 21, showed very stable nonlinear behaviour resulting from induced plastic hinge only in column's critical section. RC footing box was fully safe and damage was observed only in critical section zone of RC column, Figure 19 and Figure 20.

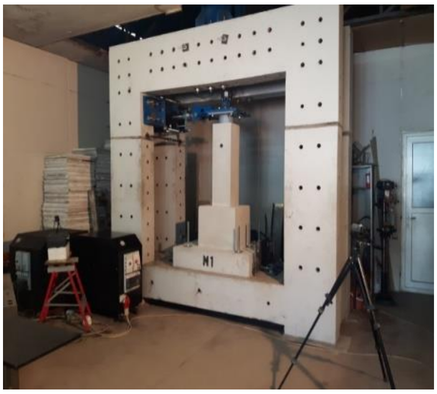


Figure 18. Set-up of 1/2 scaled model M1 in laboratory testing frame

Figure 19. Start damage at RC column

Figure 20. Heavy damage at RC column

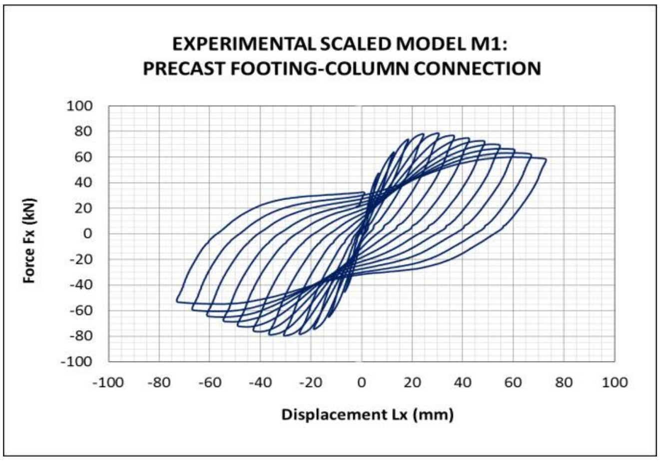


Figure 21. Force-displacement hysteresis recorded from the test of prototype model M1

Maximum horizontal restoring force of F_{max}=±80.0kN was recorded for displacement of d=±25mm. However, for induced maximum displacement of D_{max}=±74.0mm, the recorded horizontal force amounted to F=±55.0kN. So, obtained is small reduction of only 25.7% along with the recorded very stable hysteretic relation without any visible cracks in the foundation box. The test results have clearly shown *perfect and controlled* nonlinear behaviour of the assembled precast column-footing connection, confirming full validity of the developed production technology.

5. CONCLUSIONS

From the conducted extensive analytical research presented in this paper and considering the planed experimental testing of various connection types for development of novel seismically resistant prefabricated system the following conclusions are summarized:

(1) Considering the presented analytical research and observed real nonlinear response of existing (common) roof beam–column connection, it was fully confirmed that the common connection type is critical and week structural segment, since it is exposed to total failure under strong earthquakes;

(2) Experimental research is needed to improve connection and assure its stable and favorable behavior properties of the existing (common) roof beam–column connection. It will open wide possibility for their application in regions with the highest seismicity;
(3) Experimental tests and analytical research confirmed that standard footing-column connection possess required safety for seismic loads and can be used in practice. However, correct design is needed in order to assure application of columns with optimal cross sections and reinforcement;
(4) Prefabricated RC columns were designed correctly since they showed stable nonlinear hysteretic behavior under cyclic loads along with expressed ductility for the case of implemented ties spaced in prescribed small distances;
(5) Experimental research is needed to assess safety of the implemented RC corbels (short cantilevers) supporting L-beams in two story structures. Such tests are planned to be conducted;
(6) Experimental research is needed to define nonlinear behavior characteristics and safety of the presently used (standard) L beam-column connection types and roof beam–column connection types. Such tests are planned to be conducted;
(7) The proposed nonlinear analytical model parameters of connections represent highly valuable modeling data which can be successfully considered during the final seismic design process of prefabricated structures in seismic regions;
(8) From this study defined is optimal design target: (1) For the case of design earthquake, the behavior of structural connections should be basically linear and (2) For the case of maximum expected earthquake intensity, the behavior of structural connections may be in controlled nonlinear range. To efficiently satisfy both design stages, advanced structural analysis procedures during the design process is needed.

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NATO OTAN

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