International Journal of GEOMATE, May, 2018 Vol.14, Issue 45, pp.1-9
Geotec., Const. Mat. & Env., DOI: https://doi.org/10.21660/2018.45.7360
ISSN: 2186-2982 (Print), 2186-2990 (Online), Japan

CAPACITY DESIGN CRITERIA FOR SEISMIC RESISTANCE OF PRECAST CONCRETE COLUMNS USING STEEL BOX CONNECTION

*Chayanon Hansapinyo, Nutchanon Siri and Chinnapat Buachart

Center of Excellence for Natural Disaster Management, Department of Civil Engineering, Chiang Mai University, Thailand

*Corresponding Author, Received: 30 June 2017, Revised: 19 Nov. 2017, Accepted: 20 Dec. 2017

ABSTRACT: A variety of advantages of precast concrete adding to benefits of concrete has made the pre-fabrication and assembly systems widely adopted in building construction business. However, the connections may diminish the increase in the utilization due to their complication in the installation work and seismic design consideration. This paper presents an easy-to-install steel box for column base connection designed based on the standard capacity design criteria. Experimental investigations on the precast concrete columns using the steel box connection at the column base under cyclic loading were also made. The test results show the satisfactory seismic performance of the precast specimens both in seismic shear capacity, ductility and energy dissipation compared with the identical cast-in-place specimen. The failure of the precast columns, with proper details, was in the flexural ductile mode. The shear capacity was respectively 1.16-1.24 times higher than the averaged capacity of the cast-in-place column. The damping ratio was about 18-25 percent in the inelastic range, after the drift ratio of 3.0%. Finally, the seismic performance of the designed precast concrete column using the steel box connection based on the capacity design criteria is guaranteed without early brittle failures.

Keywords: Precast concrete column, Seismic design, Column steel box connection, Lateral cyclic load

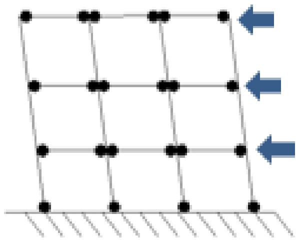
1. INTRODUCTION

The exposure of buildings to damage from seismic shaking is steadily increasing because of continuing urbanization. However, a significant reduction of the risk can be accomplished through improved building codes and construction qualities. For the design of buildings in a seismic prone area, failure mechanism of structural elements in a building must be considered to ensure ductile damage under a severe earthquake. Columns and beam-column joint are maintained in the elastic range. In other words, a good structure must be able to control the failure mode and location of damage. Under a big earthquake, flexural failure induced by yielding of the longitudinal reinforcement is expected to localize on beam elements for absorbing and dissipating energy. This phenomenon can be achieved by a systematic design for the Weak Beam-Strong Column failure mechanism using the Capacity Design criteria [1]. Following this concept, the Beam sideway mechanism (Fig.1(a)) is permitted for the failure mechanism of frame buildings under a severe earthquake load. Experience of past strong earthquakes has proved considerable fewer losses of the Beam sideways buildings compared with the Column sideways buildings (Fig.1(b)).

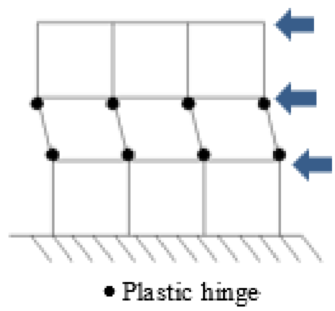
As the column and the beam-column joint elements are very important for the frame building, many past researchers investigated the shear capacity demand of the elements under the lateral loading [2-5]. The column capacity models based on key parameters were developed [6,7]. For the precast column with the base connection, Rejame et al [8] conducted an experimental investigation on precast columns in socket foundations with internal smooth interfaces. The tested specimens were failed by the yielding of the longitudinal reinforcement out of the embedded region.

Seismic design of ductile reinforced concrete frames has been developed over a long period of time. A successful design method has been based on the past lessons learned from the failed buildings. However, the design of precast concrete element has been done based mainly on information of the experimental result of a prototype element, as a wide variety of precast connection options. As a result, innovation integrated with fabricator capabilities is the key to create a successful solution. This paper presents lateral cyclic load test and seismic design of precast concrete columns using steel box connection. The connection was adapted from the column-to-foundation connection with bolted socket presented by Negro, and Toniolo [9]. The technique permits faster construction with the use of this bolted connection type, as shown in Fig. 2.

Seismic design of concrete frame buildings is first described to overview demand capacity for the design of the precast concrete columns. Following the strength-based design concept or the Capacity Design method, a ductile emulative design can be achieved maintaining the elastic performance of the connection part (Fig.2(c)) [10]. Then, precast concrete columns with the steel box connection, as shown in Fig.2(b) and identical cast-in-pace concrete column tests under lateral cyclic load is explained. From the test, seismic performance of the precast columns is discussed compared with the cast-in-place one. Eventually, the design of the precast columns and the steel box connection are described based on the test results.

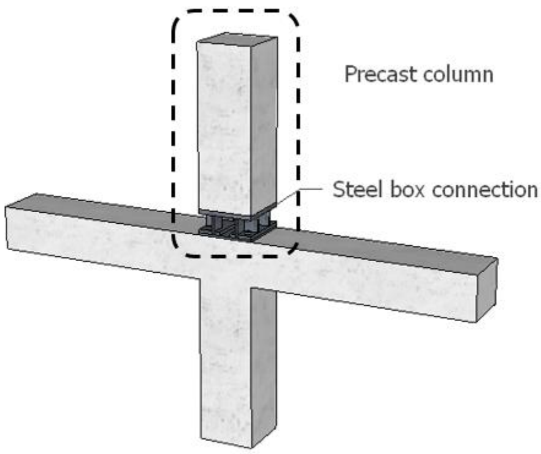


(a) Beam sideways

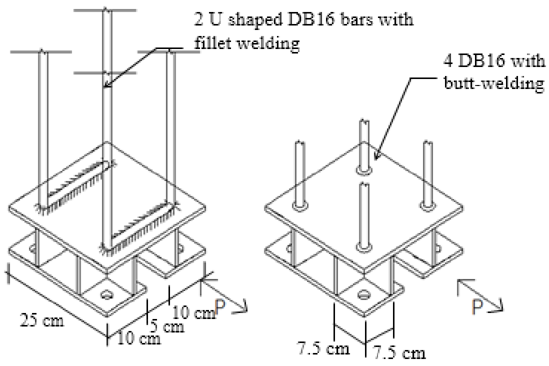


(b) Column sidesway

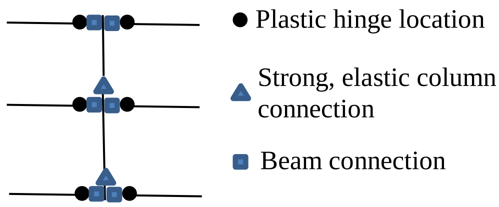
Fig.1 Failure mechanisms of frame buildings



(a) Precast concrete column in a precast frame (Precast framing beams are ignored for a clear definition of the scope of this study on the precast column connection)



(b) Steel box connection



(c) Emulative precast connection design

Fig.2 The precast concrete column with steel box connection

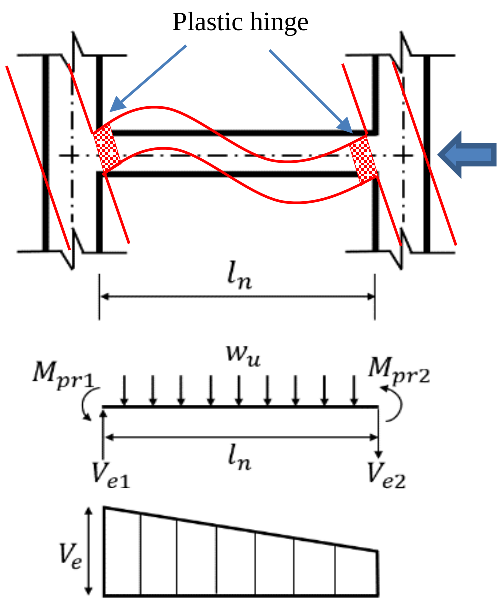


Fig.3 Probable flexural strength of the beam ends and demand shear capacity

2. SEISMIC DESIGN OF PRECAST CONCRETE FRAME

An overview of the seismic design of reinforced concrete frame buildings is described below. Following the steps, damage pattern can be controlled in flexural mode at the beam ends and the collapse of buildings under a strong earthquake can be avoided based on the Capacity Design criteria.

2.1 Estimation of Seismic Load

First, for a regular building, the lateral seismic load on the building is estimated according to a local standard. The load depends on seismic parameters including local seismic intensity, type of building, type of soil, building weight and dynamic properties. The calculation starts with base shear force estimation and then distributes the force to each floor of the building. Structural analysis of the building is next performed for the element forces induced by all cases of load combination.

2.2 Design of Frame Beam and Capacity Demand

From the envelope flexural force of the building analysis, flexural strengths required for beams are obtained and required longitudinal reinforcement can be determined accordingly. The provided beam reinforcement is used for the capacity demand to avoid undesirable damage under a strong earthquake by forming the flexural hinge at the beam ends. The demand shear force (V_e) at the stage is estimated as shown in Eq.(1) and Fig.3. The beam shear capacity designed following this procedure ensures the avoiding of shear failure in the beam element.

$$V_e = \frac{M_{pr1} + M_{pr2}}{l_n} + \frac{w_u l_n}{2} \tag{1}$$

where

M_{pr1} and M_{pr2} are the probable flexural strength of the beam ends

w_u is the factored load/length of the beam

l_n is the clear length of the beam

2.3 Design of Frame Column and Beam-Column Joint

To protect the columns against a strong earthquake, design shear and flexural strengths of the columns must be higher than the flexural strength of the joining beams. Kuntz and Browning [11] have shown that the failure mechanism as shown in Fig.1(a) can only be achieved if the column-to-beam ratio is relatively large (on the order of four). According to ACI318 [12], the flexural strengths of the columns shall satisfy

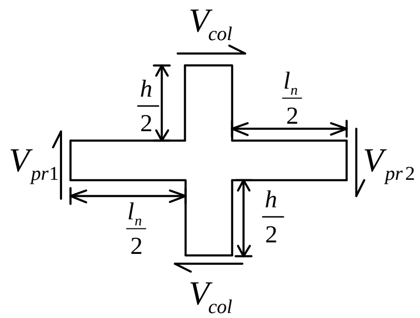
$$\sum M_{nc} \geq 1.2 \sum M_{nb} \tag{2}$$

where

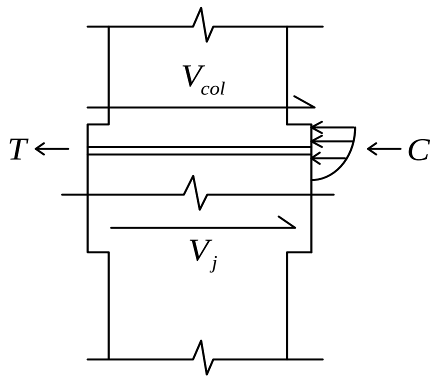
$\sum M_{nc}$ is the sum of nominal flexural strengths of columns framing into the joint

$\sum M_{nb}$ is the sum of nominal flexural strengths of beams framing into the joint

The flexural capacity of the column (M_{nc}) is then used to determine the column dimension and reinforcement. The required shear capacity of the reinforced concrete column is also estimated considering the free body diagram as shown in Fig.4(a). Then, based on the capacity of the designed columns, the required strength of the beam-column joint can be determined by considering Fig.4(b) and Eqs.(3). Consequently, the steel box connection is designed which will be explained next.



(a) Column shear



(b) Joint shear

Fig.4 Column shear and joint shear

$$V_j = T + C - V_{col} \;, \phi V_{nj} > V_j \tag{3.1}$$

$$T = 1.25 f_y A_{s,top} \tag{3.2}$$

$$C = 1.25 f_y A_{s,bottom} \tag{3.3}$$

where

V_{pr1} , V_{pr2} are the shear capacity of the framing beams calculated based on the maximum probable flexural strength of the beams

V_{col} is the column joint shear considering the force equilibrium in Fig.4(a)

V_j is the joint shear

ϕV_{nj} is the reduced joint shear capacity [12]

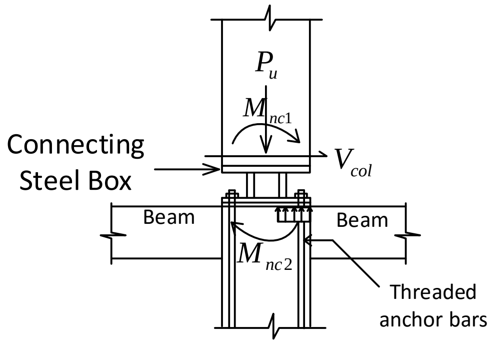
T and C are respectively the maximum tension and compression of the left and right joining beam

$A_{s,top}$ and $A_{s,bottom}$ are the top and bottom longitudinal reinforcement area, respectively

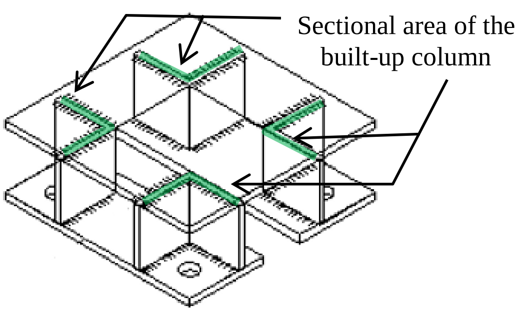
f_y is the yielding strength of the reinforcement

2.4 Design of Connecting Steel Box

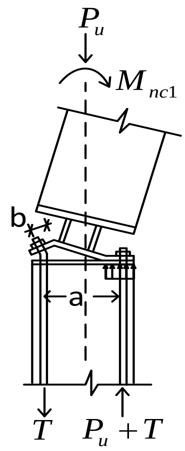
The factored axial load (P_u), shear (V_{col}) and bending moment (M_{nc1}) demand for the design of the connecting steel box are shown in Fig.5(a). The strength of a stub column (as the 4-angle built-up in Fig.5(b)) under combined axial and bending moment is examined through the interaction formula in Eq.(4)[3].



(a) Axial, shear and bending moment



(b) 4-Angle built-up steel column section



(c) Bending of base plate

Fig.5 Design of connecting steel box

$$\text{For } \frac{P_u}{\phi P_{n,con}} \geq 0.20: \frac{P_u}{\phi P_{n,con}} + \frac{8}{9}\left(\frac{M_{nc1}}{\phi M_{n,con}}\right) \leq 1 \quad (4.1)$$

$$\text{For } \frac{P_u}{\phi P_{n,con}} < 0.20: \frac{P_u}{2\phi P_{n,con}} + \left(\frac{M_{nc1}}{\phi M_{n,con}}\right) \leq 1 \quad (4.2)$$

where

$\phi P_{n,con}$ and $\phi M_{n,con}$ are the reduced nominal axial load capacity and bending moment capacity of the connecting box, respectively.

For other types of ultimate capacities, eg. shear and bending of the base plate, shear transfer across the joint plane between the plate and concrete, anchored bolted etc, the design equations are codified in AISC steel design [13] and using the force diagram shown in Fig.5(c).

3. TEST OF PRECAST CONCRETE COLUMNS

3.1 Specimens and Experimental Setup

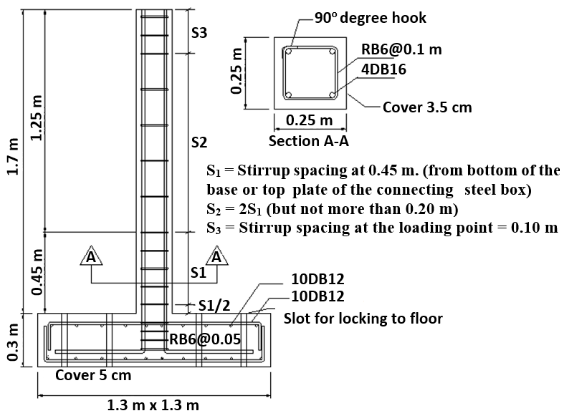
To verify the applicability of the designed steel box connection, tests of 5 precast columns were carried out at the Structural Laboratory, Department of Civil Engineering, Chiang Mai University, Thailand [14]. The test results were compared with an identical cast-in-place one. All the columns were 0.25x0.25 m2 section and 1.7 m in height from column base to top. 4-DB16 were used as the longitudinal reinforcement for all specimens. For the precast specimens, the steel box connection was used for the base connection. The difference between the precast specimens is based on the differences in construction method, e.g. grouting, welding to nuts, welding of the column reinforcement to the steel box. Fig.6 shows the specimen details. For the specimen nomenclature, M and P stand for the Monolithic cast-in-place specimen and the Precast specimen, respectively. Lateral cyclic loading was applied to the tip of the specimen, as seen in Fig.7. While the column bottom was arranged to be a fixed condition, the application of the load induces a cyclic shear and moment as if it is a half-story column in a rigid frame. The deformation control at the loading point, 1.5 m from the column base, represented as story drift was adopted as recommended in ACI T1.1-01 [15].

3.2 Section of the steel box connection

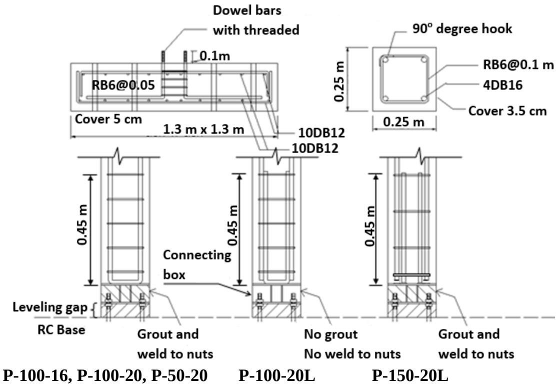
This is the additional design procedure compared with the cast-in-place column. Based on ACI318 [12], the calculated ultimate capacity of the reinforced concrete column under yielding of the longitudinal reinforcement is 25.33 kN, as shown in Table 1. The value of the ultimate capacity can be equivalent to the flexural capacity at the column base of 40 kN-m. (the point of the applying load is at 1.50 m. from the base). The capacity is used for determining the demand for shear reinforcement in the column and the connecting steel box at the column base. For the connecting steel box, based on AISC steel design [13], the estimated flexural yielding (M_y) and stiffness (*EI*) of the box are 57.9 kN-m and 5.88 kN-m2 respectively, providing 1.5 and 2.3 times relatively higher than the ultimate flexural capacity and non-cracked flexural stiffness of the reinforced concrete column section. With the capacity of the steel, box connection is higher than the column above, the connection failure can be avoided.

3.3 Number of dowel bar and threaded diameter

Column shear and combined axial-bending moment induced in the column above the connection are directly transferred to the connecting steel box and the anchored bars. For general cases,



(a) Monolithic specimen M-100



(b) Precast specimens

Fig. 6 Test specimens

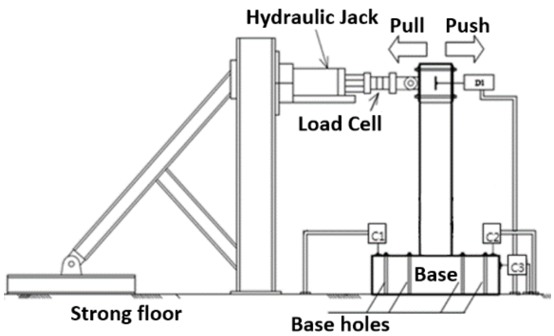


Fig. 7 Lateral cyclic load test

the critical loading stage is determined by the existence of tension. Under the tension load on the column, longitudinal reinforcement resists the entire load. Hence, at the steel box connection, number and threaded diameter of the dowel bars are calculated based on equilibrium, as shown in Eq.(5). It is noted that the buildability also decides size and number of the dowel bars. For this test, as the reinforcement of the column is 4-DB16, 4 dowel bars with the threaded diameter of 20 mm. and same yield strength of the column reinforcement was used. By using this approach, the design under tension becomes conservative. In the real situation, columns will be subjected to combined compressive load and bending. The entire column section will be curved and the stress of the section is under distributed compressive stress or compressive-tensile stress.

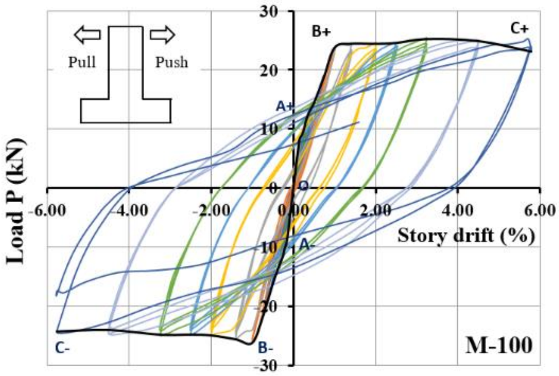
$$A_{sc} \times f_{yc} = A_{sj} \times f_{yj} \tag{5}$$

where A_{sc} , f_{yc} and A_{sj} , f_{yj} are the sectional area and yield strength of the column reinforcement adjacent to the connecting steel box and dowel bars, respectively.

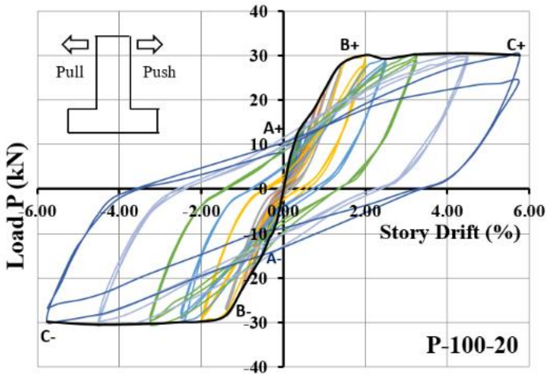
4. TEST RESULTS

4.1 Lateral Load-Deformation Relationship and Failure Mode

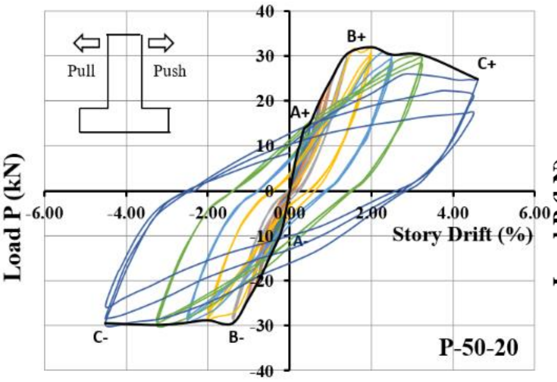
The cast-in-place specimen was failed in flexural failure with yielding of the longitudinal reinforcement at the column base. High ductility and energy dissipation were observed. For all the precast columns, depending on different reinforcement detailing, P-100-20 and P-50-20 specimens were ductile failure and P-100-20L, P-100-16 and P-150-20L were the brittle failures. The brittle failure was caused by the improper welding details of the column reinforcement on the steel box connection. Under the lateral cyclic loading, the ductile precast concrete columns with the steel box connection satisfied seismic performances, compared with the cast-in-place column. The ductile precast columns were failed under flexural mode similar to that of the cast-in-place specimen but at the section above the steel box connection. Hence, the ultimate lateral load of the ductile precast is higher than the load of the cast-in-place column due to the shorter moment arm length. Figs 8 shows the lateral load-deformation relationship of the cast-in-place concrete column and the precast columns. As seen in Table 1, the ultimate lateral load capacities of the ductile precast columns are about 1.16-1.24 times higher than the averaged capacity of the cast-in-place column.



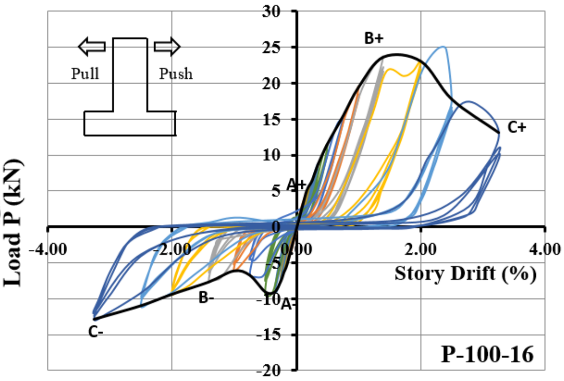
(a) M-100



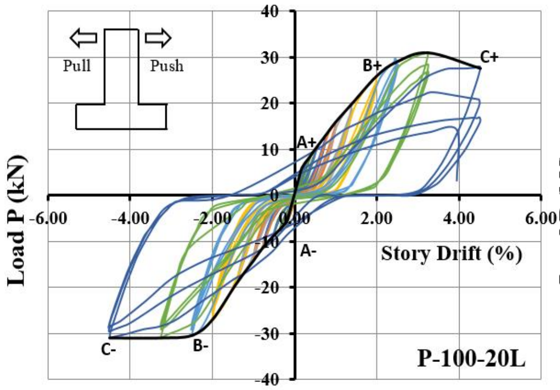
(b) P-100-20



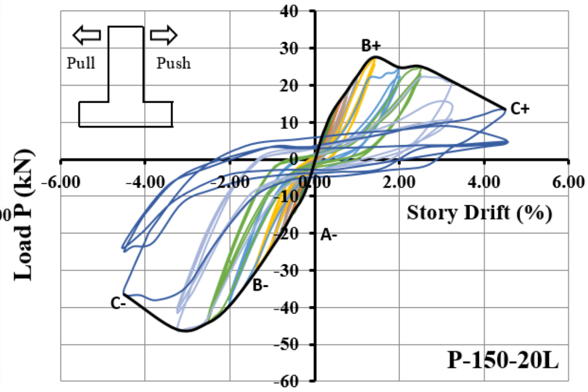
(c) P-50-20



(d) P-100-16



(e) P-100-20L



(f) P-150-20L

Fig.8 Hysteresis relation of lateral load-deformation

Table 1 Lateral load capacity and damping ratio

Specimen	P_{u_push}	P_{u_pull}	P_{u_Theory} (kN)[3]	Failure mode	Damping ratio (%) (Story drift >3%)
M-100	25.26	26.20	25.33	Flexural	22-25
	Avg. (P_{u_Ma}[1]) 25.73				
P-100-20	30.46 (1.18)[2]	30.53 (1.19)[2]	39.33	Flexural	15-21
P-50-20	32.00 (1.24)[2]	29.86 (1.16)[2]	39.33	Flexural	18-22
P-100-16	23.66 (0.92)[2]	12.86 (0.50)[2]	25.33	Slip of threaded dowel	3-4
P-100-20L	31.00 (1.20)[2]	30.93 (1.20)[2]	39.33	Fracture of welding	7-15
P-150-20L	27.46 (1.07)[2]	45.93 (1.79)[2]	39.33	Fracture of welding	5-6

[1]P_{u_Ma} the average of push and pull ultimate capacity of specimen M-100

[2]Numbers in parenthesis and ultimate capacity ratio compared with the P_{u_Ma}

[3]P_{u_Theory} is calculated based on the ultimate capacity of maximum shear and moment section (at column base). The calculated strengths are controlled by flexure

4.2 Energy Dissipation Capacity

The energy dissipation capacity of a structure indicates the degree of effectiveness of the structure to withstand earthquake loading. In the present study, the quantity of energy dissipation is shown in term of the equivalent viscous damping ratio (ζeq), recommended by Chopra [16]. The equivalent viscous damping ratio is obtained from Fig.9. The equivalent damping ratio corresponding to drift ratio of all six experiments are shown in Fig.10. The equivalent damping ratios were between 18-25 percent after drift ratio more than 3 percent, as seen in Table 1.

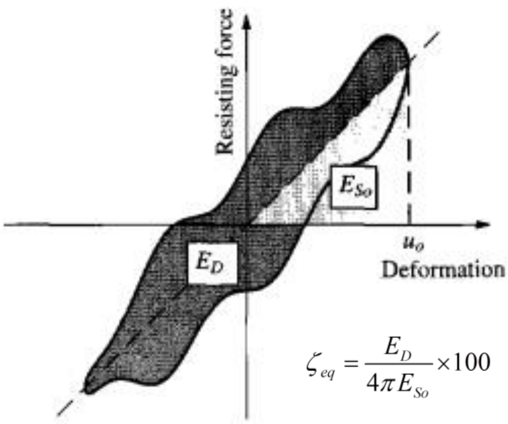


Fig.9 Determination of the equivalent viscous damping ratio (ζ_{eq}) [16]

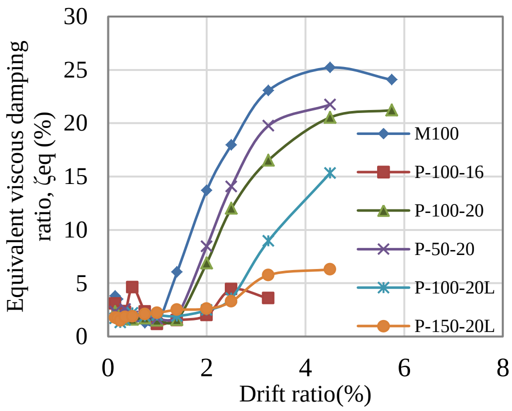


Fig.10 Equivalent damping ratio at different drifts

5. CONCLUSION

This paper presents the lateral cyclic load test and seismic design of precast concrete column using steel box connection based on the capacity design concept. First, seismic design of reinforced concrete framing beam is evaluated for the demand capacity. Then, the guideline on how to obtain the seismic design force for the columns is made. The design of precast concrete column with the steel box connection is explained. To illustrate the applicability of the designed steel box connection, lateral cyclic load tests of precast concrete columns were made and the results were compared with a resemble reinforced concrete column. The test results show the good seismic performances of the designed steel box connection. The columns with connection possessing the flexural strength and stiffness higher than the concrete column above the connection performed the ductile behavior. The flexural yielding strength and stiffness of the box were 57.9 kN-m and 5.88 kN-m^2 respectively, providing 1.5 and 2.3 times relatively higher than the ultimate flexural capacity and non-cracked flexural stiffness of the reinforced concrete column section. It is noted that welding of the column reinforcement to the steel box need to be carefully made. The ultimate capacity of the precast column is higher compared with the cast-in-place concrete column, due to the relocation of the failure section. Hence, based on the capacity design concept, the weak beam-strong column can be easily achieved.

6. ACKNOWLEDGEMENTS

The authors would like to express their sincere appreciation to Chiang Mai University for the financial support.

7. REFERENCES

[1] Park R. and Paulay T., Reinforced Concrete Structures, John Wiley & Sons. New York, pp.1-769.

[2] Justin A.M. and Mehrdad S., Seismic Shear-Axial Failure of Reinforced Concrete Columns VS. System Level Structural Collapse, Engineering Failure Analysis, Vol. 32, 2013, pp.382-401.

[3] Lu, X., Urukap, T. H. Li, S. and Lin, F., Seismic Behavior of Interior RC Beam-Column Joints with Additional Bars Under Cyclic Loading, Earthquakes, and Structures, Vol.3, No.1, 2012, pp. 37-57.

[4] Bindhu K.R., Jaya K.P. and Manicka Selvam, V.K., Seismic Resistance of Exterior Beam-Column Joints with Non-conventional Confinement Reinforcement Detailing, Struct. Eng. Mech., Vol.30, No.6, 2008, pp. 733-761.

[5] Han S. W. and Jee N. Y., Seismic Behaviors of Columns in Ordinary and Intermediate Moment Resisting Concrete Frames, Engineering Structures, Vol.27, No.6, 2005, pp. 951-962.

[6] Elwood K.J. and Moehle J.P., Drift Capacity of Reinforced Concrete Columns with Light Transverse Reinforcement, Earthquake Spectra, Vol. 21, Issue. 1, 2005, pp. 71-89.

[7] Elwood KJ, Moehle JP., An Axial Capacity Model for Shear-Damaged Columns. ACI Structural Journal; Vol. 102, Issue 4, 2005, pp.578–587.

[8] Rejame M. F. C., Eimair B. E. and Ana L. H., Analysing the Base of Precast Column in Socket Foundation with Smooth Interfaces, Materials and Structures, Vol.42,Issue 6, 2009, pp. 725-737.

[9] Negro P. and Toniolo G., Design guidelines for connections of precast structures under seismic actions. Report EUR 25377 EN, European Commission, 2012.

[10] Ghosh, S. K. and Hawkins N. M., Seismic Design Provisions for Precast Concrete Structures in ACI318, PCI Journal, Vol.46, Issue 1, 2001, pp.28-32.

[11] Kuntz G. L. and Browning J., Reduction of Column Yielding During Earthquakes for Reinforced Concrete Frames, ACI Structural Journal, Vol. 100, Issue 5, 2003, pp. 573-580.

[12] ACI318, Building Code Requirements for Reinforced Concrete, American Concrete Institute, Detroit, Michigan, 2014.

[13] AISC, Steel Construction Manual, American Institute of Steel Construction (4^{th} Edition), 2011.

[14] Hansapinyo C., Buachart C. and Wongmetha P., Cyclic Performance of Precast Concrete Columns Using Steel Box Connection,

International Journal of Civil Engineering, Vol. 15, Issue 4, 2017, pp. 663–676.

[15] ACI T1.1-01, Acceptance Criteria for Moment Frames Based on Structural Testing, American Concrete Institute, 2001.

[16] Chopra, A. K., Theory and Applications to Earthquake Engineering: Dynamic of structure (3^{rd} Edition), Prentice Hall, Englewood Cliffs, NJ, 1999.