

ASSESSMENT OF PRECAST WIDE U BEAM-COLUMN SYSTEM FOR MITIGATION OF STRUCTURAL FAILURE FROM SEISMIC HAZARD IN THAILAND

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Abstract

A large number of low to moderate earthquake incidents have been witnessed in the northern and western parts of Thailand. These incidents usually lead to various damage levels in various structures. Therefore, a sufficient lateral force resisting structures should be designed for the construction in such earthquake-prone areas. This paper presents the relevance of using a typical composite wide U beam-column system for mitigation of structural failure and collapse from the seismic hazard in Thailand. The investigated composite structural system consists of precast U beams, precast columns, and cast-in-situ concrete over the precast U beams and in the joint regions. The system also includes post-tensioning in the cast-in-situ area of the composite beams. The system has widely been used in low- to mid-rise commercial building in Thailand. In order to scrutinize the seismic performance of the system, a half-scale specimen of the post-tensioned precast U beam-column joint was tested under an incremental displacement-controlled lateral cyclic load. The laboratory test revealed the drift capacity, damage pattern, strain development in the rebars etc. of the precast wide U beam-column interior joint. Based on the obtained drift capacity, displacement-based assessment of the system was carried out. In addition, the observed damages on the tested structure were evaluated using damage grading criteria for building subjected to earthquake. The assessment showed a high potential of the post-tensioned precast wide U beam-column system to mitigate the structural failure and collapse from the earthquake hazard. However, some improvements in the current seismic performance of the system are required to employ the system in high seismic zones.

Keywords: Beam-column joint, Damage assessment, Precast U beam, Seismic hazard

Introduction

Earthquake is a natural hazard with a number of extreme incidents such as ground vibration and rupture, landslide, liquefaction, and tsunami [1]. Each of these effects can cause disaster to the human society, if appropriate mitigation techniques are not well implemented. Among various effects of earthquake, structural failure and collapse due to ground shaking is one of the major causes of human and economic losses [2]. Therefore, a special attention is paid to the design and construction of earthquake resistant structures. It is noted that design and detailing of the structures are required to be in accordance with level of the seismicity in order to preclude over- or under-design of the structures [3].

Although Thailand is located in a low and sparse seismicity area, a number of small to moderate earthquakes have been reported [4]. These earthquakes lead to various levels of damage in various structures. For example, the Mae Lao earthquake in 2014 affected a total of 10,863 buildings to various degrees of damage [5]. A report by the Department of Public Works and Town & Country Planning [6] showed that out of 10,863 buildings, the number of unsafe, repairable and minorly damaged buildings were 594, 2,376 and 7,893, respectively. In order to prevent and mitigate such devastation, a robust lateral load resisting system against seismic load is required for such earthquake-prone areas. Also, the use of precast construction is gaining popularity in Thailand. Therefore, this paper investigates the potential of a precast wide U beam-column system as one of the possible lateral load resisting frames for buildings in Thailand.

Precast wide U beam-column system is a composite system, which consists of precast columns, precast U beams with post-tensioned cast-in-situ concrete. This system possesses several advantages over the traditional beam-column system, e.g., lower inter-story height, faster construction, elimination of formworks and economical construction.

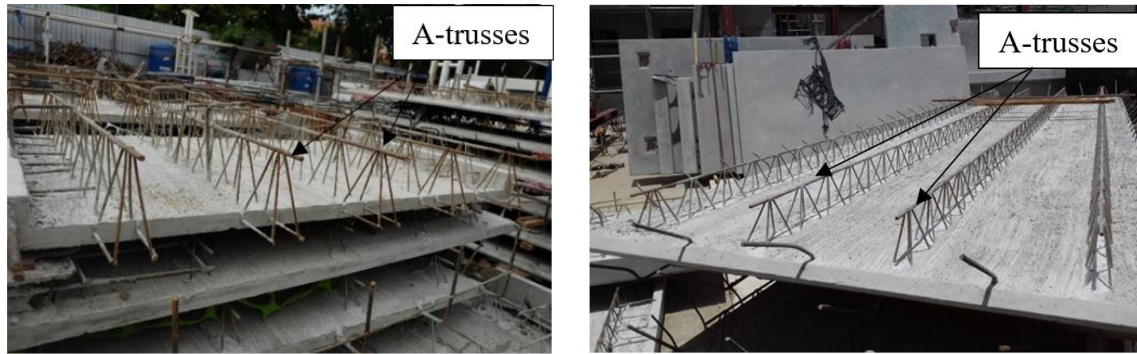
Precast wide U beam-column system has been widely used in low- to mid-rise commercial buildings in Thailand. However, actual seismic performance of this system is yet to be revealed. Bohara et al. (2019) [7] and Bohara et al. (2018) [8] carried out numerical investigations on the seismic performance of post-tensioned precast wide U beam-column interior and exterior joints, respectively. Since the numerical models were based on some assumptions, experimental study is essential to reveal the actual seismic performance of the system. Therefore, this paper aims to experimentally evaluate the performance of the post-tensioned precast wide U beam-column system. The obtained experimental results are assessed with respect to the seismicity of Thailand. A simple but rational and practical approach is implemented to classify the damage on the tested structure subjected to seismic loading. Based on the displacement-based assessment and damage grading, the potential of the precast wide U beam-column system for prevention and mitigation of structural failure is evaluated.

In this paper, seismic performance of the post-tensioned precast wide U beam-column system is evaluated with respect to the drift capacity and consequent damage levels under lateral cyclic loading. A half-scale interior beam-column joint of the system was tested under a quasi-static lateral cyclic loading. In order to determine the suitability of the system in various seismic zones of Thailand, a displacement-based assessment was carried out with respect to the drift capacity of the joint. Furthermore, the observed crack and damage patterns were investigated to assess the potential of the system to mitigate structural failure and damage from seismic hazard in Thailand.

Details of Precast Wide U Beam-Column System

The major components of precast wide U beam-column system are precast U beams and slabs with embedded A-trusses, precast columns and cast-in-situ concrete with post-tensioning. It is noted that the A-trusses work as a shear connection between the precast and the cast-in-situ concrete. Figure 1 shows the precast U beam and slab with A-trusses.

The construction methodology of the precast wide U beam-column system is as follows. First, the precast concrete lower storey columns with extended longitudinal rebars are erected. These extended rebars are inserted into the mechanical splices installed in the upper storey column, which are, finally, filled up by grout injection.



a. Precast U beam

b. Precast slab

Figure 1. Precast U beam and precast slab with A-trusses

After the erection of columns of a lower storey, scaffolding is set up and the precast panels are installed over it. The precast wide U beams are laid along the weak axis of the building and the one-way precast slab panels are spanned on these U-beams. These precast panels (U beam and slab) serve as permanent formworks. It is worth noting that no precast member is used in the vicinity of the joint but only the cast-in-situ concrete is used in the joint region. The adoption of cast-in-place concrete only in the joint region is expected to ameliorate the structural integrity of the precast system. A typical layout of the precast U beams and precast slabs during construction is shown in Figure 2.

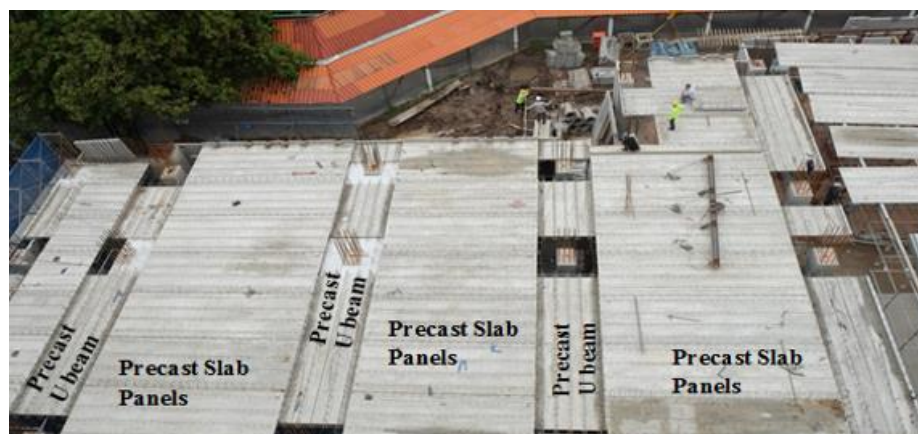


Figure 2. Precast U beam-column system during construction

After the installation of precast members and formwork in the joint region (as shown in Figure 2), joint reinforcements, beam rebars, and post-tension cables are set up. Then, cast-in-situ concrete is poured over the precast members. Post-tensioning is applied to the installed cables four days after casting of the cast-in-situ concrete.

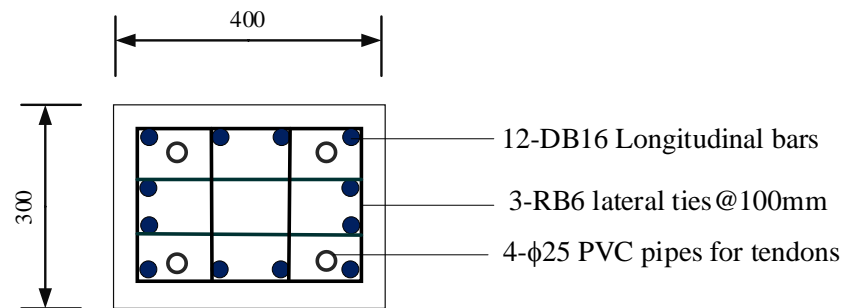
Experimental Program

Specimen Details

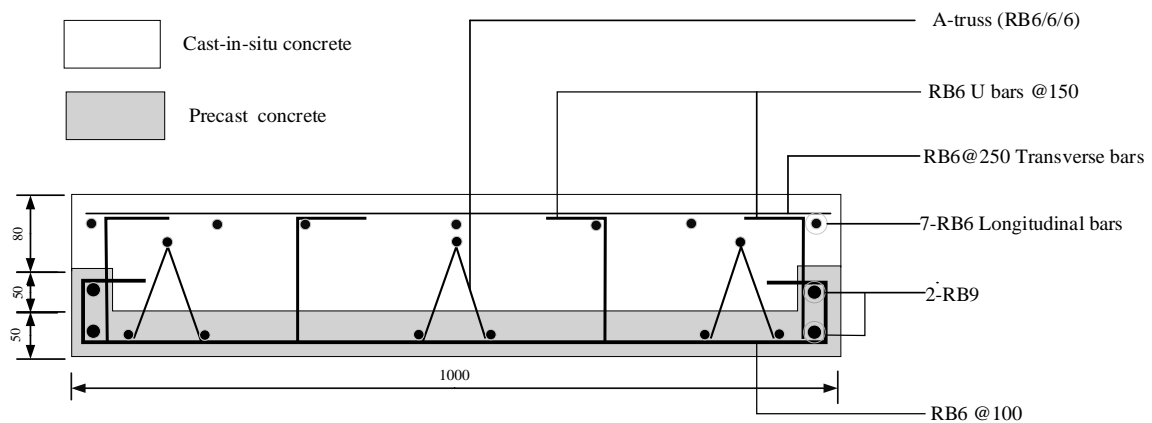
In order to investigate the seismic performance of the precast wide U beam-column system, a half-scale specimen of an interior joint of the system was tested in our laboratory. It is noted that the tested specimen represents beam-column joints used for Advanced Laboratory

Building of Sirindhorn International Institute of Technology, where the investigated joint is between two unequal spans. Therefore, the tested specimen has unequal beam span lengths. The details of the half-scale specimen are presented in Figure 3.

A precast column with dimensions of 300mm×400mm×1800mm consists of 12-DB16 rebars as longitudinal rebars and 3-RB6@100mm as lateral ties. Four PVC pipes with a diameter of 25mm are provided in the precast column to insert pre-stressing tendons (see Figure 3a). The elevation sizes of the half-scale composite wide beam are 180mm×1000mm, which consist of two layers of concrete (50mm precast and 130mm cast-in-situ). RB6 U bars @150 mm and three rows of RB6 A-trusses are rooted in the precast U beam (see Figure 3b). At the top and bottom of each A-truss, additional RB6 bars are welded. At the corner of precast U beam, 2-RB9 rebars are installed and tied with RB6@100mm bars. 7-RB6 bars are provided throughout the span as top longitudinal bars. Furthermore, RB6 rebars are distributed at the spacing of 250mm in transverse direction.



a. Details of precast column



b. Details of wide beam

Figure 3. Details of precast column and composite wide beam (unit: mm)

As shown in Figure 4, three layers of 6-RB9 rebars having a length of 1.5m are distributed in the joint region. On the top reinforcement layer in the joint region, extra 6 RB9 rebars with a length of 1.5m are provided. Also, 4-DB12 bars are provided in both longitudinal and transverse directions of the joint. The spacing of U bars is reduced to 100mm in the joint region.

7-wire 2-strand tendons with a parabolic profile were used to prestress the U-beam. The layout and profile of the tendons are presented in Figure 5. It is worth mentioning that an extra 250 mm cast-in-situ concrete was added at prestressing end of the beam in order to provide a uniform prestressing force to the section of the wide beam. Therefore, distances of clamps from beam ends on short and long spans are 550 mm and 300 mm, respectively.

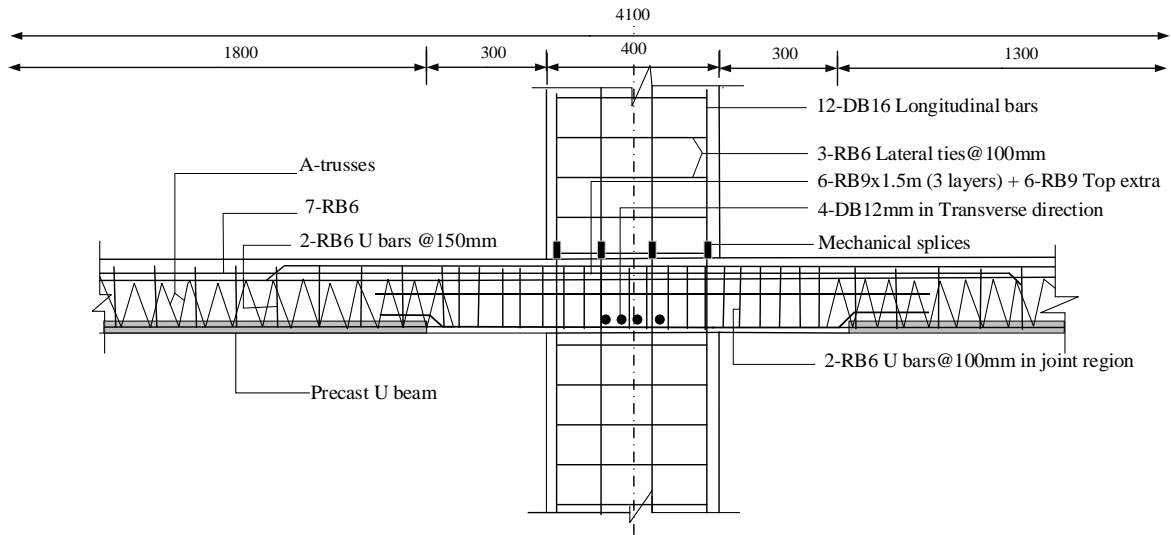
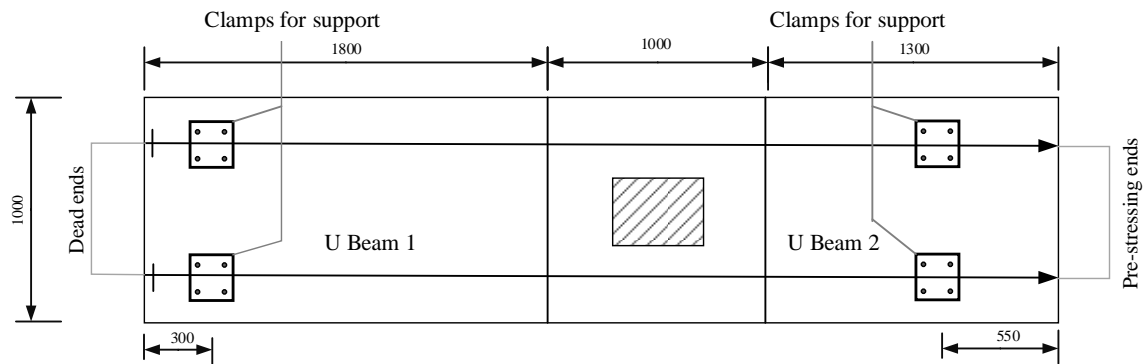
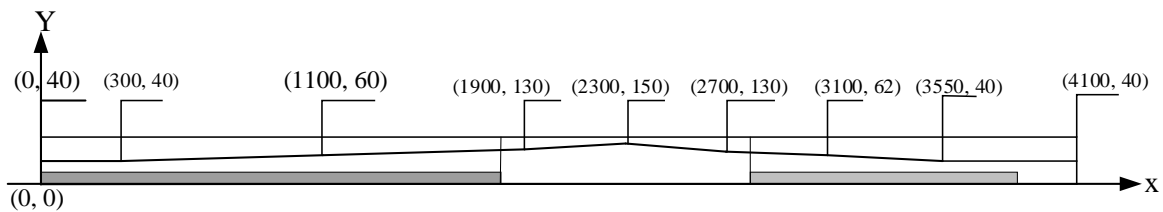


Figure 4. Longitudinal section of precast wide U beam-column interior joint (unit: mm)



a. Tendon layout



b. Tendon profile

Figure 5. Details of tendons (unit: mm)

Test Set up and Loading

Similar to the previous researches [9][10], only the portion between inflection points was prepared and set up for the testing. The ends of the beam were supported with rollers whilst hinge support was assigned at the column base. An axial load on the column was applied through four prestressed tendons. Each tendon was subjected to an axial load of 9.8 tons. The application of the axial load on the column represents the gravity load from upper stories. A detailed illustration of the test set up is shown in Figure 6.

The specimen was subjected to incremental displacement-controlled laterally cyclic load. The applied drifts were $\pm 0.10\%$, $\pm 0.35\%$, $\pm 0.60\%$, $\pm 0.85\%$, $\pm 1.10\%$, $\pm 1.35\%$, $\pm 1.85\%$, $\pm 2.35\%$, $\pm 2.85\%$, $\pm 3.85\%$ and $\pm 4.85\%$. Each drift cycle was applied twice (2 cycles in each drift). During the test, lateral force reaction, lateral displacement of the top column, crack propagation, crack widths and strains in the rebars were recorded.

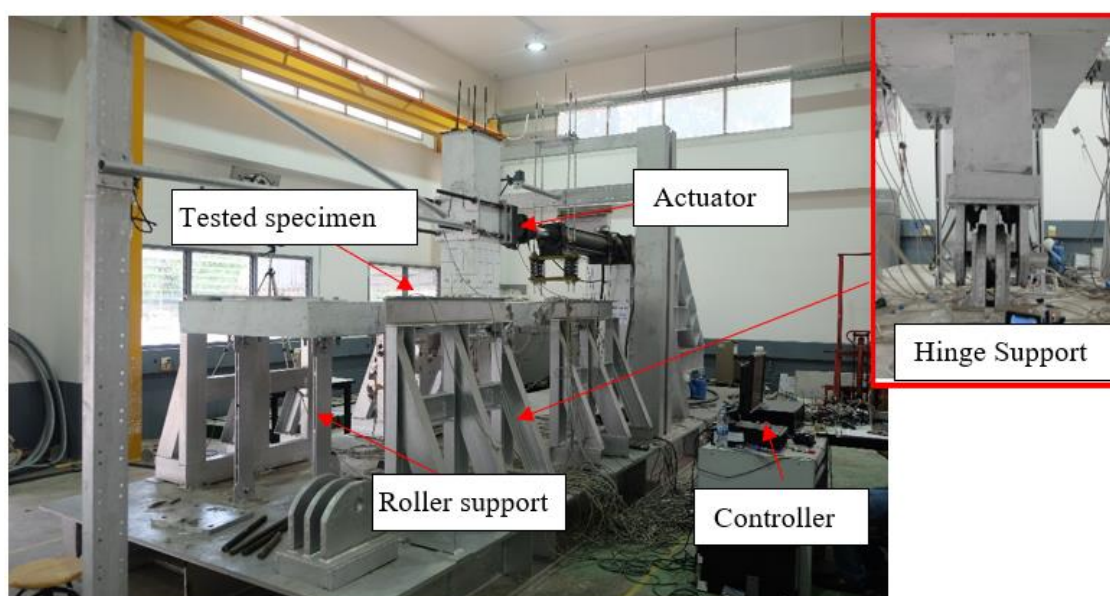


Figure 6. Test set up of the wide U beam-column joint specimen under lateral cyclic load

Displacement-based Assessment

The damage and failure of a structure under lateral load take place when it reaches certain limit inter-story drift. Therefore, it is preferable to compare the inter-story drift demand and the drift capacity of the building [11]. In addition, displacement-based assessment of the structure directly links the structural performance to the ground motion, and provides the practical medium to relate the laboratory results to the real structural performance in the site during seismic events.

The drift demand of the structure is the function of seismicity of the building site location and characteristics of the structure. In displacement-based assessment of the structure, the inter-story drift demand of the structure is calculated based on the specific earthquake data and characteristics of the building. On the other hand, the drift capacity of the structure is evaluated by experiment or numerical analysis or analytical calculation. Then, the drift capacity and drift demands are compared to assess the performance of the structure for a specific seismic event. Alternatively, the safe PGA corresponded to the drift capacity of the system can be compared with the actual PGA during a particular earthquake.

In this study, similar to Goldworthy and Abduka (2012) [12], an elastic spectral pseudo acceleration shape was assumed for firm ground site locations. In the assumed elastic spectral pseudo acceleration, a transition of constant acceleration to constant velocity was considered to take place at 0.5sec with a damping of 5% of the critical damping.

Based on the moment magnitude of earthquake (M_w), a corner period of the displacement spectra (T_c) is calculated using Equation 1.

$$T_c = 1 + 2.5(M_w - 5.7) \quad \text{in second} \quad (1)$$

Drift capacity (θ_{max}) obtained from the experiment is used to estimate the displacement capacity at effective height (Δ_f) by using Equation 2.

$$\theta_{max} = F_\theta \frac{\Delta_f}{H_e} \quad (2)$$

In Equation 2, F_θ which accounts for higher modes and stiffness variation in different stories is taken to be 1.3 [9]. H_e is the effective height of the building which can be calculated from $0.66H_n$, where H_n is the height of the building.

Based on fundamental period of building, $T_f (=0.1H_n)$ and T_c , suitable equations (Equation 3 or 4) for Δ_f is selected and maximum elastic response displacement (Δ_{TC}) is calculated.

$$\Delta_f = \Delta_{TC} \frac{T_f}{T_c} \quad (\text{if } T_f \leq T_c) \quad \text{in meter} \quad (3)$$

$$\Delta_f = \Delta_{TC} \quad (\text{if } T_f > T_c) \quad \text{in meter} \quad (4)$$

Finally, the safe PGA on the firm soil with respect to an earthquake of moment magnitude M_w is estimated by using Equation 5.

$$PGA = \frac{\Delta_{TC}}{0.031 \times (1 + 2.5(M_w - 5.7))} \quad \text{m/s}^2 \quad (5)$$

Results and Discussion

Determination of Drift Capacity of the System

The hysteresis curve (force-displacement) obtained from the experiment is shown in Figure 7. The solid and dashed lines in Figure 7 represent the hysteresis curves for the first cycle and second cycle of loading, respectively. The tested post-tensioned precast wide U beam-column interior joint exhibited the drift capacity (drift at peak load) of 3.35% and 2.85% in positive and negative cycles, respectively. The asymmetric drift capacity of the joint was mainly due to its unequal beam span lengths. The specimen was tested till 4.85% drift in both positive and negative cycles.

After attaining the peak load, the specimen exhibited a rapid load drop (28% of the peak load) with the increased drift ratio. Crushing of concrete was observed on the wide beam near the column face. Ductility factor (μ), which is the ratio of ultimate displacement (Δ_u) to yield displacement (Δ_y), was adopted to evaluate ductility of the structure. As per JSCE guideline [13], Δ_y is the displacement when the main tensile reinforcement yields, and Δ_u is the maximum displacement at which load is equal to yield strength. Definition of Δ_y

and Δ_u are presented in Figure 8a [13]. The values of Δ_y and Δ_u for the tested specimen are shown in Figure 8b. It is worth mentioning that longitudinal bars of both column and beam yielded at 2% drift, which was equivalent to the lateral displacement of 36 mm. More details of the measured strain in rebars will be presented in the section “Strain Development in the Joint Core”.

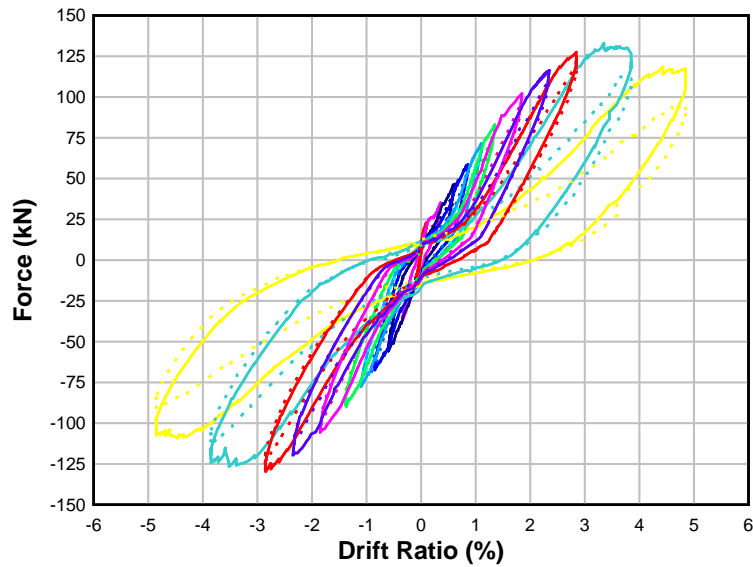
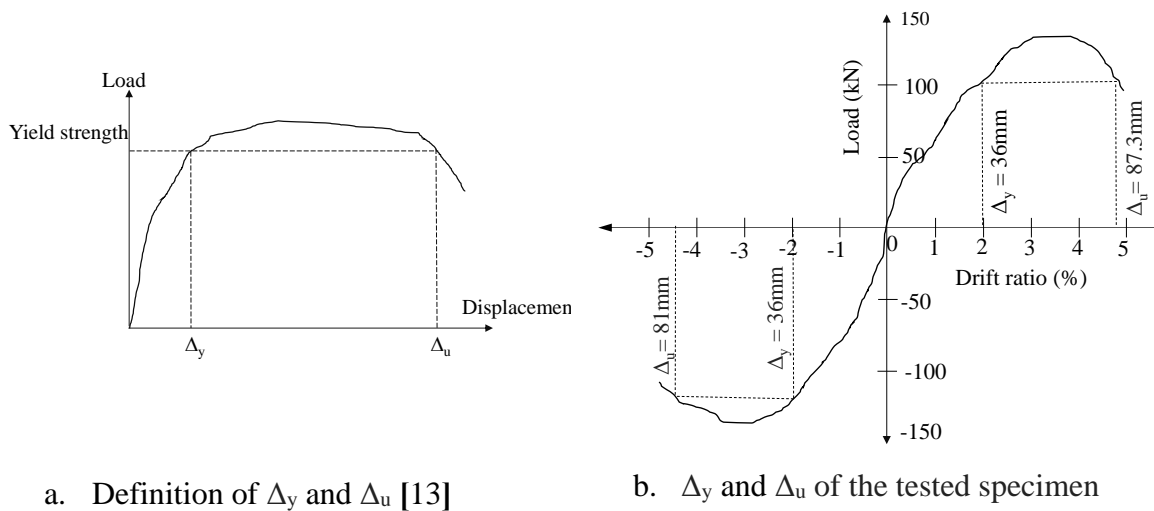


Figure 7. Hysteresis curve



a. Definition of Δ_y and Δ_u [13]

b. Δ_y and Δ_u of the tested specimen

Figure 8. Yield and ultimate displacement of the specimen

Based on the above-mentioned criteria, the values of μ were obtained to be 2.42 and 2.25 in positive and negative loading cycles, respectively. A value of μ greater than 1 and less than 1.25 represents a nominally ductile structure. Limited ductile structure and ductile structure exhibit μ in the range of 1.25 to 3, and 3 to 6, respectively [14]. Therefore, the tested structure exhibited a limited ductile behavior. Furthermore, the thin hysteresis loops indicate poor energy dissipation of the system under this loading action. Therefore, some modifications in structural detailing of the specimen are required in order to make the system more ductile and be able to exhibit a higher energy absorption.

Displacement-based Assessment

The drift capacity of the joint obtained from the experiment was used for the assessment. Between the two drift capacities (in positive and negative cycles), the lower one (2.85%) was used for the assessment.

The ground motion parameters for Thailand were adopted from Ornthammarath et al. (2011) [4], Warnitchai and Lisantono (1996) [15], and Pailoplee and Charusiri (2016) [16]. In this assessment, based on the study of historical seismicity data from 1912 to 2007 presented by Ornthammarath et al. (2011) [4], a 7.0 M_w earthquake was taken to be a possible maximum magnitude of earthquake in Thailand. A probabilistic seismic hazard (PSH) map based on 50-year return period and 2% probability of being exceeded by Pailoplee and Charusiri (2016) [16] were the basis for adoption of peak ground acceleration (PGA). Based on the study by Pailoplee and Charusiri (2016) [16], 0.50g was taken as the maximum PGA for Thailand. A summary of ground motion parameters to cover an entire area of Thailand is shown in Table 1.

Table 1. Ground Motion Parameters to Cover an Entire Area of Thailand

Parameters	Notation	Value
Moment magnitude of earthquake	M_w	7.0
Peak ground acceleration	PGA	0.5g

An eight-storey building with 3.6 m story height was considered as an example for the analysis. Therefore, the height of the building (H_n) was 28.8 m. The building parameters used for the displacement-based assessment are given in Table 2.

By using the equations 1 to 5 and parameters in Table 1 and Table 2, the safe PGAs for the system were estimated. The calculated parameters of displacement-based assessment are presented in Table 3.

Table 2. Building Parameters for Displacement-based Assessment

Parameters	Notation	Formulation	Value
Fundamental period of building	T_f	$0.1H_n$	2.88 sec
Effective height of the building	H_e	$0.66H_n$	19 m

Table 3. Summary of Calculated Parameters

Parameters	Value
T_c	4.25 sec
Δ_{TC}	0.61 m
Δ_f	0.42 m
Safe PGA for firm soil	0.47g

The safe PGA, thus obtained, is for the firm soil. In order to determine the safe PGA for intermediate and soft soil, the safe PGA was reduced by factors of 1.4 and 1.8, respectively. The obtained safe PGAs for firm, intermediate and soft soil are presented in Table 4.

Table 4. Safe PGAs on Different Soil Types

Soil	M_w	PGA
Firm		0.47g
Intermediate	7.0	0.33g
Soft		0.26g

These safe PGAs were compared with the contours of PGAs obtained from the PSH map by Pailoplee and Charusiri (2016) [16] (shown in Figure 9). Considering the firm soil condition, the capacity of the tested system fulfils the demand of all zones except the vicinity of Kanchanaburi (P4). However, the drift capacity of the system based on intermediate and soft soil conditions is insufficient in several locations in Thailand. For example, the tested system cannot fulfil the drift demand for the construction in intermediate soil in Tak (P10). In addition, the drift performance of the tested system is insufficient for the soft soil sites in Phuket (P8), Ranong (P9), and many northern areas of Thailand (P2, P3, P5, P6 and P7). Therefore, a modification in the detailing of the system is recommended so that it is able to fulfil the drift demand on all the seismic zones and soil types in Thailand.

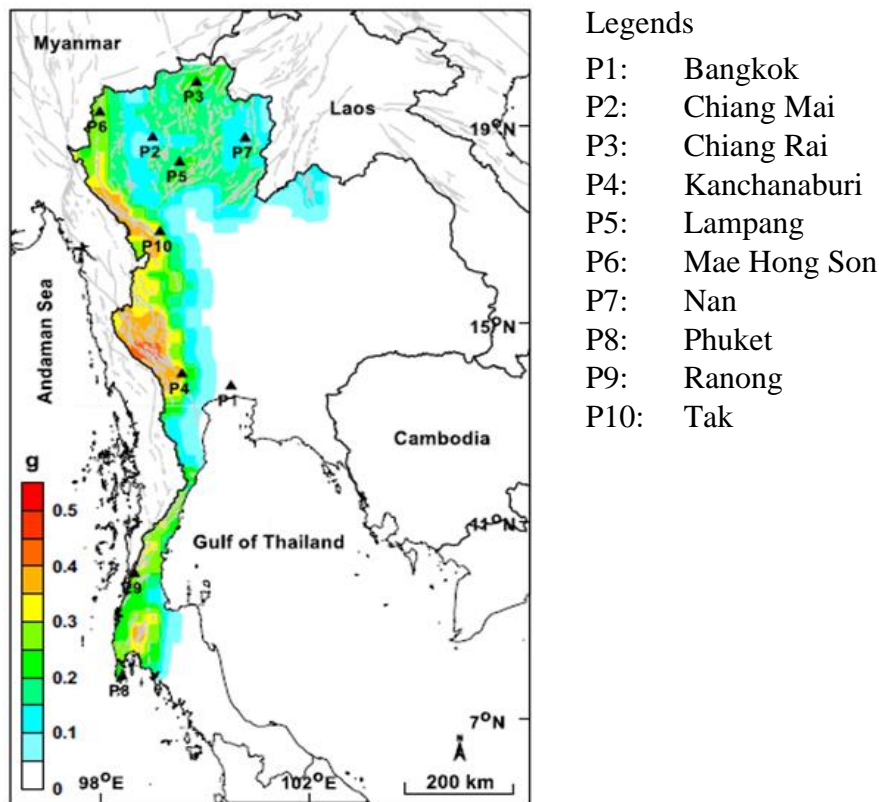


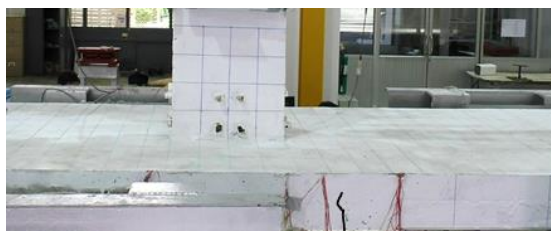
Figure 9. Contours of PGA for an earthquake of 2% of probability of being exceeded within the next 50 years (Pailoplee and Charusiri) [16]

Crack Patterns

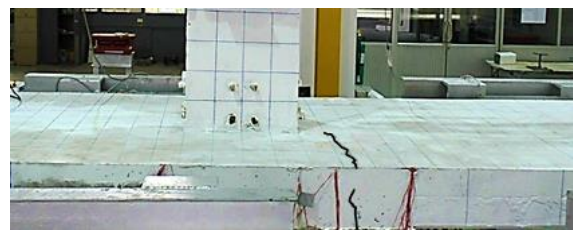
The crack patterns observed at various drift ratios are presented in Figure 10. The tested specimen exhibited the first crack at the second cycle of 0.35% drift ratio on the longer span (U Beam 1 in Figure 5a). The first crack originated from the bottom face of the wide beam and propagated upward to the upper face of the beam (see Figure 10a). A decrease in the slope of hysteresis curve was observed due to the formation of this crack (see Figure 7).

The first transverse crack on the top face of the wide beam was initiated at the drift ratio of 0.60% (see Figure 10b). This crack was located exactly above the first crack, which merged with this newly formed crack at 1.10% drift ratio. These transverse cracks were generated due to flexural action in the beam.

The first diagonal shear crack appeared at the drift ratio of 1.10% (see Figure 10c). After 1.10% drift ratio, several diagonal shear cracks were observed on both short and long spans of the tested joint specimen. Two diagonal cracks crossing each other on the beam near side face of the column appeared at the drift ratio of 1.85% (see Figure 10d). With the increasing drift ratio from 1.85% to 3.85%, these cracks became longer and reached the outer edge of the beam at 3.85% drift ratio (see Figure 10f). These cross-cracks in the joint witnessed the presence of torsional force in the joint vicinity when subjected to lateral cyclic load.



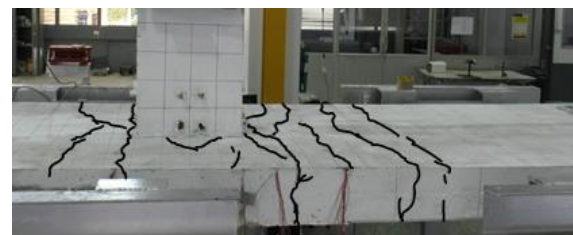
a. Drift Ratio 0.35%



b. Drift ratio 0.60%



c. Drift ratio 1.10%



d. Drift ratio 1.85%



e. Drift Ratio 2.35%



f. Drift Ratio 3.85

Figure 10. Crack patterns at various drift ratios

Even though the wide beam experienced severe damage, there is no significant damage on the column up to 4.85% drift ratio. The diagonal shear cracks and transverse flexural cracks were the major crack types observed on the tested beam-column joint structure. Based on the observed damage on the tested specimen, it is worth mentioning that the specimen showed a strong column – weak beam behaviour. Only minor flexural cracks were observed on the column and there is no sign of rebar buckling failure in the column during the lateral cyclic load test. However, in addition to the aforementioned cracks, wide beam exhibited crushing. The crack pattern and crushing of concrete around the joint region of the tested specimen at 4.85% drift ratio is shown in Figure 11.

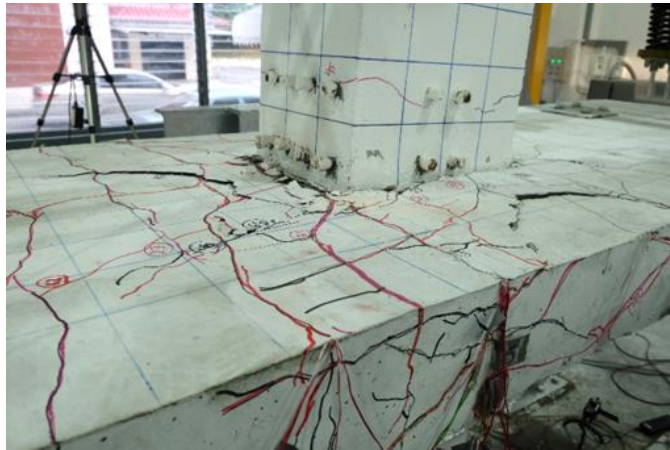


Figure 11. Crack pattern and concrete crushing around the joint region of the tested specimen at 4.85% drift ratio

Strain Development in the Joint Core

Beam-column joint core plays a vital role in maintaining the safety of a building frame. Therefore, design of an intact joint core is essential for a robust lateral load resisting structural system. In order to examine the behavior of the joint core of the tested specimen, strain gauges were attached on the longitudinal bars in the column of the joint region. The strain development in the column longitudinal reinforcement bars at different locations, and the first yielding of the longitudinal bar in beam are presented in Figure 12 and Figure 13, respectively.

Among the measured rebars, depending on the location the column cross section, the rebars yielded at different drift ratios. The rebar ‘a’ yielded first at 2.00% drift ratio in negative cycle and 2.15% drift ratio in positive cycle. Then, the rebar ‘b’ yielded at 3.00% drift ratio in both cycles. The rebar ‘c’ reached the yielding strain at 4.85% drift ratio in the negative loading cycle only. This deviation of strain in positive and negative loading cycles was mainly due to asymmetric span lengths of the beam. During the early cycles of lateral load, strain measured at different rebars did not exhibit a significant difference. However, strain of different rebars varied remarkably after 1.50% drift ratio. In comparison to the mid rebar (rebar ‘a’ in Figure 12), the corner rebars showed smaller strain in the joint core. This behavior was due to the presence of prestressed tendons (used to apply axial load to the column) near the corner rebars. Since some part of the tensile stress in the joint core is carried by the tendons, strains in the nearby longitudinal rebars was found to be lower than those in other positions.

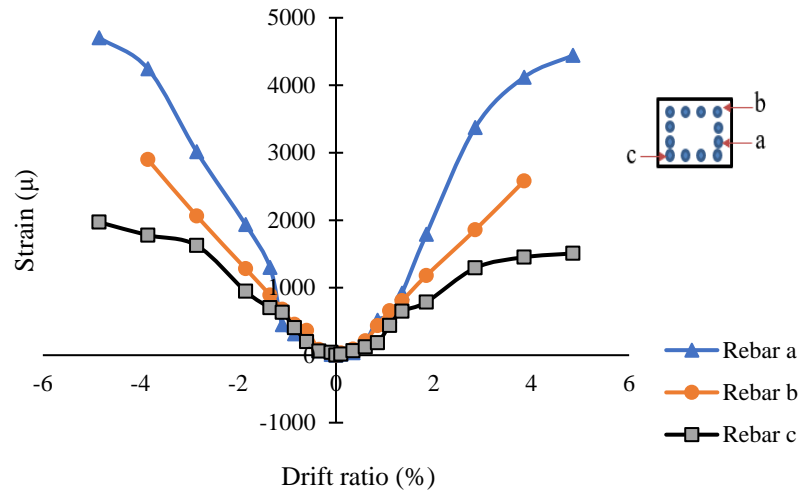


Figure 12. Strain development in the column rebars in the joint core

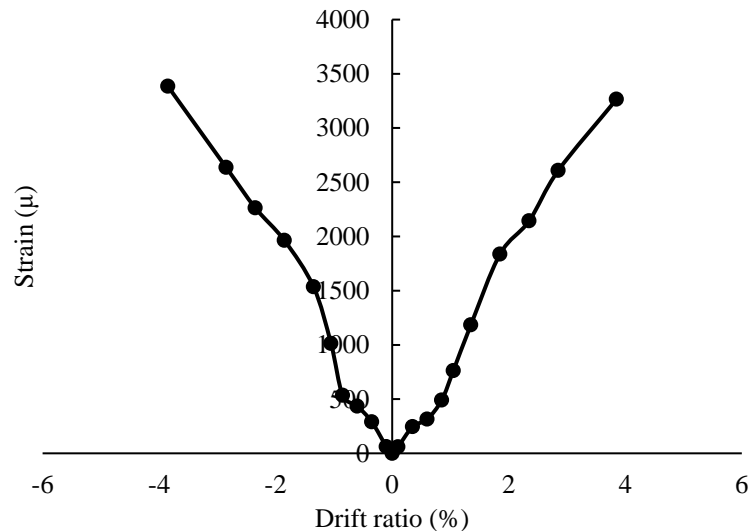


Figure 13. Strain development in the beam longitudinal bar passing through the joint core

The yielding of rebars in the joint core can introduce some damages in the joint core, which should be prevented. Therefore, some improvements in the detailing of the joint core are required to apply to the system constructed in a high seismic zone. In this system, the width of the beam is wider than that of the column. As a result, bending stresses generated due to cyclic loading in the outer zone (away from column) of beam cannot be directly transmitted to the column core. These residual bending stresses on beam generate torsion. Therefore, additional reinforcement to account for this torsion should be considered for modification of reinforcement detailing in the future. It is noted that the torsional cracks can be observed in Figure 10d, 10e and 10f.

Damage Level Grading

The Japan Building Disaster Prevention Association [17] classifies damage levels on structural members into five classes. A summary of the damage levels on the structural member in different damage classes is presented in Table 5.

Table 5. A Summary of Damage Classes and Respective Damage Levels on Structural Members [18]

Damage Class	Damage Level on the Structural Member
I	<ul style="list-style-type: none"> • A few cracks having width smaller than 0.2 mm
II	<ul style="list-style-type: none"> • Visible cracks having width in the range of 0.2-1.0 mm
III	<ul style="list-style-type: none"> • Spalling of concrete covering • 1.0-2.0 mm wide cracks
IV	<ul style="list-style-type: none"> • Significant crushing of concrete leading to exposure of reinforcing bars • Cracks wider than 2.0 mm
V	<ul style="list-style-type: none"> • Buckling of rebars • Damage in concrete core • Column deformation • Side-sway

Based on the classification given in Table 5 and the observed damages of the tested structure, damage assessment of the tested structure was conducted. Since the damage is more severe at higher drift ratio, damage class of the tested structure was classified at different drift ratios. Table 6 presents the damage classes of the tested structure at different drift ratios. The tested specimen exhibited the maximum damage class IV at 4.85% drift ratio.

Table 6. Damage Classes of the Tested Structure at Different Drift Ratios

Drift Ratio (%)	0.10	0.35	0.60	0.85	1.10	1.35	1.85	2.35	2.85	3.85	4.85
No. of Cracks	0	1	4	12	17	24	43	48	55	64	69
Maximum Crack Width (mm)	-	<0.2	<0.2	<0.2	<0.2	0.2	0.8	1.2	2	5	6
Damage Class	-	I	I	I	I	II	II	III	III	IV	IV

Conclusions

In this paper, potential of precast wide U beam-column system is investigated to mitigate the structural failure from seismic hazard. A half scale specimen of post tensioned precast wide U beam-column interior joint was tested under displacement controlled lateral cyclic loading. The test results show that the tested specimen exhibited a strong column – weak beam behavior, which is desirable for seismic design of structures. The tested structure did

not collapse till 4.85% drift ratio. However, it is noted that the tested structure showed abrupt strength degradation (load drops by 28% of the peak load). In addition, the tested system exhibits limited ductility with the ductility factors of 2.42 and 2.25 in positive and negative loading cycles, respectively.

Drift capacity obtained from the experiment was used to carry out a displacement-based assessment. The assessment of seismic performance of the tested structure indicates that the drift capacity of the tested system fulfils the demand of all zones with firm soil except the vicinity of Kanchanaburi province. However, the drift capacity of the system based on intermediate and soft soil conditions is insufficient in several locations in Thailand.

Therefore, some future improvements in structural detailing of the system are required to prevent the observed abrupt strength degradation, limited ductility and insufficient drift capacity in order to make the system suitable for all the seismic zones and soil sites in Thailand.

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Glossary of Symbols

Δ_f	Displacement capacity at effective height (meter)
Δ_{TC}	Maximum elastic response displacement (meter)
Δ_u	Ultimate displacement (millimeter)
Δ_y	Yield displacement (millimeter)
F_θ	Factor for higher modes and stiffness variation in different stories
H_e	Effective height of the building (meter)
H_n	Height of the building (meter)
M_w	Moment magnitude of earthquake
PGA	Peak ground acceleration (m/s^2)
T_c	Corner period of the displacement spectra (second)
T_f	Fundamental period of building (second)
θ_{max}	Drift capacity
μ	Ductility factor

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