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# Performance assessment of precast concrete pier cap system

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**Abstract.** The purpose of this study was to investigate the performance of precast concrete pier cap system. The proposed precast pier cap provides an alternative to current cast-in-place systems, particularly for projects in which a reduced construction time is desired. Five large-scale pier cap specimens were constructed and tested under quasistatic monotonic loading. The computer program, RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology) was used for the analysis of reinforced concrete structures. A bonded tendon element is used based on the finite element method, and can represent the interaction between the tendon and concrete of a prestressed concrete member. A joint element is used in order to predict the inelastic behaviors of segmental joints with a shear key. This study documents the testing of the precast concrete pier cap system under monotonic loading and presents conclusions and design recommendations based on the experimental and analytical findings. Additional full-scale experimental research is needed to refine and confirm design details, especially for actual detailing employed in the field.

**Keywords:** performance; precast concrete pier cap; construction time; quasistatic; computer program

## 1. Introduction

Improved speed of construction and economy can be achieved through the use of precast bridge substructures (Matsumoto *et al.* 2001). A shortened construction time, in turn, leads to important safety and economic advantages when traffic disruption or rerouting is necessary. Precasting also eliminates the need for forming, casting, and curing of concrete in the work zones, making bridge construction safer while improving quality and durability.

Recently, various studies have been carried out in America, Taiwan and Korea on the inelastic behavior and performance of precast segmental bridge columns (Billington *et al.* 2001, Hewes 2002, Billington and Yoon 2004, Chou and Chen 2006, Wang *et al.* 2008, Yamashita and Sanders 2009, Marriott *et al.* 2009, Ou *et al.* 2010, Kim *et al.* 2010a, Kim *et al.* 2010b). Precast segmental construction of concrete bridge columns is a method in which bridge columns are segmentally

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prefabricated off site and erected on site typically with post-tensioning. Recent developments, although limited in number, have shown that precast segmental bridge columns are feasible and advantageous for a wide variety of project types.

The aim of this study is to establish the behavior of precast concrete pier cap and to formulate a design procedure. The proposed precast pier cap system provides an alternative to current cast-in-place systems (Young 2000, Young *et al.* 2002), particularly for areas where reduced construction time is desired.

The development of a precast concrete pier cap system is expected to be an important step in the advancement of precast substructures (Sumen 1999, Waggoner 1999). In addition, a precast pier cap system could accommodate special construction conditions, such as sites with difficult access or harsh environments more easily than cast-in-place systems.

This paper will present simulations performed in large-scale experiments on precast concrete pier cap. This study involved both the experimental and analytical investigations of the behavior of the precast pier cap system under quasistatic monotonic loading.

An evaluation method for the performance of precast concrete pier cap system is proposed. The proposed method uses a nonlinear finite element analysis program (RCAHEST, Reinforced Concrete Analysis in Higher Evaluation System Technology) developed by the authors (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007b, Kim *et al.* 2007c, Kim *et al.* 2008, Kim *et al.* 2009). A modified joint element is incorporated into the structural element library for RCAHEST so that it can be used to predict the inelastic behaviors of segmental joints with a shear key for the precast segmental pier cap system.

## 2. Proposed precast concrete pier cap system

Fig. 1 shows the developed precast segmental PSC bridge columns with a shear resistant connecting structure. The ends of each column segment have a shear resistant connecting structure to facilitate shear transfer between segments. They also play an important role in its performance in terms of hysteretic energy dissipation and ductility (Kim *et al.* 2010a). The segments are precast with aligned ducts to allow for the threading of post-tensioning strands through the column once the segments are placed in the field. The introduction of post-tensioning in the substructure has the potential to reduce residual displacements and improve joint shear performance. The precast concrete footing system is made up of three basic types: precast concrete footing segment, headed bars with coupler and cast-in-place footings (Kim *et al.* 2010b). After the shaft is drilled, spread footings or pile cap foundations at the bridge site are completed, and the precast concrete footing segment can be hauled to the site for erection. The precast footing segment is match-cast in its vertical position. Detailed information is given in Kim *et al.* (2010a, 2010b).

The aim of this study is to develop precast concrete pier cap for precast segmental PSC bridge columns. Fig. 2 shows the design concept of the proposed precast segmental pier cap system. Precast pier cap systems eliminate the need for forming, reinforcement, casting, and curing of concrete on the jobsite removing the precast pier cap construction from the critical path.

A criterion for the proposed precast concrete pier cap is that the system be compatible with developed precast segmental PSC bridge columns. The connection between the column and pier cap was similar to column-to-column as shown in Fig. 2. A segmentally precast pier cap system consists of relatively small, easily handled segments.

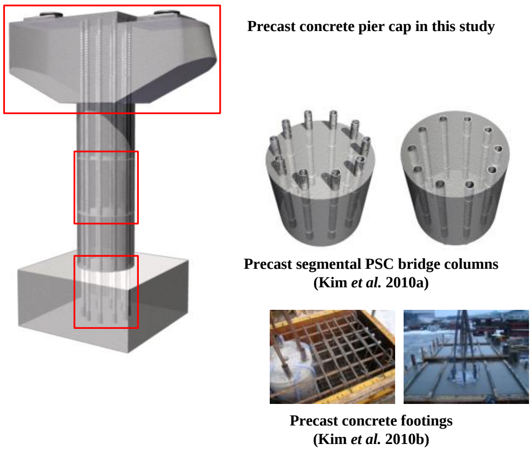



Fig. 1 Developed precast segmental PSC bridge column system

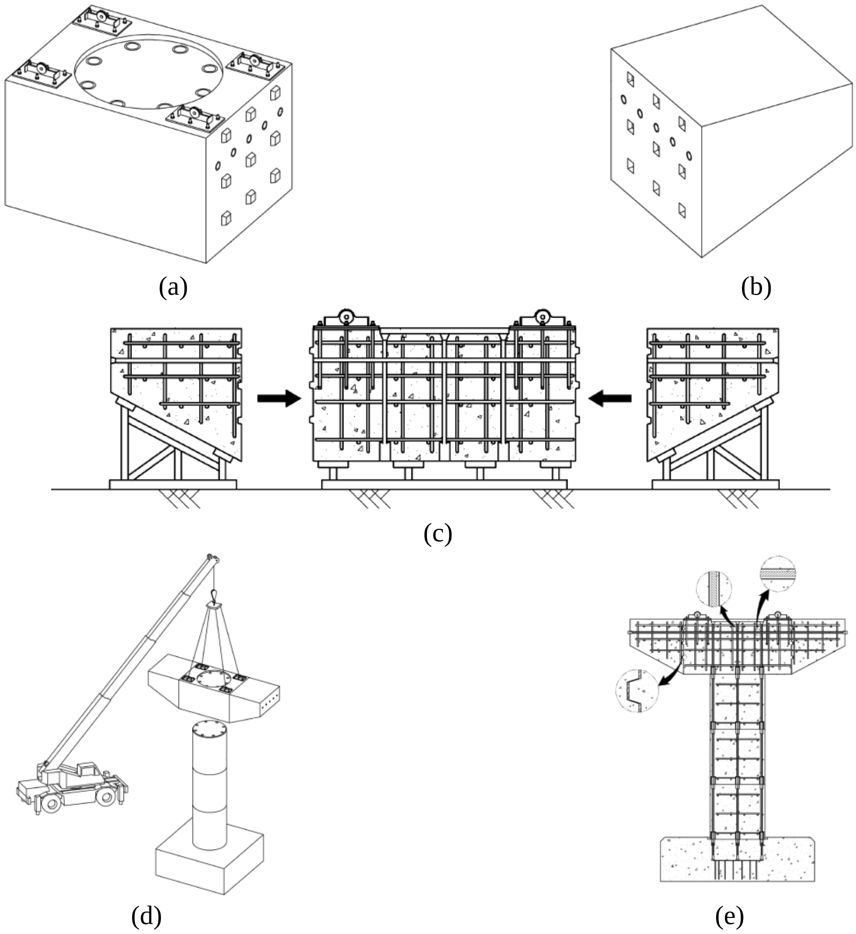



Fig. 2 Proposed precast concrete pier cap system: (a) primary segment; (b) secondary segment; (c) assembly of precast concrete pier cap segment; (d) installation of precast concrete pier cap segments; and (e) completion of precast concrete pier cap system

The precast concrete pier cap segment is match-cast in its horizontal position. Connection details are developed based primarily on constructability and economic considerations. However, because the integrity of the precast pier cap system depends on the connection performance, representative details should be tested. This will ensure a proper understanding of structural behavior and thus help develop a conservative approach for analysis and design.

## 3. Investigation of precast concrete pier cap

### *3.1 Experimental investigation*

Two precast pier cap specimens were designed for testing under monotonic loading, designated as PC-HS0, PC-HS1. In addition, two prestressed concrete pier cap specimens and one reinforced concrete pier cap specimen were designed for conventional pier cap under monotonic loading, designated as PSC-HS0, PSC-HS1, and RC-HS1. The specimens are designed in accordance with Section 1 and 4 of KRBD (2005). PC-HS0 and PSC-HS0 had vertical web reinforcement and no horizontal web reinforcement. The segment joints are also designed with sufficient shear friction resistance to prevent sliding. The mechanical properties of the specimens are listed in Table 1 and the geometric details are shown in Fig. 3 through Fig. 5.

Precast pier cap specimens were expected to exhibit the least ductility and design strength, and were intended to provide a baseline result for comparison with the other three specimens.

The specimens consisted of precast segments. The pier cap specimens were 675 mm by 2900 mm by 650 mm (width by length by height). The precast segments were connected with a shear key, and had no continuous bonded reinforcing across the segmental joints. Each precast pier cap specimen (PC) and prestressed concrete pier cap specimen (PSC) had four prestressing strands.

Fig. 6 shows the construction sequence for the proposed precast concrete pier cap system. The precast segments of the specimens were fabricated. To maximize construction speed and substructure durability, a system of match-cast segments with epoxy joints was developed. The joint material placed in these locations must be durable and the tendons must be protected. When the pier cap segment has been assembled, post-tensioning strands are tensioned to a predetermined stress level to satisfy both service and ultimate limit state requirements for the pier cap.

All the specimens were simply supported and loaded through at the midspan, as shown in Fig. 7. Bearing plates of $700 \times 400 \times 150$ mm (width$\times$length$\times$height) were used at the supports for all specimens and also at the loading point.

The specimens were quasistatically loaded in incremental force control using two 2000 kN actuators. Each specimen was instrumented to measure the mid-span deflection and strains in both the vertical and horizontal web reinforcement.

The load-versus-deflection relationships for specimens are shown in Fig. 8 through Fig. 12. Figs. 8 through 12 also show the design shear strength of the pier cap and the damage pattern of the specimens at failure. The design shear strengths obtained from the design code (KRBD, 2005) are conservative for five pier cap specimens (PC-HS0, PC-HS1, PSC-HS0, PSC-HS1, and RC-HS1). The pier cap specimens might have gained some extra strength through arch action with the support system.

In all the test specimens, flexural cracks first occurred near or at the section of maximum moment. Upon first crack, specimen stiffness began to decrease, and this continued as more cracks

Table 1 Properties of test specimens

| Specimen | | Precast concrete pier cap | Prestressed concrete pier cap | Reinforced concrete pier cap |
|---|---|---|---|---|
| Length (mm) | | | 2900 | |
| Height (mm) | | | 650 | |
| Width (mm) | | | 675 | |
| Prestressing steel | Material | 2-2@ Φ 15.2 mm seven-wire strands | | |
| | Yielding strength (MPa) | 2026 | | |
| | Prestressing force (MPa) | 1302 | | |
| Longitudinal reinforcement | Material | | | D19 |
| | Yielding strength (MPa) | | | 567 |
| Web reinforcement | Material | | D10 | |
| | Yielding strength (MPa) | | 490 | |
| Strength of concrete (MPa) | | 42.1 | 50.3 | 31.2 |

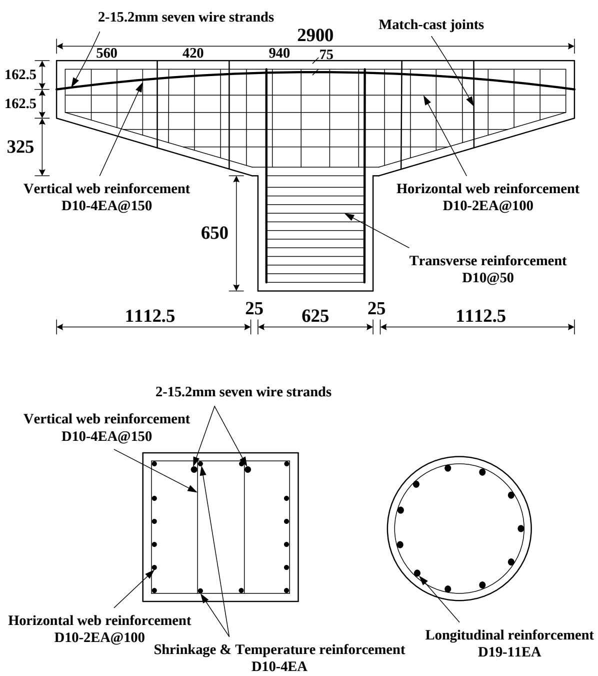



Fig. 3 Details of precast concrete pier cap specimen

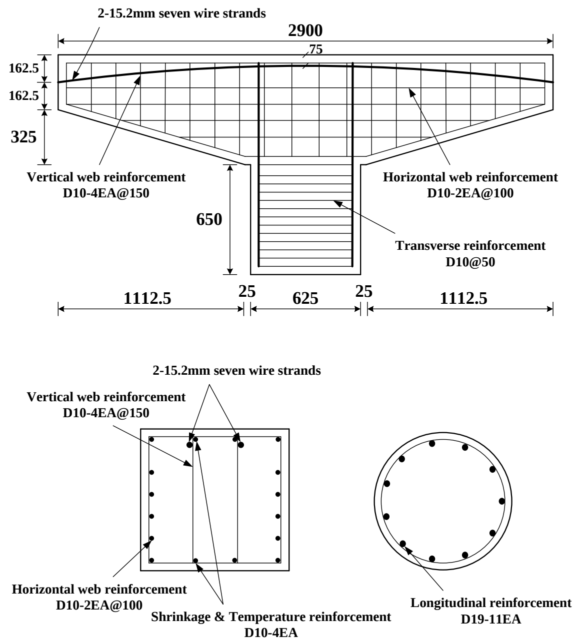



Fig. 4 Details of prestressed concrete pier cap specimen

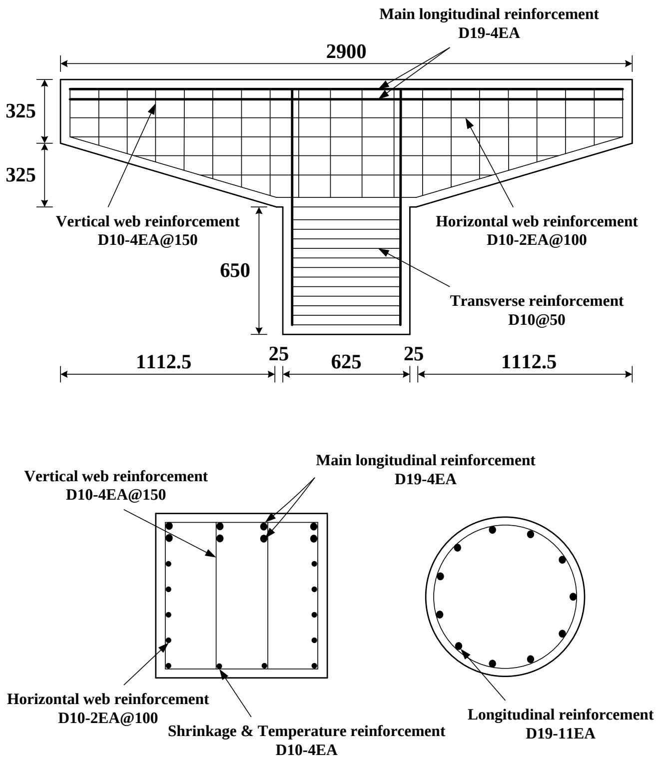
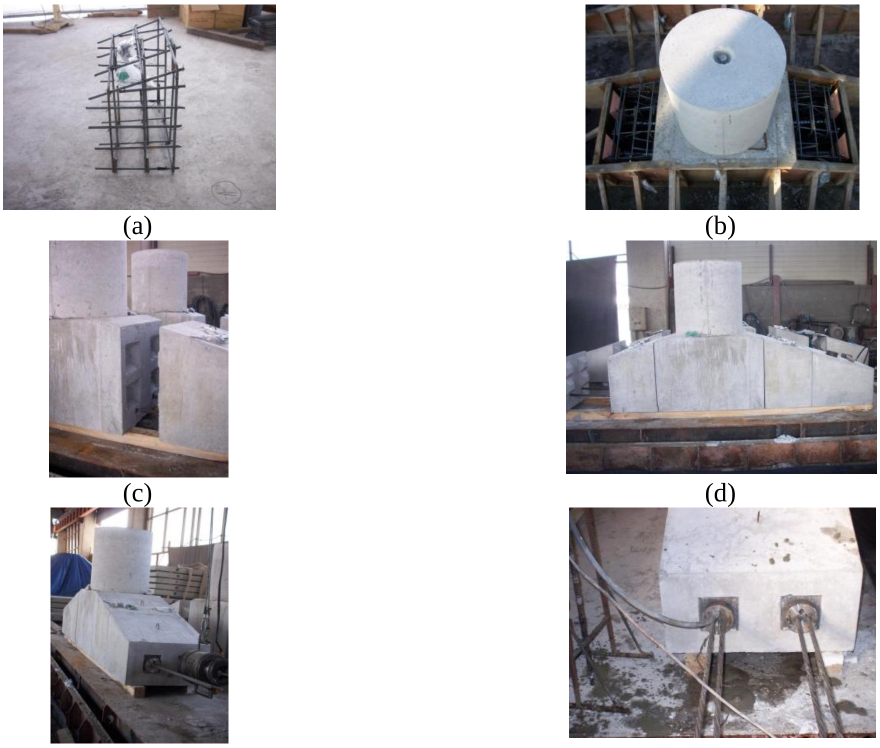



Fig. 5 Details of reinforced concrete pier cap specimen



(a) (b)
(c) (d)
(e) (f)

Fig. 6 Construction sequence: (a) secondary segment; (b) match cast of segment; (c) application of epoxy; (d) installation of segment; (e) post-tensioning; and (f) pressure grouting operation

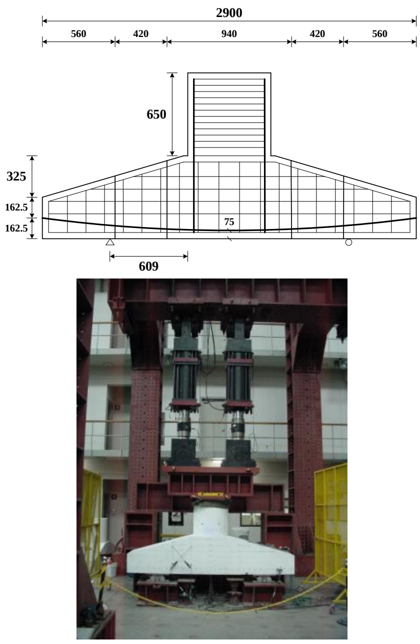



Fig. 7 Loading setup

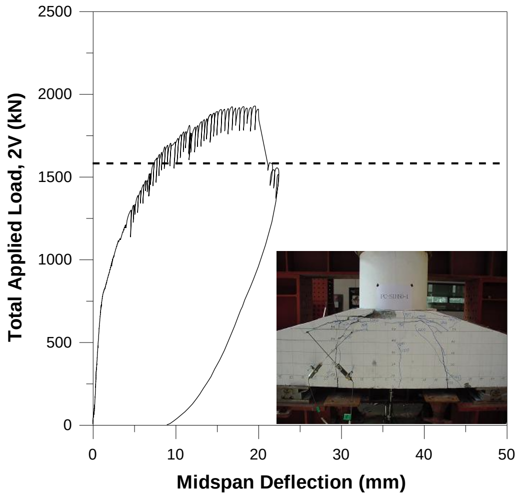



Fig. 8 Load-versus-deflection relationship for specimen PC-HS0

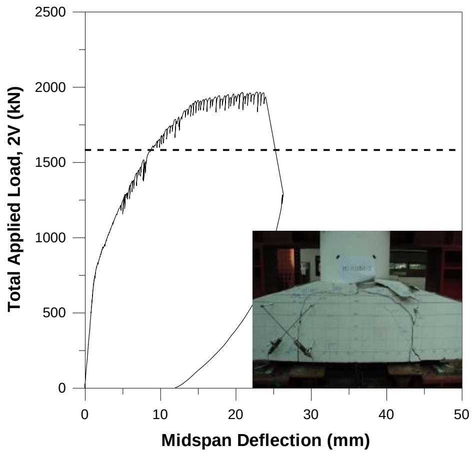



Fig. 9 Load-versus-deflection relationship for specimen PC-HS1

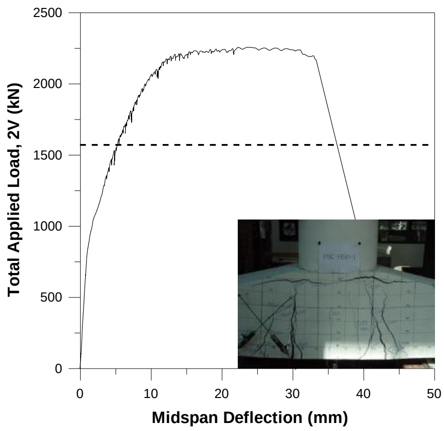



Fig. 10 Load-versus-deflection relationship for specimen PSC-HS0

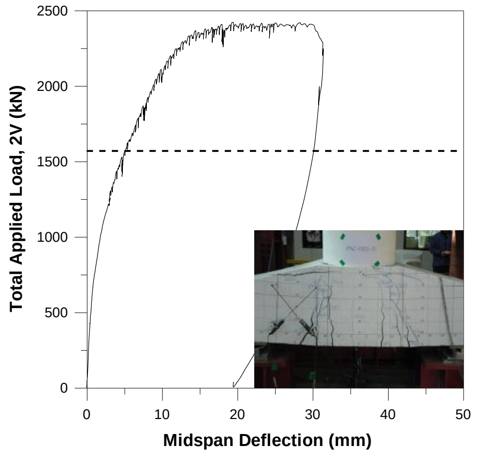



Fig. 11 Load-versus-deflection relationship for specimen PSC-HS1

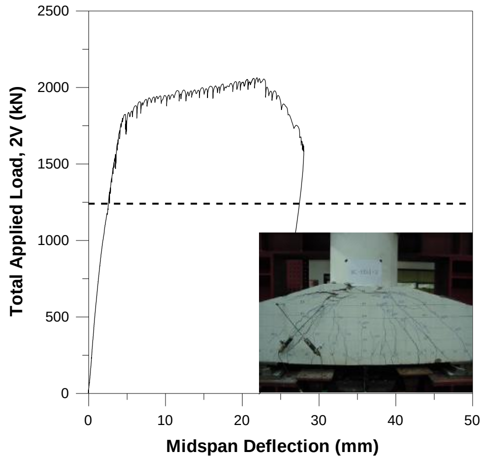



Fig. 12 Load-versus-deflection relationship for specimen RC-HS1

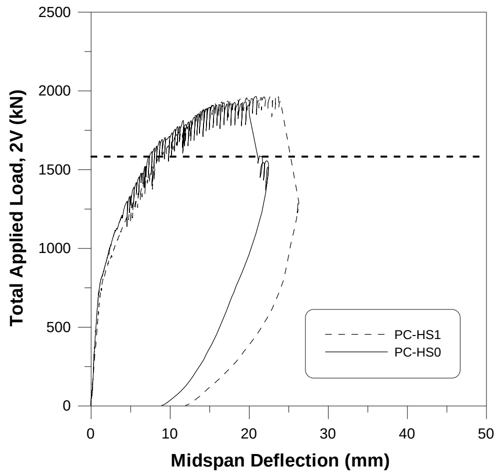



Fig. 13 Load-versus-deflection relationships

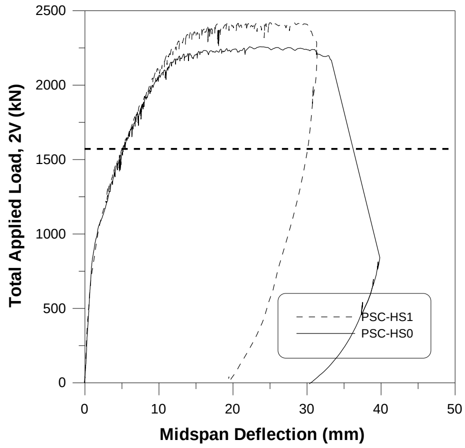



Fig. 14 Load-versus-deflection relationships

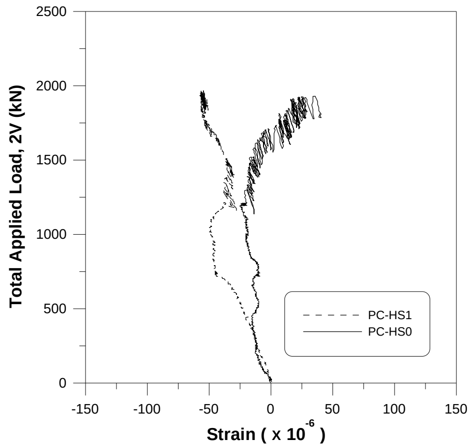



Fig. 15 Load-strain curves of vertical web reinforcement

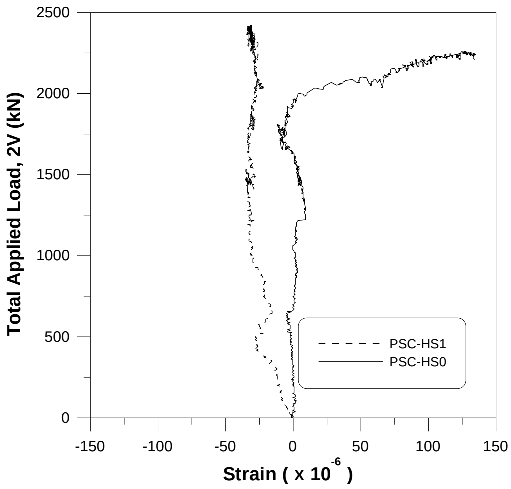



Fig. 16 Load-strain curves of vertical web reinforcement

Table 2 Experiment results and comparison with predictions

| Specimen | $2V_d$ (kN) | Experiment | | | |
|---|---|---|---|---|---|
| | | $2V_{cr}$ (kN) | $2V_u$ (kN) | $\delta_{cr}$ (mm) | $\delta_u$ (mm) |
| PC-HS0 | 1580.0 | 720.0 | 1927.2 | 1.4 | 21.2 |
| PC-HS1 | | 740.0 | 1965.0 | 1.5 | 25.2 |
| PSC-HS0 | 1568.8 | 650.0 | 2254.8 | 1.1 | 34.9 |
| PSC-HS1 | | 480.0 | 2421.4 | 0.7 | 31.5 |
| RC-HS1 | 1237.4 | 117.0 | 2061.4 | 0.3 | 26.8 |

developed. A significant decrease in stiffness was observed with the major inclined crack formation in the shear span. Increasing the shear resistance and concrete core confinement effectively reduced the shear-transfer demands on the main compression strut from the applied load to the support by increasing the participation of the compression-fan region. Concentrating the load path into one region can result in a brittle load-carrying mechanism in which high-principal tensile stresses acting perpendicular to this main strut result in cracking parallel to the strut. In cases where the specimen failed in shear-compression, the load decreased abruptly upon reaching the ultimate value and failure was brittle (see Fig. 8 and 9).

Table 2 presents the total design shear strengths ($2V_d$), cracking strengths ($2V_{cr}$) and ultimate strengths ($2V_u$) of test specimens. The design strength was multiplied by two to compare the ultimate strength of pier cap specimens. This general improvement in cracking strength and serviceability can be attributed to prestressing. It can be observed that the increase in cracking strengths is greater in precast concrete pier cap than in prestressed concrete pier cap. It seems to be depends on the pier cap segment. Table 2 also shows that precast pier cap specimens showed least ductility prior to the abrupt shear failure.

Total applied load versus midspan deflection curves for pier cap with horizontal web reinforcement and without horizontal web reinforcement are shown in Fig. 13 and Fig. 14. The shear capacity is hardly influenced by the horizontal web reinforcement in precast segmental pier cap system. The two specimens differed by no more than 2%. A relatively larger reduction in shear capacity of up to 7.4% in the prestressed concrete pier cap specimen was observed.

Cracks were measured and reinforcement strain data were recorded throughout the load history. Figs. 15 and 16 show the typical measured steel strains in the vertical web reinforcement for pier cap specimens. PC-HS0 and PSC-HS0 had vertical web reinforcement and no horizontal web reinforcement. The strains of vertical web reinforcement are hardly influenced by the horizontal web reinforcement. The testing of the specimen showed that the maximum strain in the web reinforcements is lower than the yield strain (2000 microstrains).

### *3.2 Analytical investigation*

A two-dimensional finite element model for the precast concrete pier cap system is developed in this study. The model was created and analyzed using general-purpose finite element software, RCAHEST (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007b, Kim *et al.* 2007c, Kim *et al.* 2008, Kim *et al.* 2009). RCAHEST is a nonlinear finite element analysis program used for analyzing reinforced concrete structures (see Fig. 17).

The proposed structural element library RCAHEST is built around the finite element analysis program shell named FEAP, developed by Taylor (2000). The elements developed for the nonlinear finite element analyses of reinforced concrete bridge columns are a reinforced concrete plane stress element and an interface element (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007b, Kim *et al.* 2007c, Kim *et al.* 2008, Kim *et al.* 2009). Accompanying the present study, the authors attempted to implement a bonded tendon element (Kim *et al.* 2008) and a modified joint element (Kim *et al.* 2007a) for the segmental joints with a shear key (see Fig. 17).

The nonlinear material model for the prestressed concrete comprises models for concrete and models for the reinforcing bars and tendons. Models for concrete may be divided into models for uncracked concrete and for cracked concrete. For cracked concrete, three models describe the behavior of concrete in the direction normal to the crack plane, in the direction of the crack plane, and in the shear direction at the crack plane, respectively. The basic and widely-known model adopted for crack representation is based on the non-orthogonal fixed-crack method of the smeared crack concept. The post-yield constitutive law for the reinforcing bar in concrete considers the bond characteristics, and the model is a bilinear model. For prestressing tendons that do not have a definite yield point, a multilinear approximation may be required. In this study, the trilinear model has been used for the stress-strain relationship of the prestressing tendon.

Details of the nonlinear material model used are given by the authors in previous research (Kim *et al.* 2003, Kim *et al.* 2005, Kim *et al.* 2007b, Kim *et al.* 2007c, Kim *et al.* 2008, Kim *et al.* 2009). The modeling techniques of precast concrete pier cap are described in the following sections.

Fig. 18 shows the finite element discretization and the boundary conditions for two-dimensional plane stress nonlinear analyses of the pier cap specimens. The joints between the precast segments with a shear key were modeled using modified six-noded joint elements. The bonded post-tensioning tendons were modeled with two-noded truss elements that were attached at their end nodes to the concrete element nodes at the anchorage locations. The tendon is defined by local eccentricities of the tendon point at each nodal cross-section. The proposed numerical models can represent the interaction between concrete and tendon including tendon slip and friction on the interface. The modeling is based on the analysis method for reinforced concrete bridge piers with unbonded reinforcing or prestressing bars as proposed by Kim et al. (2008).

The analysis was conducted in multiple steps to simulate the actual behavior of a pier cap specimen. The pier cap system was initially loaded under a prestressing force from the tendons. An initial stress equal to the prestress in the tendons was applied to the truss elements. Finally, the pier cap specimen was subjected to the applied vertical displacements at the top, while the corresponding force determined by the shear developed at the bottom of the footing.

The comparison between the simulated and experimental load-deflection values for a sample of specimens are shown in Figs. 19 and Fig. 20. The value given by all specimens was similar to the analytical results; comparative data is summarized in Table 3. In predicting the results of the specimens, the mean ratios of experimental-to-analytical maximum strength were 0.97 at a covariance of 6%.

Specimens that did not fail in flexure also experienced diagonal splitting, which eventually led to a shear-compression failure resulting in the crushing of concrete in the compression zone of the pier cap specimen. The failure of specimens was sudden and explosive. The predictions of the failure modes of all the pier cap specimens agree with the experimental results (see Fig. 19 and 20).

In general, the analytical model presented herein correlated reasonably well with the experimentally observed behavior of the pier cap for each test. The predicted strength was higher than the actual pier cap strength. The stiffness in the simulation is greater than that of the experiment. In light of this, and of the uncertainty in the initial prestress force and the fact that the pier cap had been tested previously, it can be said that the analytical prediction concurs well with the experimental behavior.

The joints between precast segments were found to have become cracked and opened, as was expected due to the absence of continuous bonded reinforcement. In the simulation, the used joint elements representing these segmental joints had also cracked and opened.

The importance of identifying and evaluating the adequacy of simulation methods is an important and necessary step in applying performance-based assessment techniques for assessing new, enhanced performance systems under consideration. Such an assessment can help to speed the implementation of such systems in current applications.

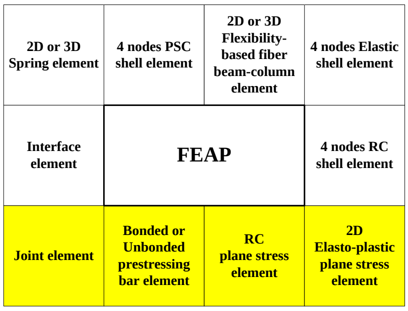



Fig. 17 RCAHEST nonlinear finite element analysis program

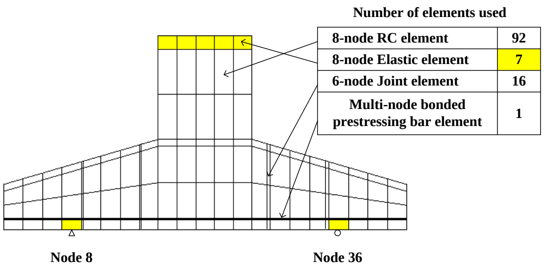



Fig. 18 Finite element mesh for precast concrete pier cap specimen

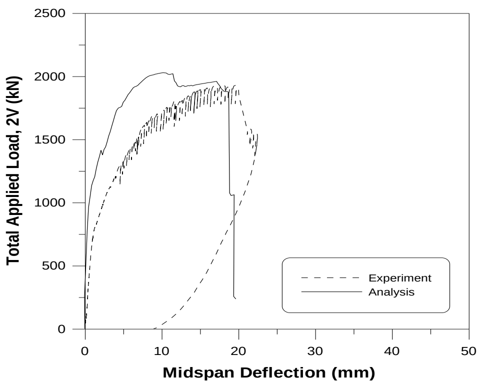



Fig. 19 Comparison of results from the experimental results (specimen PC-HS0)

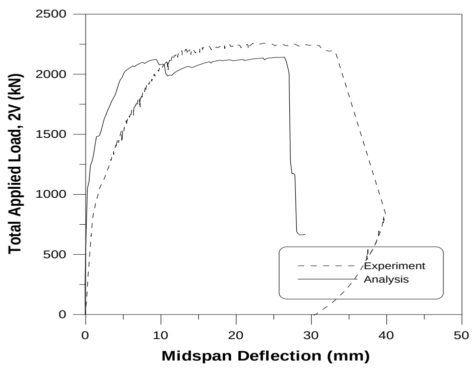



Fig. 20 Comparison of results from the experimental results (specimen PSC-HS0)

Table 3 Comparison with experimental and analytical results

| Specimen | Experiment | Analysis | Ratio of experimental and analytical results |
|---|---|---|---|
| | $2V_{\max}$ (kN) | $2V_{\max}$ (kN) | $2V_{\max}$ |
| PC-HS0 | 1927.2 | 2027.4 | 0.95 |
| PC-HS1 | 1965.0 | 2115.3 | 0.93 |
| PSC-HS0 | 2254.8 | 2139.6 | 1.05 |
| PSC-HS1 | 2421.4 | 2442.8 | 0.99 |
| RC-HS1 | 2061.4 | 2260.8 | 0.91 |
| Mean | | | 0.97 |
| Coefficient of variation | | | 0.06 |

## 4. Conclusions

This study investigated the use of precast concrete pier cap in moderate seismic regions. The proposed segmental pier cap system under investigation in this study is designed with the goal of achieving a degree of strength and ductility.

From the results of the experimental and analytical studies, the following conclusions were reached.

1. An experimental and analytical study was conducted to quantify performance measures and examine one aspect of detailing for a developed pier cap system. It was concluded that the design concepts and construction sequence are promising solutions to the application of precast concrete pier cap.

2. The presence of prestressing in the pier cap system contributes to delay cracking. The increase in cracking strengths is greater in precast concrete pier cap than in prestressed concrete pier cap.

3. In general, horizontal web reinforcement appears to have minimal influence on the ultimate shear strength in precast segmental pier cap. The strains of vertical web reinforcement are also hardly influenced by the horizontal web reinforcement. This investigation was undertaken to provide more information on the behavior of the precast concrete pier cap.

4. The concurrence between the analytical and experimental load-deflection response curves was generally sound. The joint element used seems to give a good prediction of the inelastic behaviors of segmental joints that have a shear key. Such an assessment tool can help to speed the implementation of developed systems in current applications.

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