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Hollow precast segmental PSC bridge columns with a shear resistant connecting element

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Draft

Abstract

The purpose of this study was to investigate the performance of hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element. A model of hollow precast segmental PSC bridge columns was tested under a constant axial load and a cyclically reversed horizontal load. The computer program, RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology) was used. Non-dimensional P-M interaction diagrams were developed to predict the design resistance of precast segmental PSC bridge columns. Among the numerous parameters, this study concentrates on concrete compressive strength, prestressing reinforcement ratio, effective prestress, diameter of circle passing through tendon centerlines-to-outside diameter of section ratio (D_s / D_o) and inside diameter of section-to-outside diameter of section ratio (D_i / D_o). This study documents the testing of hollow precast segmental PSC bridge columns under cyclic loading and presents conclusions and design recommendations based on the experimental and analytical findings.

Key words: hollow, precast segmental, shear resistant connecting element, P-M interaction diagram, parameters.

1. Introduction

Recently, various studies have been carried out abroad on the inelastic behavior and performance of precast segmental bridge columns (Billington and Yoon 2004; Wang et al. 2008; Kim et al. 2010a; Kim et al. 2010b; and Kim et al. 2014; Thonstad et al. 2016; Zhang and Alam 2016). Precast segmental construction of concrete bridge columns is a method in which bridge columns are segmentally prefabricated off site and erected on site typically with post-tensioning.

The use of precast segmental construction for concrete bridges has increased in recent years due to the demand for shorter construction periods and the desire for innovative designs that yield safe, economical and efficient structures (Billington et al. 2001; Chou and Chen 2006). A shortened construction time, in turn, leads to important safety and economic advantages when traffic disruption or rerouting is necessary.

Recent developments, although limited in number, have shown that precast segmental bridge columns are feasible and advantageous for a wide variety of project types. Precasting allows for an increased use of high performance concrete in substructures, thus improving durability. High performance concrete may be used more consistently with higher quality control in a precasting plant.

The aim of this study is to establish the behavior of hollow precast segmental prestressed concrete bridge columns under lateral seismic loading and to formulate a design procedure. Shear resistant connecting elements, which are continuous across the segment joints for enhanced shear transfer, were introduced in the prestressing tendon ducts.

Hollow bridge columns have become increasingly popular in bridge construction during the last few decades. Hollow sections are often used for tall bridge columns to reduce their mass, reduce seismic inertia forces, and reduce foundation forces. However, in the case of bridge columns with hollow sections, there are no specific design provisions, both in domestic and overseas codes (Lignola et al. 2011; Cardone et al. 2013; Han et al. 2014). Therefore, such hollow sections need to be studied both by tests and by analysis.

This study aims to identify the behavioral characteristics of hollow precast segmental prestressed concrete bridge columns with various design parameters. This paper deals with the effect of concrete compressive strength, prestressing reinforcement ratio, effective prestress, diameter of circle passing through tendon centerlines-to-outside diameter of section ratio (D_s / D_o) and inside diameter of section-to-outside diameter of section ratio (D_i / D_o).

This study also presents the results of parametric studies of the P-M interaction diagram of hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element. The column strength is generally checked by comparing the applied axial load and the total moment with the axial force-moment capacity of the column section. Therefore, the strength analysis of the column section may be essential to the practical design of hollow precast segmental prestressed concrete bridge columns.

An evaluation method for the performance of hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element 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. 2008; Kim et al. 2010a; Kim et al. 2010b; Kim et al. 2014).

2. Developed precast segmental prestressed concrete bridge columns

Figure 1 shows the developed precast segmental prestressed concrete bridge columns (Kim et al. 2010a; Kim et al. 2010b; Kim et al. 2014). The ends of each column segment have a shear resistant connecting element to facilitate shear transfer between segments. Shear resistant connecting elements also play an important role in the performance of the column segments in terms of hysteretic energy dissipation and ductility. 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 (see Fig. 1). 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. Vertical casting has many advantages: formed surfaces will make up all finally visible faces of the column; the concrete can be better consolidated around the ducts; and handling will

be easier, since the segments will be stored, hauled and erected in the same orientation as they were cast.

Figure 1 also shows the design concept of the precast segmental pier cap system for moderate seismic regions. 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. The precast concrete pier cap segment is match-cast in its horizontal position. Connection details are developed based primarily on constructability and economic considerations.

Detailed information is given in Kim et al. (2010a; 2010b; 2014). Recent developments, although limited in number, have shown that precast segmental bridge columns are feasible and advantageous for a wide variety of project types (see Fig. 2).

3. New hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element

Hollow bridge columns have become increasingly popular in bridge construction during the last few decades. Hollow sections are often used for tall bridge columns to reduce their mass, reduce seismic inertia forces, and reduce foundation forces (Lignola et al. 2011; Cardone et al. 2013; Han et al. 2014).

In locations where the cost of concrete is relatively high, or in situations where the weight of concrete members must be kept to a minimum, it may be economical to use hollow prestressed concrete vertical members. The hollow core also ensures greater

quality control during construction by reducing the heat of hydration on the interior of the section and hence minimizing shrinkage cracks caused by temperature differences inside the curing column.

Precast segmental bridge columns provide the benefits of increasing construction speed and quality, reducing environmental pollution, and decreasing life cycle costs. There is limited knowledge available related to the seismic behavior of segmental columns, particularly the seismic behavior of segment joints in segmental columns. The detailing of segmental joints is emphasized in the proposed design. The development of any new methods of precast bridge construction will require rigorous research on the design details of precast connections to ensure that they will perform as expected over the lifetime of the bridge. Shear resistant connecting elements that are continuous across segment joints are added to the solid columns for supplemental energy dissipation and are detailed to prematurely fracture at the critical joint (Kim et al. 2010a; Kim et al. 2010b).

The aim of this section is to verify the new hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element.

3.1 Experimental investigation

The mechanical properties of the specimens are listed in Table 1 and the geometric details are shown in Figure 3. The column specimens were tested under a $0.075f_{ck}A_g$ = 1200 kN constant compressive axial load to simulate the gravity load from bridge superstructures.

Each segmental column specimen had eighteen prestressing strands. The concrete

segments and segment joints were designed with a sufficient shear capacity to prevent shear failure. To achieve satisfactory hysteretic energy dissipation, the shear resistant connecting element was designed to be continuous across the segment joints. The segment joints were evaluated to ensure that joint shear slip failure would not occur. The confinement steel was designed to ensure that the core concrete exhibited a sufficient ductility capacity in compression. It is considered appropriate to use the current code provisions (KHBDC 2010) on the concrete confinement for the potential plastic hinge regions in the design of precast segmental columns for use in moderate seismic regions.

A schematic representation of the test set-up for the specimen is shown in Figure 4. For each test, the column footing was connected to the laboratory strong floor by high strength post-tensioning bars. The cyclic lateral point load was applied at the column top by a servo-controlled 2600 kN capacity hydraulic actuator with a ± 375 mm stroke reaching off the laboratory strong wall. Horizontal load levels in the actuator were monitored during the test through a load cell and the horizontal displacement at the actuator level was measured using a string displacement transducer and an independent reference column.

Measurements were then manually triggered based upon the lateral actuator running in displacement control. Strains in the transverse reinforcement and shear resistant connecting element were measured with the strain gauges. The strain gauges placed in several locations in the region were affected by significant inelastic flexural behavior. The post-yield strain gauges used had a resistance of $120\ \Omega$ and a 5 mm gauge length. Several measures were taken to ensure inelastic behavior of the hollow column

specimens.

For the specimens subject to cyclic loading, the loading was applied under displacement-control to drift levels of 0.25%, 0.5%, 1.0%, 1.5%, 2.0%, 2.5%, 3.0%, 3.5%, 4.0%, 4.5%, 5%, 6% and 7%. The drift was defined as the lateral displacement at the height of the loading point divided by the distance from the loading point to the top of the footing. Each cycle was repeated twice to allow for the observation of strength degradation under repeated loading with the same amplitude. Sometime, three cycles per each drift amplitude might be planned to address degradation (Brunesi et al. 2015).

The lateral load-drift responses for specimens are shown in Fig. 5. Figure 5 also shows the design shear strength of the columns and the damage pattern of the specimens at failure. The failure time is defined corresponding to a situation where the strain reaches the failure criterion. The design shear strengths obtained from the design code (KHBDC 2010) are conservative for the column specimens.

As noted for the test, the similarity in the shape of the hysteresis curves of specimens Model (1) and Model (2) is primarily due to the geometry of the test setup. The self-centering characteristic of the precast system is evidenced by the pinched hysteresis loops near the origin. The self-centering behavior was due to the horizontal component of the prestressing force, which acts on the column as it deflects laterally.

All two segmental specimens exhibited ductile behavior under cyclic loading. The ductility factors ranged from 5.3 to 5.8 (see Table 2). The hysteretic energy dissipation of the specimens was evaluated based on the cumulative dissipation energy as shown in Fig. 6. It was found that the hysteretic energy dissipation increased as the column drift

increased. Two specimens showed similar hysteretic behavior until drift 6.0%. For specimens Model (1) and Model (2), the cumulative dissipation energy were 437863 kN-mm (Drift 6.0%) and 528135 kN-mm (Drift 7.0%), respectively. In addition, the hysteretic energy dissipation increased as the shear resistant connecting element was used.

3.2 Analytical investigation

A three-dimensional finite element analysis is tedious and expensive, and requires a high number of elements to achieve a good accuracy. Therefore, two-dimensional eight-noded smeared elements are used to model the experimental specimens.

The nonlinear material model for the reinforced and 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. Thus, the constitutive law adopted for cracked concrete consists of tension stiffening, compression and shear transfer models (see Fig. 7). 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 as shown in Fig. 8. In this study, the trilinear model has been used for the stress-strain relationship of the prestressing tendon (see Fig. 9). The transverse reinforcements confine the compressed concrete in the core region and

inhibit the buckling of the longitudinal reinforcing bars. In this study, a lateral confining effect model was adopted and incorporated it into the structural element library for RCAHEST, so that it can be used to assess the performance of hollow precast segmental PSC bridge columns. This is similar to the formula suggested by Mander et al. (1988) for the triaxial stress condition, but the reduced confinement effectiveness coefficient corresponding to the ratio of the inside-to-outside diameters of the bridge column section is applied. Details of the nonlinear material models used have been provided by the authors in previous research (Kim et al. 2003; Kim et al. 2008; Kim et al. 2010a; Kim et al. 2010b; Kim et al. 2014).

Figure 10 shows the finite element discretization and the boundary conditions for two-dimensional plane stress nonlinear analyses of the hollow column specimens. The joints between the precast segments with a shear resistant connecting element were modeled using modified six-noded joint elements. In the joint model, the inelastic behavior of the joint elements is governed by normal and tangential stiffness coefficients. The interface elements between the precast concrete footing segment and the cast-in-place footings enhance the modeling of the effects of localized discontinuous deformation. 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.

Figure 10 also shows a method for transforming a circular section into rectangular strips when using plane stress elements. For rectangular sections, equivalent strips are calculated. After the internal forces are calculated, the equilibrium is checked. In this transformation of a hollow section to a rectangular section, a section with minimum

error was selected through iterative calculations concerning the moment of inertia for the section and area of concrete and reinforcements, to ensure that the behavior was similar to the actual behavior of bridge columns with hollow sections.

Geometric nonlinearity was included in the analysis. Geometric nonlinearity was considered for prestressing effects to be carried over from the initial step to the following steps and to take into account the P-delta effect from the axial force.

A comparison between the simulated and experimental load-drift values for the specimens is shown in Fig. 11. The value given by all specimens was similar to the analytical results; comparative data is summarized in Table 2. The proposed analytical model successfully predicts the load-drift relationship of hollow precast segmental prestressed concrete bridge columns with flexure failure.

4. Evaluation of design parameters

This section study mainly focuses on hollow precast segmental prestressed concrete bridge columns for which additional design considerations are encountered. The effects of the following parameters on the load-moment interaction diagram are considered: (1) concrete compressive strength, (2) prestressing reinforcement ratio, (3) effective prestress in the tendons, (4) diameter of circle passing through tendon centerlines-to-outside diameter of section ratio (D_s / D_o) and (5) inside diameter of section-to-outside diameter of section ratio (D_i / D_o).

4.1 Load-moment interaction diagram

Load-moment interaction diagrams presented here have been developed based on a design procedure that accommodates the KHBDC (2010) and AASHTO-LRFD (2014) code provisions for fully prestressed concrete compression members.

Section strength is accurately calculated by a strain compatibility procedure and an analytical representation of the nonlinear stress-strain relations of the prestressing tendons. A reduced resistance factor (AASHTO-LRFD 2014) is adopted for joints in segmental construction. Non-dimensional load-moment interaction diagrams for precast segmental prestressed concrete bridge columns of rectangular or circular cross sections have been also addressed.

This section summarizes the basic equations used in this study, and more details are provided in references (Naaman 2004; Nawy 2009).

The rectangular stress block at ultimate of a circular hollow section may act over either a segment of a circle or an annulus. Formulas for calculating the area of the effective compression zone and the distance of its geometric centroid from the center of the section are presented in Figure 12 (Naaman 2004). Figure 12 also illustrates the two possible cases for the compression block of a circular hollow section, depending on the location of the neutral axis at ultimate. The following results can be derived:

For case (a):

$$(1) \quad A_e = \frac{1}{2}(\theta - \sin \theta)r_o^2$$

$$(2) \quad \bar{x} = \frac{\sqrt{2}}{3} \frac{(1 - \cos \theta)^{1.5}}{\theta - \sin \theta} r_o$$

For case (b):

$$(3) \quad A_e = \frac{1}{2}(\theta_o - \sin \theta_o)r_o^2 - \frac{1}{2}(\theta_i - \sin \theta_i)r_i^2$$

$$(4) \quad \bar{x} = \frac{\sqrt{2}}{3} \frac{(1 - \cos \theta_o)^{1.5} \times r_o^3 - (1 - \cos \theta_i)^{1.5} \times r_i^3}{(\theta_o - \sin \theta_o)r_o^2 - (\theta_i - \sin \theta_i)r_i^2}$$

where A_e = effective compression area of the stress block; $\bar{x}$ = distance from the section centroid to the centroid of the compression area; r_o = outside radius of the section; r_i = inside radius of the section; and θ = angles in radians as defined in Figure 12.

Assuming the material and section properties are given, the strain in the concrete under effective prestress can be found using the following equation:

$$(5) \quad \varepsilon_{ce} = \frac{A_{ps}f_{pe}}{A_n E_c} = \frac{A_{ps}f_{pe}}{(A_g - A_{ps})E_c}$$

where ε_{ce} = uniform compressive strain in the concrete under effective prestress;

$A_{ps} = \sum_i (A_{ps})_i$ = area of prestressing steel; f_{pe} = effective prestress; A_n = nominal area

of concrete section; E_c = modulus of elasticity of concrete; and A_g = gross area of concrete section. It is assumed that all tendons have the same effective prestress.

At ultimate, the strain change in the i th steel layer becomes:

$$(6) \quad (\Delta \varepsilon_{ps})_i = \varepsilon_{ce} + \varepsilon_{cu} \left(\frac{d_i - c}{c} \right)$$

where ε_{ps} = strain in prestressing steel; ε_{cu} = ultimate compressive strain of concrete;

d_i = distance of prestressing steel layer i from outside edge of section; and c =

distance of neutral axis from outside edge of section.

The strain in the i th layer of steel is then given by:

$$(7) \quad (\varepsilon_{ps})_i = \varepsilon_{pe} + (\Delta\varepsilon_{ps})_i$$

where ε_{pe} = strain in prestressing steel under effective stress.

The stress in the i th layer of steel is calculated using the assumed stress-strain relationship for the prestressing bars. The tensile force in each layer is given by:

$$(8) \quad T_{ip} = (A_{ps})_i (f_{ps})_i$$

where T_{ip} = tensile force in prestressing steel layer i ; f_{ps} = stress in prestressing steel.

Assuming that the area occupied by the steel in the compression zone is negligible, the compressive force in the concrete is obtained from:

$$(9) \quad C = 0.85 f_{ck} A_e$$

where C = total effective compressive force; f_{ck} = uniaxial compressive strength of concrete; and A_e = effective compression area of concrete.

The nominal axial force and nominal moment resistances can be obtained by writing equilibrium equations for forces and moments:

$$(10) \quad P_n = C - \sum_i T_{ip}$$

where P_n = nominal axial load capacity of section.

For circular hollow section:

$$(11) \quad M_n = C\bar{x} + \sum_i T_{ip}(d_i - r_o)$$

where M_n = nominal moment capacity of section; and $\bar{x}$ = distance from the section centroid to the centroid of the area of the effective compression zone.

The corresponding eccentricity of the applied load can be computed using:

$$(12) \quad e = \frac{M_n}{P_n}$$

where e = eccentricity of applied load.

The above procedure can be repeated for various strain diagrams and leads to the determination of the load-moment interaction diagram. A flowchart outlining the basic operation of the developed P-M interaction diagram is presented in Fig. 13. This program was used first to evaluate the influence of various parameters on the load-moment interaction diagram and then to generate design charts for a practical range of those parameters found to have a non-negligible influence on the load-moment interaction diagram.

The P-M interaction diagram for hollow precast segmental prestressed concrete bridge column specimens in the previous section is shown in Fig. 14. The column strength is generally checked by comparing the applied axial load and the total moment with the axial-force moment capacity of the column section. For the normal strength columns, the analyses provide conservative column strengths compared with the test results. The design procedure suggested here should lead to a conservative yet economically attractive design

4.2 Effect of compressive strength

The concrete compressive strength varied from 30 to 80 MPa. The sequential increment of compressive strength was set to 30 MPa, 35 MPa, 40 MPa, 50 MPa, 60 MPa, 70 MPa and 80 MPa. The compressive strength of the concrete has a significant influence at high axial loads. However, this effect diminishes considerably at lower axial loads (see Fig. 15). Figures 15(a) and 15(b) show 141 and 106 percent increase, respectively, in nominal moment resistance in the pure flexure region of the load-moment interaction diagram by increasing the compressive strength from 30 to 80 MPa.

4.3 Effect of prestressing reinforcement ratio

The sequential increment of prestressing reinforcement ratio was set to 0.002, 0.004, 0.006, 0.008 and 0.010. The amount of prestressing reinforcement significantly affects strength. This effect is more pronounced in the low axial load region of the interaction diagrams (see Fig. 16). Figures 16(a) and 16(b) show 14 and 27 percent increase, respectively, in nominal moment resistance in the pure flexure region of the load-moment interaction diagram by increasing the prestressing reinforcement ratio from 0.002 to 0.010.

4.4 Effect of effective prestress

The effective prestress varied from 651 MPa to 1023 MPa. The sequential increment of effective prestress was set to 651 MPa, 842.3 MPa and 1023 MPa. The effective prestress in the tendons does not have a significant effect on the load-moment

interaction diagram, especially in the low axial load region (see Fig. 17).

4.5 Effect of D_s / D_o

The D_s / D_o varied from 0.85 to 0.75. The sequential decrement of D_s / D_o was set to 0.85, 0.80 and 0.75. The ratio of the diameter of the circle passing through the tendons' centerline to the section's outer diameter, D_s / D_o , generally does not have a significant effect on the strength (see Fig. 18).

4.6 Effect of D_i / D_o

The sequential increment of D_i / D_o was set to 0.35, 0.45, 0.55, 0.65 and 0.75. The ratio of the inside diameter to the outside diameter of the section, D_i / D_o , has a marked effect on the resulting strength (see Fig. 19). Figures 19(a) and 19(b) show 35 and 27 percent decrease, respectively, in nominal moment resistance in the pure flexure region of the load-moment interaction diagram by increasing the D_i / D_o ratio from 0.35 to 0.75.

5. Conclusions

This study investigated the use of hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element. The proposed segmental column system under investigation in this study is designed with the goal of achieving a degree of energy dissipation and ensuring ductility in a bonded system.

From the results of the experimental, analytical and parametric studies, the following

conclusions were reached.

- An experimental and analytical study was conducted to quantify performance measures and examine one aspect of detailing for an applied system. It was concluded that the design concepts and construction methods are promising solutions to the application of hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element.
- A key advantage of the proposed precast column system is its expected seismic performance. Hence, hollow precast segmental prestressed concrete bridge columns with a shear resistant connecting element will remain functional immediately after a moderate seismic event and will require minimal repair.
- Non-dimensional load-moment interaction diagrams were developed in this study to predict the nominal resistance of precast segmental prestressed concrete bridge columns. The results are presented in forms suitable for both analysis and design. For most applications, it would be advantageous to develop design charts concentrating around the pure flexural region of the load-moment interaction diagrams.
- The use of load-moment interaction diagrams as a design aid should provide a quick and economical solution for the design of precast segmental prestressed concrete bridge columns when a complete computer solution is not available.
- Several parameters and corresponding performance of hollow precast segmental prestressed concrete bridge columns have been studied in order to determine the suitable and optimum cross-sections. Additional parametric research is needed to

refine and confirm design details, especially for actual detailing employed in the field. Future work by the authors will include the formulation of a constitutive model for time dependent effects such as concrete creep, shrinkage and relaxation of prestressing tendons.

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List of Tables

Table 1. Properties of test specimens.

Table 2. Experiment and analysis results.

Draft

List of Figures

Fig. 1. Developed precast segmental prestressed concrete bridge columns.

Fig. 2. Applied precast segmental prestressed concrete bridge columns (Unit: mm).

Fig. 3. Details of specimen.

Fig. 4. Loading setup.

Fig. 5. Lateral load-drift relationship for specimen (a) Model (1), and (b) Model (2).

Fig. 6. Hysteretic energy dissipation.

Fig. 7. Construction of cracked concrete model (Kim et al. 2010a).

Fig. 8. Model for reinforcing bar in concrete (Kim et al. 2010a).

Fig. 9. Model for prestressing tendons (Kim et al. 2010a).

Fig. 10. Finite element mesh for specimen (a) transformation of a hollow column to an idealized equivalent rectangular column, and (b) finite element mesh for analysis.

Fig. 11. Comparison of results from the experimental results (a) Model (1), and (b) Model (2).

Fig. 12. Cases of effective compression zone at ultimate for circular hollow section (Naaman 2004).

Fig. 13. Flowchart of P-M interaction diagram.

Fig. 14. P-M interaction diagram for hollow precast segmental prestressed concrete bridge columns.

Fig. 15. Influence of concrete compressive strength on the P-M interaction diagram (a) KHBDC, and (b) AASHTO-LRFD.

Fig. 16. Influence of prestressing reinforcement ratio on the P-M interaction diagram (a) KHBDC, and (b) AASHTO-LRFD.

Fig. 17. Influence of effective prestress on the P-M interaction diagram (a) KHBDC, and (b) AASHTO-LRFD.

Fig. 18. Influence of D_s / D_o on the P-M interaction diagram (a) KHBDC, and (b) AASHTO-LRFD.

Fig. 19. Influence of D_i / D_o on the P-M interaction diagram (a) KHBDC, and (b) AASHTO-LRFD.

Table 1. Properties of test specimens.

Specimen		Prototype	Model
Diameter of cross section (mm)		2200 (1000)	800 (360)
Effective height (mm)		7522	2800
Prestressing steel	Material	Φ 15.2 mm seven-wire strands	Φ 15.2 mm seven-wire strands
	Yielding strength (MPa)	1860.0	1968.2
	Prestressing force (MPa)	842.3	842.3
	Material	D16	D10
Longitudinal reinforcement	Yielding strength (MPa)	400.0	544.6
	Reinforcement ratio (%)	0.2 (Minimum)	0.2 (Minimum)
	Material	D22	D13
Transverse reinforcement	Yielding strength (MPa)	400.0	476.3
	Volumetric ratio (%)	1.2	1.2
	Material	D32	D32
Footing reinforcement	Yielding strength (MPa)	400.0	463.6
	Material	STK490 (139.8/165.2 mm)	STK490 (60.5/76.3 mm)
Shear resistant connecting element	Yielding strength (MPa)	315	315
	Footing	40.0	48.1
Strength of concrete (MPa)	Precast footing segment	40.0	48.1
	Column	40.0	48.1

Table 2. Experiment and analysis results.

Specimen	Experiment		Analysis		Ratio of experimental and analytical results	
	$V_{\max}$	μ	$V_{\max}$	μ	$V_{\max}$	μ
Model (1)	568.1	5.3	591.9	5.0	0.96	1.06
Model (2)	577.0	5.8	592.1	5.5	0.97	1.05
Mean					0.97	1.05