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Ultimate axial load and moment interaction diagrams for prestressed HPC thin-walled short columns

H. Akbarzadeh Bengar[1,*], A. A. Maghsoudi[2]

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Abstract

In order to lighten the prestressed concrete solid members, nowadays, it is possible to make use of the advantage of HPC (f_c'> 60 MPa) as well as replacing the solid section with a PSC thin-walled section for certain members such as circular and box columns. Using the strength theory of ACI, a numerical procedure along with a computer program was developed for the analysis of such sections subjected to axial compression or tension load and bending moments. The program solves for all possible variables such as, concrete compressive strength (f_c'= 60-100 MPa), type of prestressed steel, concrete cover, ratio of wall thickness to the section dimensions and the PS steel arrangements to satisfy the given loading cases, thus leading to an optimal cost solution. However, since the cross section is thin-walled circular or box and the PS steel is located at discrete points along the periphery of a circle or rectangle, the equations of equilibrium are complex for hand computations (especially for circular section) but suitable for computer program. So, by use of MATLAB software the interaction diagrams were also drawn for the analysis of such sections for all mentioned variables. The use of prestressed thin-walled column diagrams is a safe and easy tool for the analysis of such columns. Finally, the accuracy of the proposed method is demonstrated by comparing its results to those of the available experimental values and is indicate that the proposed method predict very well the capacity of prestressed thin-walled column.

Keywords: Analysis, Prestressed HPC, Thin-walled columns, Interaction diagrams.

1. Introduction

With the rapid development of concrete technology in recent years, higher strength concrete can be produced much more easily than before. Since 1980, several investigations on mechanical properties and structural behavior of high strength concrete (HSC) have been reported [1-5]. Considering several advantages of HSC, nowadays, this type of concrete is widely accepted and is used in reinforced concrete (RC) structures. Current knowledge of HSC shows that there are definite advantages, both technical and economical in using HSC in structures. Greater strength per unit cost and per unit weight, increased modules of elasticity and reduced shrinkage and creep are some of these advantages. These factors are more important when calculating the losses of prestressed (PS) concrete forces.

Considering PS concrete, the concrete will resist a large amount of stresses and hence one way to reduce the stresses, is to increase the cross section area of the PS concrete members, which is not desired. Although, generally the PS structural concrete weights are lower than conventional RC structures, perhaps a better solution can be achieved while using the HSC or high performance concrete strength (HPC) in PS concrete elements. The research is required to understand the effect of concrete strength (i.e., strength more than what is recommended in current prestressed concrete codes) on PS concrete elements such as columns.

However, a negative structural effect occurs when using HSC or HPC in the PS columns as will be seen, can become more problematical since the vertical stiffness of the columns is reduced [6-7]. This arises due to the reduced cross-sectional areas which result from the use of HSC, and the functional relationship between strength and elastic modules. It is possible to over come this disadvantageous, by increasing the moment of inertia of the section, when for example using thin-walled column sections. The well designed thin-walled sections not only can resist the applied stresses but is also possible to ignore the buckling effect while the walls thickness are not narrow and therefore more loads can be applied. However, considering the flexural rigidity, EI, of slender PS concrete column at their

* *Corresponding Author: h.mazandaran@umz.ac.ir*
1 Assistant Professor of Civil Engineering at University of Mazandaran, Babolsar, Iran
2 Associate Professor of Civil Engineering at Kerman University, Kerman, Iran

ultimate capacity, a new method was proposed for computing the EI values by shuraim et al [7]. The proposed EI model was used with the moment magnification formula to obtain moment versus axial load interaction diagrams for a number of slender PS concrete columns. The diagrams were compared with those obtained from a finite element analysis and with available experimental test result and good agreement was observed. Comparison with current code formulations, ACI committee 318-1995 and PCI recommendations are made, and they proposed a new procedure [8] and [9].

Wiggins developed an approximate procedure for the design of cast-in-place reinforced concrete piers (for transmission tower foundations) subjected to bending and axial compression/tension [10]. He substituted circular cross sections with equivalent rectangular sections. The results obtained by his procedure are highly conservative and approximate. Moreadith presented recommendations for the design of reinforced concrete members subjected to combined bending and tension [11]. Gouwens and Cichy developed a computer program for determining the strength and stiffness of round columns [12]. They replaced the discrete reinforcing bars by a thin continuous ring of reinforcement. Hsu developed a computer program to study the three-dimensional strength interaction diagrams and failure surfaces for reinforced concrete members subjected to biaxial bending and tension, and proposed design formulas for square and rectangular cross sections [13]. Ehsani and Ross et al developed computer programs for the analysis and design of columns subjected to axial compression and uniaxial/biaxial bending [14] and [15]. The Precast and Prestressed Concrete Institute 1999, the Concrete Reinforcing Steel Institute 1984, and the American Concrete Institute published tables and charts for the analysis and/or design of reinforced concrete solid circular columns under combined bending and axial compression [16] and [17]. Xiao et al investigated the full scale experimental studied on high-strength concrete columns [18]. Gupta et al developed a computer program and a numerical procedure for analysis/design of RC circular cross section subjected to axial loads and bending moments [19]. Design and behavior of thin-walls in hollow concrete bridge piers and pylons was studied by Taylore et al [20]. The behavior of thin-walled segmentally constructed post-tensioned bridge piers was reported by Rowell [21]. The behavior of thin-walled concrete box piers were investigated experimentally and analytically and reported by Taylor et al [22]. The use of partially prestressed reinforced concrete columns to reduce post-earthquake residual displacement of bridges was investigated by Mahin et al [23]. The partially and fully prestressed concrete sections under biaxial bending and axial load were studied by Rodriguez-Gutierrez et al [24]. Lateral load tests on prestressed concrete piles supporting integral abutments was investigated and reported by Burdette et al [25]. The behavior of precast prestressed concrete pile to cast-in-place pile-cap connection was reported by Harries et al [26]. Experimental studies on precast prestressed concrete pile to cast-in-place concrete pile-cap connections was reported by Xiao [27].

Application of basic principles of equilibrium and compatibility offers the most direct approach for analyzing and designing reinforced concrete members subjected to an axial force and a bending moment, using strength theory. However, since the column cross section of this study are thin-walled circular and box shapes and the PS steel is located at discrete points along the periphery of a circle and rectangular, the equations of equilibrium are complex and no explicit solutions can be determined. It is therefore necessary to use trial-and-error technique which becomes too lengthy for hand computation, but which can be efficiently solved using a computer.

The analysis of PS thin-walled sections especially circular columns is quiet tedious while compare either to the PS or RC solid columns, it is mainly because the time assumed to determine the properties and neutral axis of the sections. Imagine the work involved in a design situation where various sizes, shape of cross-section, concrete strength, steel percentages and wall thickness need to be considered. Hence, the simple and easy method of using interaction diagrams to analysis of RC columns is well established and such diagrams are available for solid reinforced normal concrete columns but no theoretical and such diagrams were observed for thin-walled PS concrete circular and box columns made of HPC.

This paper briefly reviews the analysis of PS concrete thin-walled circular and box columns with particular emphasis given to the buckling effect made of HPC, a concrete strength more than 60 MPa. It is reminded that, in available PS concrete codes and standards, the permitted range of concrete strength is 30-60 MPa, and hence a revise is necessary while using HPC in PS concrete.

2. The prestressed concepts and materials used

Prestressed concrete is defined as: the concrete in which there have been introduced internal stresses of such magnitude and distribution that the stresses resulting from given external loadings are counteracted to a desires degree. Different concepts may be applied to explain and analyze the basic behavior of the prestressed concrete [28, 29]. However, the concept-prestressing for combination of high-strength steel and concrete is shown more similarity between prestressed and reinforced concrete. This concept is to consider prestressed concrete as a combination on the steel and concrete, similar to RC, whit steel taking tension and concrete taking compression so that the two materials form a resisting couple against the external moment. As mentioned earlier, to produce, the PS concrete elements, the use of high strength steel and concrete is necessary. For the purpose of this paper high performance concrete is defined as concrete with a characteristic compressive strength more than 60 MPa. However, such strength is the upper limit of concrete strength permitted in the available PS codes. Hence the validity of PS codes needs to be reviewed for the new definition of HPC.

Considering the advantageous of HPC in PS columns, it is possible to reduce not only the weight of the structures but also their performance and durability will be increased. Therefore, the stress-strain diagrams for HPC, and tensile prestressed steels which are differ from the normal RC is assumed, and briefly the analysis of thin-walled circular and box PS high performance concrete columns discussed and then based on the latest version of ACI standard a computer program is generated (see Figures 1 and 2) to analyze such columns shape, and their relative columns interaction diagrams are plotted and presented using MATLAB software [30, 31].

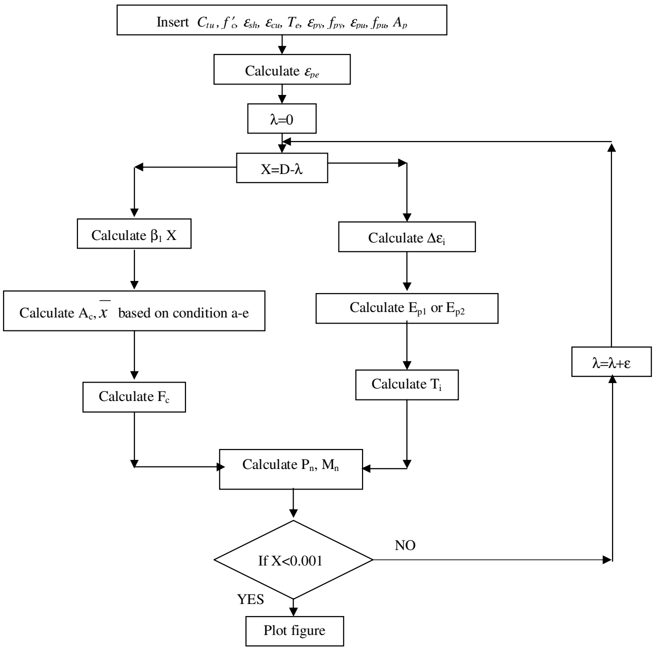


Fig. 1. Flow chart for the computer program for interaction diagram of PS thin-walled circular columns

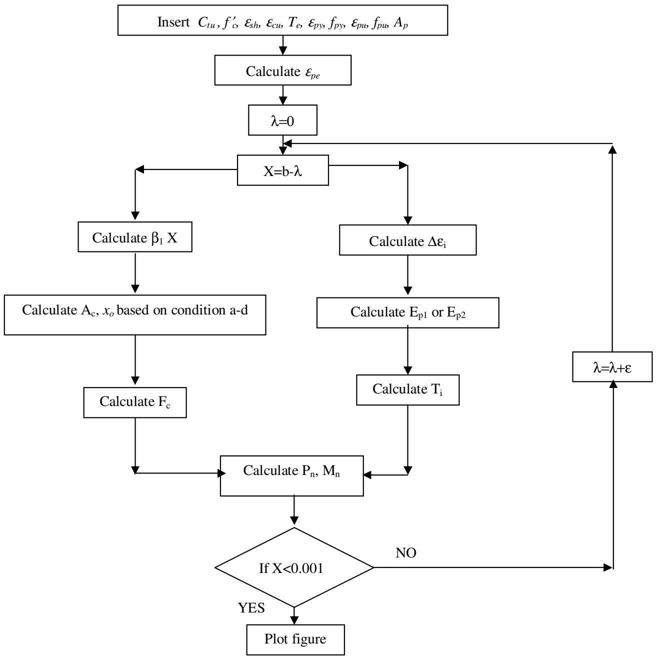


Fig. 2. Flow chart for the computer program for interaction diagram PS thin-walled box columns

2.1. steels prestressing

High-tensile steel for prestressing usually takes one of these forms: wires, strands or bars. While the ultimate strength of high-tensile steel can be easily determined by testing, its elastic limit or its yield point can not be so simply ascertained, since it has neither a yield point nor a definite proportional limit. The selected steel types used for PS are the three types (i.e., wires, strands and bars) conform to ASTM Specification A-416.

For the purpose of analyzing and plotting of the PS concrete columns interaction diagrams and for every load increments, the bi-linear steel stress-strain curves shown in Figure 3 was assumed for all the three types of PS steels.

Where

f_{pu} = ultimate stress in prestressed steel ; f_{py} = yield tensile stress in prestressed steel

ε_{pu} = ultimate strain in prestressed steel ; ε_{py} = yield tensile strain in prestressed steel

For analysing and design purpose of prestressed HPC thin-walled columns the assumed bi-linear PS steel stress-strain diagram shown in Figure 3, was used and the program was testified with the worked example given in Nilson, for square prestressed columns section [32]. For simplicity the worked example is reproduced here, and the comparison of the results is shown in Table 1. It is clear that, a much closed agreement is achieved.

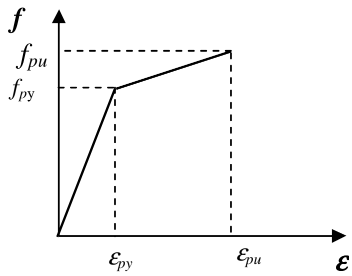


Fig. 3. Prestressed steel stress-strain curves

Table 1. Comparison of worked example and proposed method [32]

	X (cm), mm	(5),50	(10),100	(15),150	(20),200	(25),250	(30),300	∞
P_n (t) kN	Nilson[32]	-16.25 -162.5	34.65 346.5	76.07 760.7	119.15 1191.5	163.08 1630.8	205.76 2057.6	298.66 2986.6
	Proposed method	-14.77 -147.7	32.87 328.7	75.52 755.2	119.15 1191.5	163.08 1630.8	205.76 2057.6	298.66 2986.6
M_n (t.m) kN.mm	Nilson[32]	6.13 61.3	10.78 107.8	12.64 126.4	13.30 133.0	12.30 123.0	9.87 98.7	0.0 0.0
	Proposed method	5.98 59.8	10.26 102.6	12.69 126.9	13.30 133.0	12.30 123.0	9.87 98.7	0.0 0.0

X is the neutral axis depth

b=300 mm

prestressed steel= 8 cable 3/8 in., (9.53 mm)

f_{pu}=17500 kg/cm^2, (1750 N/mm^2)

E_p= 2.03×106 kg/cm^2, (2.03×105 N/mm^2)

f_{pe}= 10500 kg/cm^2, (1050 N/mm^2)

ε_{pe}= 0.0052

f'_c= 420 kg/cm^2, (42 N/mm^2)

2.2. Applied assumption in computer program

Based on the equilibrium of forces, strains compatibility and strength reduction coefficients using equivalent rectangular stress block ACI for different variables such as, concrete compressive strength (f_c'= 60-100 MPa), type of prestressed steel, concrete cover, ratio of wall thickness to the section dimensions and the PS steel arrangements, a computer program was developed that can analyse a PS thin-walled circular and box concrete cross section subjected to axial compression/tension combined with a moment. The columns interaction diagrams are also plotted by the MATLAB software for the analysis of prestressed HPC thin-walled circular and box columns (see Figure 1 and 2). The analysis is based on the following assumptions:

1. The relationship between concrete compressive stress distribution and concrete strain is assumed to be rectangular, with the equivalent rectangular stress block satisfying the requirements of ACI for HSC.
2. Tensile strength of concrete is negligible.
3. Strain in PS steel and concrete is assumed directly proportional to the distance from neutral axis.
4. Maximum strain in extreme concrete compression fiber for HSC is 0.003 [33].
5. The bi-linear stress-strain curve for PS steel is assumed.
6. The amount of jacking force is the minimum values ($0.94f_{py}$, $0.85f_{pu}$) suggested by the ACI-05.
7. Total prestressed losses to reach the effective prestresseing during the service state assumed to be 20%.

The procedure can be employed with any grade of concrete strength (f_c'= 60-100 MPa), or any three PS steel type. Thus, the solution obtained is exact enough, for the assumptions made. However, it is reminded that an exact calculation for finding the prestressed losses is not possible as this can be only estimated [28, 32].

To analysis the members, an equivalent rectangular stress block coefficients proposed by ACI code was used for compression zone (Figures 4-b and 5-b).

$$\alpha = 0.85 \tag{1}$$

$$\beta_1 = 1.09 - 0.008 f_c' \quad MPa \quad 0.65 \leq \beta_1 \leq 0.85 \tag{2}$$

$$K_3 = 0.6 + 10 / f_c' \quad MPa \quad K_3 \leq 0.85 \tag{3}$$

Where, factor K_3 is used to consider the different between the column actual concrete compressive strength and the standard cylinder strength.

3. Analysis of prestressed hpc thin-walled section

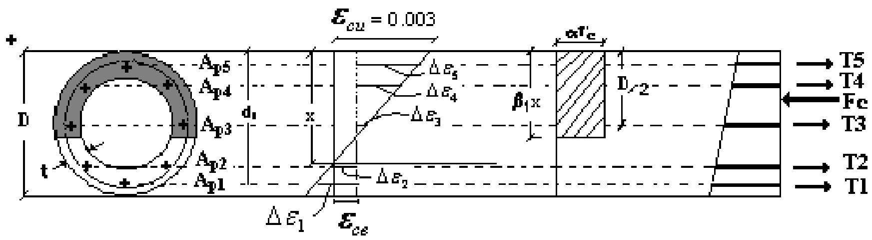
It is seldom that a prestressed concrete member is utilized to stand compression and is prestressed for compression's sake. Evidently, concrete can carry compressive load better without being precompressed by steel. And it is difficult to conceive of steel wires as addiding any appreciable strength to a member carrying axial compression. In other words, some compression members like most of the structural columns are actually flexural members, and all the advantageous of prestressing a beam would apply to the prestressing of those members.

For analyses purpose, assuming that, cross section shape and PS steel arrangements are symmetry, the strain in the section due to effective prestressed force can be obtained which has a uniform strain distribution (Figure 4 and 5-b) by;

$$\varepsilon_{ce} = \frac{T_e}{E_c A_g} + \varepsilon_{sh} + \varepsilon_{cc} \tag{4}$$

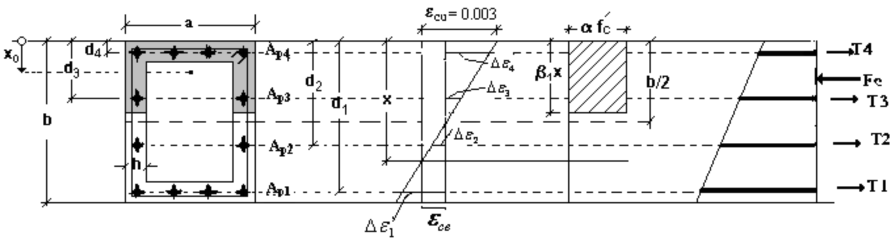
$$\varepsilon_{cc} = C_{tu} \frac{T_e}{E_c} \tag{5}$$

Where, T_e is the effective PS force at service, E_C is concrete



a) Cross section b) Strain distribution and stress block c) Force distribution

Fig. 4. Thin-walled circular sections



a) Cross section b) Strain distribution and stress block c) Force distribution

Fig. 5. Thin-walled box sections

modulus of elasticity ($E_c = 3320\sqrt{f_c'} + 6900 \quad MPa$), A_g is gross cross sectional area of column, ε_{sh} is the concrete shrinkage strain (for HPC a typical value of 500×10^{-6} can be assumed), ε_{cc} is the concrete creep strain, and C_{tu} is the ultimate creep coefficient [30].

The effective prestressed steel strain is given by;

$$\varepsilon_{pe} = \frac{f_{pe}}{E_p} = \frac{T_e}{A_p E_p} \tag{6}$$

Where, E_p is the modulus of elasticity of PS steel and A_P is area of PS steel.

An arbitrary value for neutral axis depth, X, can be selected (theoretically for pure compression load, it can be reached infinity however, for calculating purpose, the maximum value of depth of concrete compression stress block, a, is assumed to be same as the external circular column diameter, D or section height of box, b), and then the strain variation for each PS steel row (A_{pi}) while the member is loaded towards ultimate is obtained by;

$$\Delta\varepsilon_i = \varepsilon_{ce} - \varepsilon_{cu}\frac{X - d_i}{X} \tag{7}$$

Now, based on net strain, it is possible to compute the tensile force in each PS steel row;

$$T_i = A_{pi} f_{pi} = A_{pi}(E_{p1}\, or E_{p2})\varepsilon_{spi} = A_{pi}(E_{p1}\, or E_{p2})(\varepsilon_{pe} + \Delta\varepsilon_i) \tag{8}$$

A negative and positive sign convention is used for compression/tension force respectively. The PS steel rows are numbered from bottom to top of section (i.e., row 1 is for the farthest distance of extreme concrete compressive fiber (Figures 4 and 5)).

Where, X is the neutral axis depth from the extreme concrete compressive fiber, ε_{spi} is the PS steel strain in i^{th} row, and d_i is the i^{th} distance of PS steel row from the extreme concrete compressive fiber.

To draw thin-walled circular and box columns interaction diagrams for failure cases shown in Figure 6, a value of X equal to the column circular diameter or section height for box is assumed and in each step the value of neutral axis depth is decreased until it is become to zero.

3.1 Solution method

The following equations are based on the equilibrium of forces and compatibility of strains. The pure (obsolete) axial load strength of PS cross section P_o is equal to the sum of compression strength of concrete F_c and the axial load strength of PS steel T_i. Thus, the pure compression column capacity at point A, in Figure 6 can be calculated as;

$$P_0 = F_c + T_i \tag{9}$$

or

$$P_0 = K_3 f_c'(A_g - A_p) + A_p E_{pl}(\varepsilon_{pe} - 0.003 + \varepsilon_{ce}) \tag{10}$$

Where, A_p and A_g are total PS steel area and the gross cross sectional area of column respectively.

3.1.1 Thin-walled circular column

The different positions of $\bar{x}$ in PS circular thin-walled HPC column of points A to B in Figure 6, are shown in Figure 7. Thus, using the equivalent rectangular stress block, the values of compressive concrete area , A_c and the resultant concrete compressive force, F_c are given as;

a) If $\beta_1 x \leq t$ (Figure 7-a):

$$A_c = D^2(\theta - \sin\theta\cos\theta)/4 \tag{11}$$

$$\bar{x} = \frac{D\sin^3\theta}{3(\theta - \sin\theta\cos\theta)} \tag{12}$$

$$\theta = Arc\cos\left(1 - \frac{2\beta_1 x}{D}\right) \tag{13}$$

b) If $t < \beta_1 x \leq D/2$ (Figure 7-b):

$$A_c = \frac{D^2}{4}(\theta_1 - \sin\theta_1\cos\theta_1) - \frac{(D-2t)^2}{4}(\theta_2 - \sin\theta_2\cos\theta_2) \tag{14}$$

$$\bar{x} = \frac{D^3\sin^3\theta_1 - (D-2t)^3\sin^3\theta_2}{3[D^2(\theta_1 - \sin\theta_1\cos\theta_1) - (D-2t)^2(\theta_2 - \sin\theta_2\cos\theta_2)]} \tag{15}$$

$$\theta_1 = Arc\cos\left(1 - \frac{2\beta_1 x}{D}\right) \tag{16}$$

$$\theta_2 = Arc\cos\left(\frac{D/2 - \beta_1 x}{D/2 - t}\right) \tag{17}$$

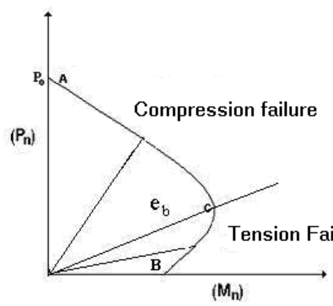


Fig. 6. A typical column interaction diagrams indicating the failure types

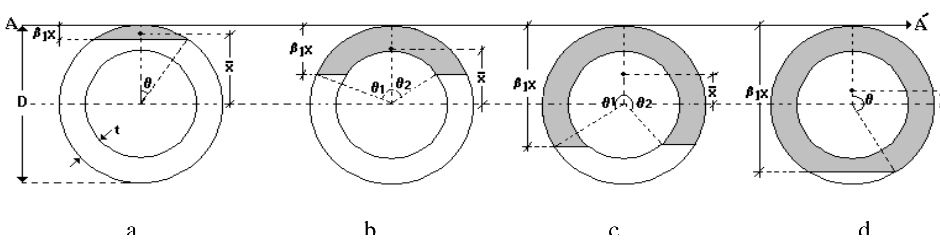


Fig. 7. Different positions of $\bar{x}$ in PS thin-walled circular column

c) If $D/2 < \beta_1 x \leq D - t$ (Figure 7-c):

$$A_c = \frac{D^2}{4}(\theta_1 - \sin\theta_1 \cos\theta_1) - \frac{(D-2t)^2}{4}(\theta_2 - \sin\theta_2 \cos\theta_2) \quad (18)$$

$$\bar{x} = \frac{D^3 \sin^3\theta_1 - (D-2t)^3 \sin^3\theta_2}{3[D^2(\theta_1 - \sin\theta_1 \cos\theta_1) - (D-2t)^2(\theta_2 - \sin\theta_2 co\theta_2)]} \quad (19)$$

$$\theta_1 = \pi/2 + Arc\sin(\frac{2\beta_1 x}{D} - 1) \quad (20)$$

$$\theta_2 = \pi/2 + Arc\sin(\frac{\beta_1 x - \frac{D}{2}}{\frac{D}{2} - t}) \quad (21)$$

d) If $D - t < \beta_1 x \leq D$ (Figure 7-d):

$$A_c = \frac{D^2}{4}(\theta - \sin\theta \cos\theta) - \frac{\pi(D-2t)^2}{4} \quad (22)$$

$$\bar{x} = \frac{D^3 \sin^3\theta}{3[D^2(\theta - \sin\theta \cos\theta) - \pi(D-2t)^2]} \quad (23)$$

$$\theta = \pi/2 + Arc\sin(\frac{2\beta_1 x}{D} - 1) \quad (24)$$

e) If $D < \beta_1 x$

$$A_c = \pi(tD - t^2) \quad (25)$$

$$\bar{x} = 0 \quad (26)$$

3.1.2 Thin walled box column

The different positions of $\bar{x}$ in PS box thin-walled HPC column of points A to B in Figure 6, are shown in Figure 5. Thus, using the equivalent rectangular stress block, the values of compressive concrete area , A_c and the resultant concrete compressive force, F_c are given as;

a) If $\beta_1 x \leq h$:

$$A_c = a\beta_1 x \quad (27)$$

$$\beta_1 x / 2 \; x_0 = \quad (28)$$

a: section width

b) If $h \leq \beta_1 x \leq b - h$:

$$A_c = ah + 2h\beta_1 x - 2h^2 \quad (29)$$

$$x_0 = \frac{a(\beta_1 x)^2 - (a-2h)(\beta_1 x - h)(h + \beta_1 x)}{2((\beta_1 xa) - (a-2h)(\beta_1 x - h))} \quad (30)$$

c) If $b - h \leq \beta_1 x \leq b$:

$$A_c = 2ah + 2bh + a\beta_1 x - 4h^2 - ab \quad (31)$$

$$x_0 = \frac{a(\beta_1 x)^2 - (b-2h)(a-2h)b}{2(\beta_1 xa - (b-2h)(a-2h))} \quad (32)$$

$$A_c = 2ah + 2bh - 4h^2 \quad (33)$$

$$x_0 = b/2 \quad (34)$$

Considering the equilibrium of effective PS forces about the centroidal area of the section, it is possible to calculate the nominal axial load (P_n) as;

$$F_c = \alpha . f_c' \, A_c \quad (35)$$

$$p_n = F_c - \sum_{i=1}^{n} T_i \quad (36)$$

Similarly, the nominal flexural moment M_n is equal to the sum of the nominal flexural moment due to concrete M_c and the nominal flexural moment due to PS steel M_{ps}.

$$M_n = M_c + M_{ps} \quad (37)$$

For circular thin-walled column:

$$M_n = F_c \bar{x} + \sum_{i=1}^{n} T_{si}(D/2 - d_i) \quad (38)$$

For box thin-walled column:

$$M_n = F_c(\frac{b}{2} - x_0) + \sum_{i=1}^{n} T_{si}(b/2 - d_i) \quad (39)$$

4. Interaction diagrams for anlyzing the prestressed hpc thin-walled circuiar and box coulmns

Based on the above mentioned equations and variables, a computer program was developed for drawing interaction diagrams by MATLAB software for analyzing prestressed HPC thin-walled circular and box columns. Figure 1 and 2 show flow chart of a computer program for this purpose. Some typical interaction diagrams of both sections are shown in Figures 8-10 and 11-13 respectively. In Table 2 and 3 for different type of PS steels, the maximum allowable PS steel amount, so that the column section is not located in the tensile zone of the interaction curves, and the maximum resisting force and moment (for ρ= 1%) are given. As an example, if the used steel amount in column section is more than allowable amount, the column design is controlled by tension zone.

For instance, if a circular HPC column section includes 8 stranded cables of type Grade 270 with diameter of 15.24 mm, f_c'=70 MP_a, γ=0.8 and t/D=0.2, the maximum allowable PS steel amount which can be used is 4%. Also for this column with maximum allowable PS steel amount of 4%; the maximum axial load and bending moment are 17.28, 3.19 MPa respectively (to coincide with the RC interaction diagrams the vertical and horizontal axis of PS interaction diagrams are drawn in terms of stress).

5. Comparison with test result

To testify the proposed method, available experimental data on prestressed thin-walled column is selected. The test specimen was contracted at the large scale structures laboratory at the University of Nevada, Reno [34]. The details

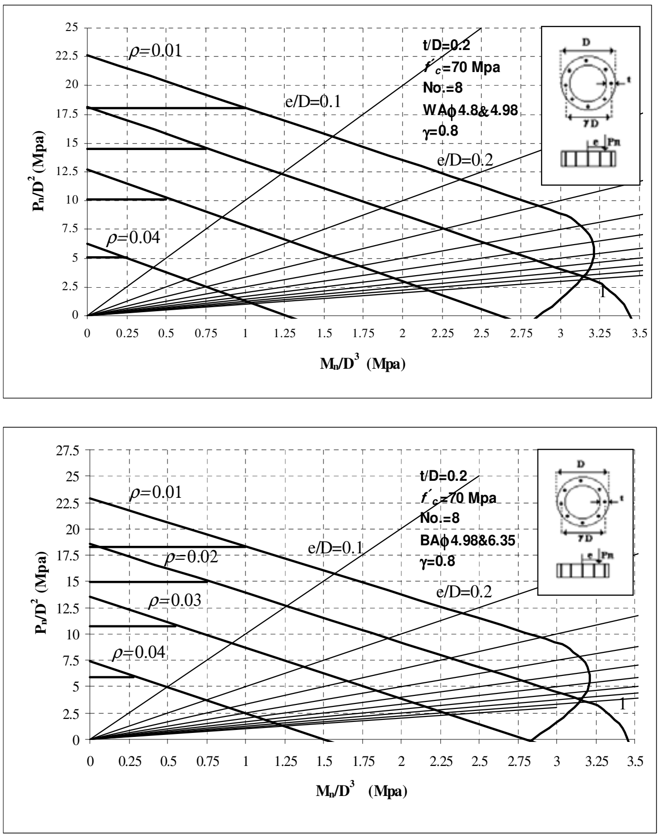


Fig. 8. Interaction diagrams for analysis of PS thin-walled circular column with rounded wires

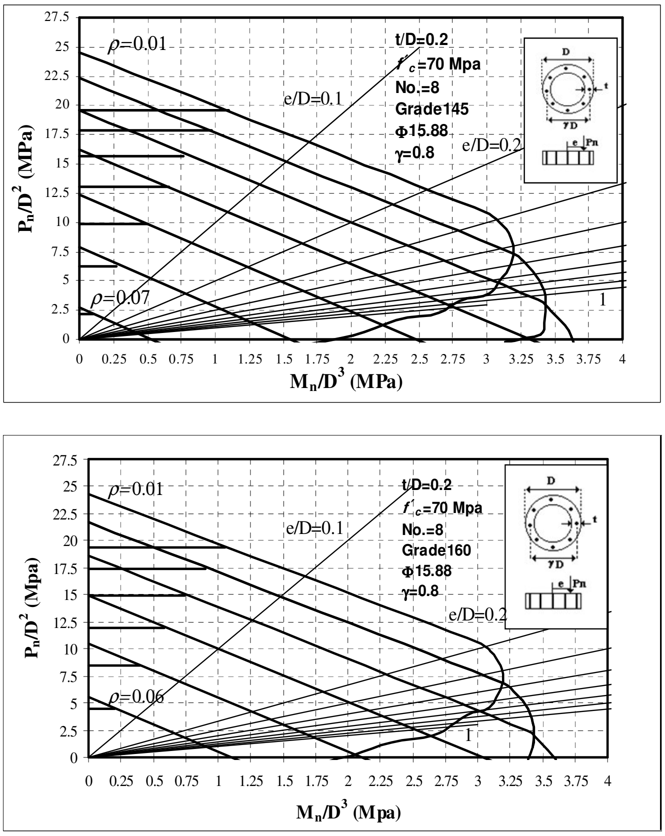


Fig. 10. Interaction diagrams for analysis of PS thin-walled circular column with Alloy Steel Bars

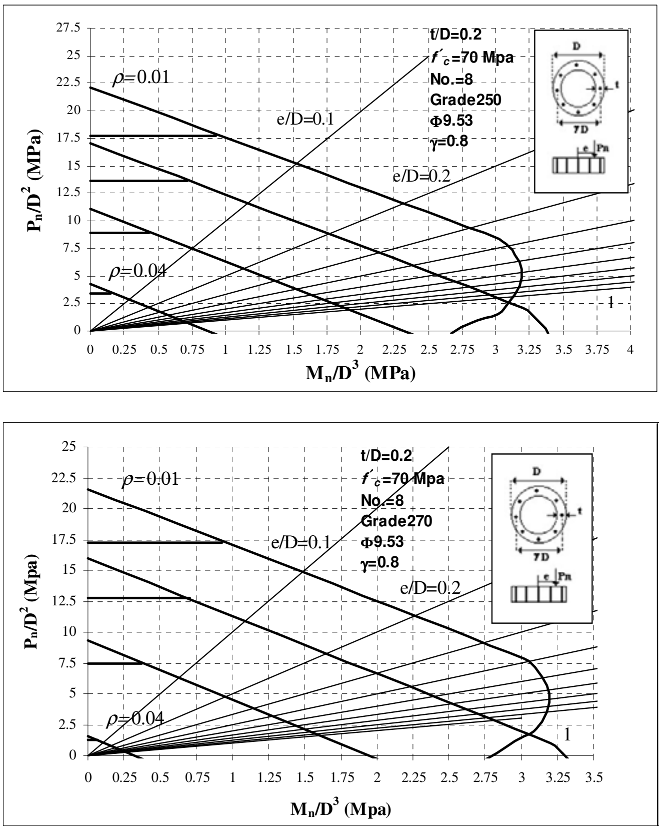


Fig. 9. Interaction diagrams for analysis of PS thin-walled circular column with stranded cables

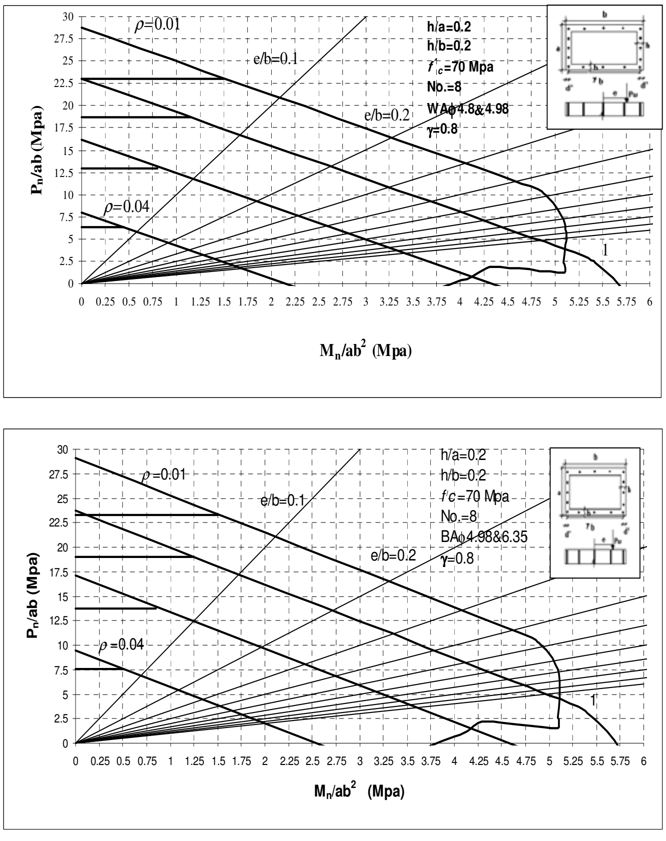


Fig. 11. Interaction diagrams for analysis of PS thin-walled box column with rounded wires

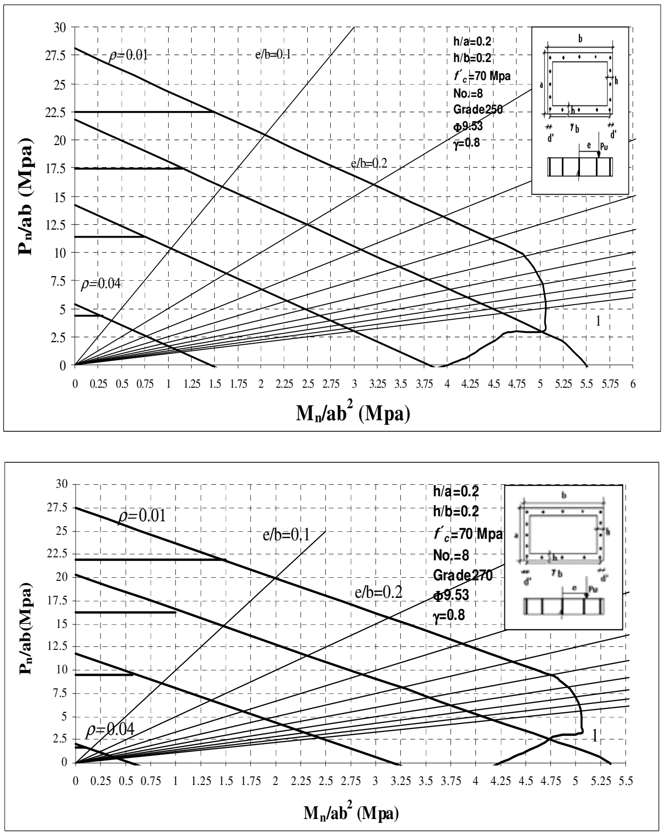


Fig. 12. Interaction diagrams for analysis of PS thin-walled box column with stranded cables

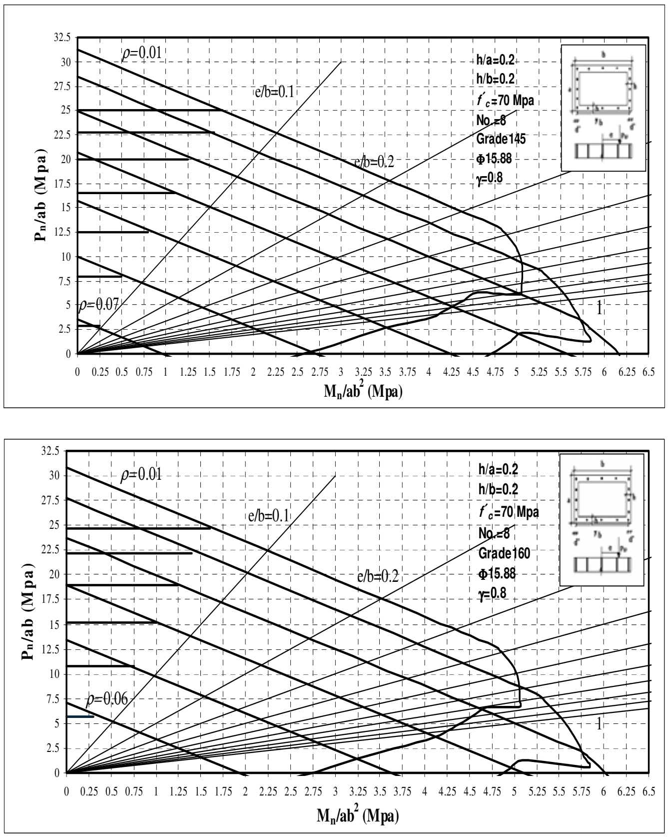


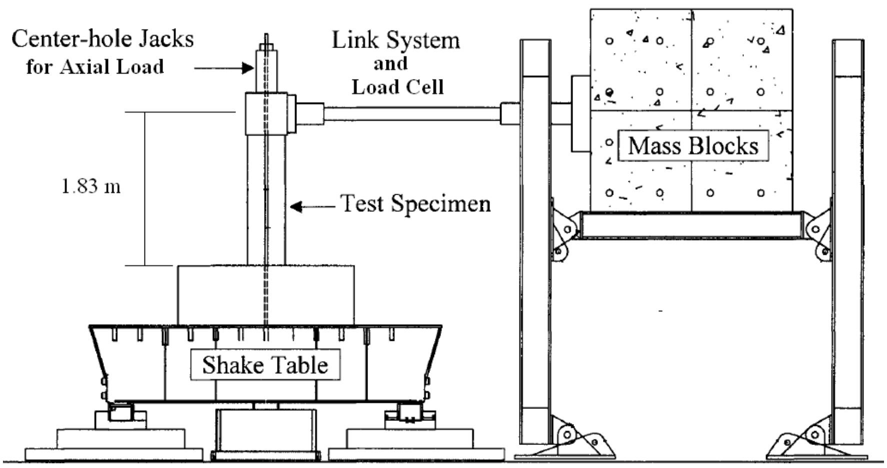
Fig. 13. Interaction diagrams for analysis of PS thin-walled box column with Alloy Steel Bars

Table 2. Maximum allowable amount of PS steel (for ρ =1%) for different PS steel types (thin-walled circular column)

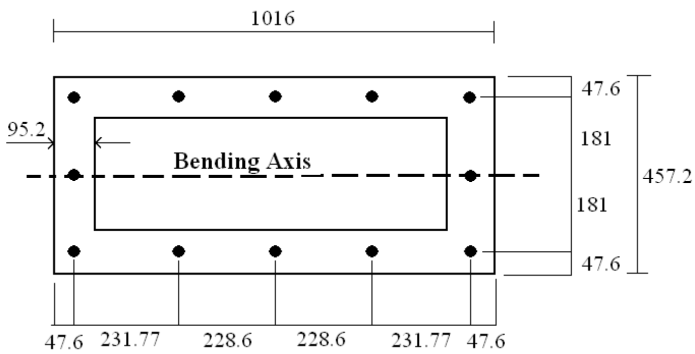
f'_c (MPa)	Type of prestressed steel	γ	t/D	Number of steel	Maximum allowable amount of steel	$P_{n,max}/D^2$ (ρ=1%)	$M_{n,max}/D^3$ (ρ=1%)
						Rounded Wires	
70	WAϕ4.88 &4.98	0.8	0.2	8	4	18.1	3.21
70	WAϕ6.35	0.8	0.2	8	4	18.28	3.21
70	WAϕ7.01	0.8	0.2	8	4	18.37	3.21
70	BAϕ4.98 &6.35	0.8	0.2	8	4	18.29	3.21
						Stranded Cables	
70	Grade250 ϕ6.35	0.8	0.2	8	4	17.62	3.19
70	Grade250 ϕ9.53	0.8	0.2	8	4	17.67	3.19
70	Grade250 ϕ15.24	0.8	0.2	8	4	17.67	3.19
70	Grade270 ϕ9.53	0.8	0.2	8	4	17.24	3.19
70	Grade270 ϕ15.24	0.8	0.2	8	4	17.28	3.19
						Alloy Steel Bars	
70	Grade145 ϕ15.88	0.8	0.2	8	7	19.62	3.19
70	Grade145 ϕ34.93	0.8	0.2	8	7	19.63	3.19
70	Grade160 ϕ15.88	0.8	0.2	8	6	19.40	3.19
70	Grade160 ϕ34.93	0.8	0.2	8	6	19.40	3.19

Table 3. Maximum allowable amount of PS steel (for ρ=1%) for different PS steel types (thin-walled box column

f'_c (MPa)	Type of prestressed steel	γ	h/a And h/b	Number of steel	Maximum allowable amount of steel	$P_{n,max}/D^2$ (ρ=1%)	$M_{n,max}/D^3$ (ρ=1%)
							Rounded Wires
70	WAϕ4.88 &4.98	0.8	0.2	8	4	23.04	5.11
70	WAϕ6.35	0.8	0.2	8	4	23.27	5.11
70	WAϕ7.01	0.8	0.2	8	4	23.39	5.11
70	BAϕ4.98 &6.35	0.8	0.2	8	4	23.27	5.11
							Stranded Cables
70	Grade250 ϕ6.35	0.8	0.2	8	4	22.44	5.06
70	Grade250 ϕ9.53	0.8	0.2	8	4	22.50	5.06
70	Grade250 ϕ15.24	0.8	0.2	8	4	22.50	5.06
70	Grade270 ϕ9.53	0.8	0.2	8	4	21.96	5.06
70	Grade270 ϕ15.24	0.8	0.2	8	4	22.00	5.06
							Alloy Steel Bars
70	Grade145 ϕ15.88	0.8	0.2	8	7	24.98	5.06
70	Grade145 ϕ34.93	0.8	0.2	8	7	24.99	5.06
70	Grade160 ϕ15.88	0.8	0.2	8	6	24.7	5.06
70	Grade160 ϕ34.93	0.8	0.2	8	6	24.70	5.06



a- Schematic diagram of the test setup



b- Cross section of column

Fig. 14. Test setup and specimen details [34]

Table 4. Property of concrete and prestressed steel [34]

f'_c (MPa)	Prestressed Steel					
	Type	Area (mm^2)	Number of steel	f_{pu} (N/mm^2)	f_{py} (N/mm^2)	f_{pe} (N/mm^2)
35	Cable	144	12	1860	1614	1100

of tested specimen are summarized in Figure 14 and Table 4. This column is subjected to axial load before test. Amount of axial load is 445 kN. The shake table was conducted with the Kobe earthquake motion recorded at the Kobe Oceanic Meteorological Observatory. The shake table testing consisted of fifteen runs, where the amplitude of acceleration from the Kobe motion was increased until failure. The envelope of the hysteretic curve for all runs is shown in Figure 15. The Figure is shown lateral displacement and lateral force and the amount of lateral load and lateral displacement are 341 kN and 81 mm respectively at peak load. Therefore, maximum bending moment was obtained 660 kN.m. Amount of eccentricity for this moment and axial load is equal 1.483 m. Analytical interaction diagram of moment-axial load for tested specimen based on proposed method was obtained and is shown in Figure 16. Also, the capacity of experimental axial load and bending moment for tested specimen is shown in Figure 16 too. Based on eccentricity of tested specimen (1.483 m), analytical capacity of axial load and bending moment are obtained 410 kN and 615 kN.m respectively, that amount of error is 7 percent. Proposed method predicts capacity of prestressed thin walled column conservatively. Therefore, the comparison of experimental and analytical result indicate that, the proposed method predict very well the capacity of prestressed thin-walled column.

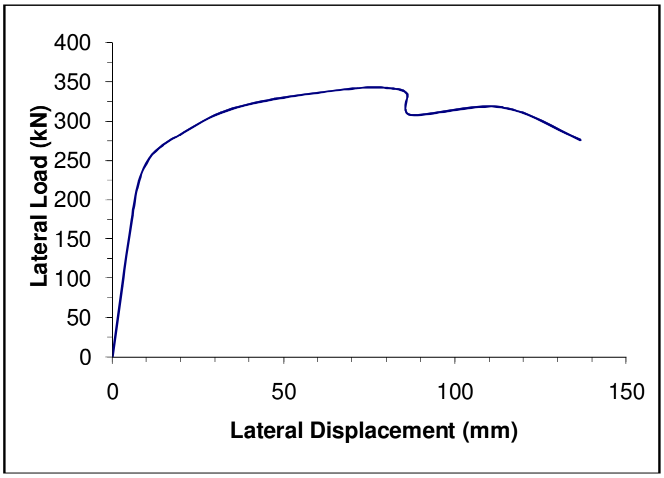


Fig. 15. Envelope of the hysteresis [34]

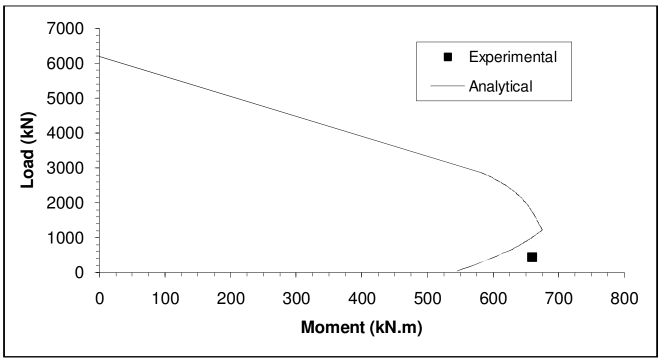


Fig. 16. Comparison of analytical and experimental result

5.Conclusions

i) In this study first; equations which are related to the analysis of prestressed HPC thin-walled circular and box columns were obtained and then the column interaction diagrams for different variable were drawn.

ii) For different type of prestressed steels, maximum allowable PS steel amount were obtained for HPC thin-walled circular and box columns. If the amount of PS steel used in column section is more than allowable amount, the column will be located in tension control zone.

iii) For circular and box thin-walled column; maximum allowable PS steel amount for rounded wires of type BA, for rounded wires of type WA, for stranded cables of type Grade250 with diameter 9.53mm, for stranded cables of type Grade270 with diameter 9.53mm, for Alloy Steel Bars of type Grade145 with diameter 15.88mm and for Alloy Steel Bars of type Grade160 with diameter 15.88mm were obtained 4%, 4%, 4%, 4%, 7% and 6% respectively.

iv) The use of such diagrams is a safe and easy tool for the analysis of prestressed HPC thin-walled circular and box columns.

v) The comparison of experimental and analytical result indicate that, the proposed method predict very well the capacity of prestressed thin-walled column.

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