Seismic Analysis of Partial Concrete-Filled Steel Tubular Columns (CFST)

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Abstract: Steel tubular columns are used in bridge piers in Iraq when necessary and when site conditions limited the work using ordinary R.C. columns. Al- Tala'a overpass in Baghdad is one of these cases. Since local buckling will occur in the lower part of such columns, it will be necessary to strengthen it through filling by concrete. Thin-walled concrete partially filled steel tubular (CFST) columns may undergo local and global buckling when subjected to seismic loading. The local buckling effects of steel tube walls have not been widely considered in existing numerical models for CFST columns. This work presents a development of a numerical dimensional model for the nonlinear inelastic analysis of thin-walled circular CFST columns under earthquake incorporating the effects of local buckling, and concrete filling level. The numerical model using SAP 2000 V.17 is used to check and calculate the local buckling location and the value of first-buckling load and failure load for five cases of concrete filling (namely 0%, 25%, 30%, 50% and 100%) of column height used in columns bridge piers of Al- Tala'a overpass under the general loading of bridges with an actual earthquake loading took place in Iraq, Ali Al-Gharbee 2014. The strength and ductility of these columns are also calculated and discussed with the deformation associated with these local buckling cases. It is concluded that filling the tubular columns of Al- Tala'a overpass to 30% of its length will remove the local buckling taking place at 200mm height and increase columns capacity by (36%). A final outcome from this work, the concrete partially filled steel tubular columns with (30-50) % filling ratios can be used effectively in bridge piers under seismic loadings instead of the completely filled columns with the same efficiency and capacity.

Keywords: Partially Concrete-Filled Steel tubular Columns, CFST, Earthquakes, SAP2000.

1. Introduction

Techniques and stages used to evaluate the effect of applying actual earthquake happened in Iraq and reflecting it on an overpass in Baghdad having piers with steel tubular columns will be studied. The cross-section of columns used in bridge piers greatly differs from those of buildings, as shown in Figure (1). The main characteristics of the crosssection of bridge pier columns compared with those in buildings are:

- 1)The cross-section is large (about 0.8- 2 m in diameter in bridge piers compared with 0.5-1 m in tall buildings).
- 2)Sometimes Stiffened plates are used.
- 3)Ratio of applied axial compressive load to the squash load (the ultimate strength in compression without bending) of the cross-section is small (less than about 0.15). This ratio is as high as (0.6-0.7) for the leeward columns of a tall building.
- 4)For concrete-filled sections, the ratio of the squash load of the outer steel cross-section (hollow section) to the squash load of the whole (steel plus concrete) section, γ , is small. (γ is about 0.2 for bridge piers and is about 0.75 for columns in buildings.) [1].

The forces, displacements and stresses generated on the partial concrete filled-steel tubular columns will be calculated and the behavior of tubular columns will be investigated. The effect of concrete filled ratio (hc/h) on the ductility, strength and local buckling that caused by subjecting earthquake will be studied by modeling these columns and obtaining the results of induced forces and stresses.



Concrete Core

Figure 1: Column cross-section in bridge piers.

2. Case Study under Investigation

The piers under investigation herein are those of Al- Tala'a overpass in Baghdad. Al- Tala'a overpass is a new one to be built in Al-karkh side of Baghdad city. It will connect Bab Al- Muadhum Bridge to main highway connecting Al-Alawi center to Aden square. It is a long overpass of more than 2800m length. It crosses all railway lines, getting out of Baghdad central railway station. Due to the existing and crossing eleven railway lines, this fact put two restrictions to the overpass namely long spans of not less than 40m and difficulty in erecting forms and casting the reinforced concrete columns of the overpass. This fact was the main idea behind using steel tubes for the columns. Since there are more than 68 piers in the overpass, each with three columns per pier (total of 204 columns of 6m height), the amount of concrete used in filling these steel columns will be huge. This is the reason behind trying to minimize concrete amount through studying reducing concrete level in the columns up to the level where buckling occurs, without affecting the load carrying capacity and strength of the pier under earthquake.

Volume 5 Issue 12, December 2016 <u>www.ijsr.net</u> Licensed Under Creative Commons Attribution CC BY This overpass is made of multiples of 2 symmetric continuous spans of 40m each as shown in Figure (2). Cross head and slab are made of reinforced concrete but the columns and main girders are constructed of steel sections. All dimensions and properties of materials are listed in Tables (1) to (4).





ruble 1. material properties of cross neads				
	Concrete			
E_c	Young modulus (MPa)	29725		
fc`	Compressive strength (MPa)	40		
V	Poisson's ratio	0.2		
Yc	Density of fill concrete (kN/m ³)	24		
	Steel			
Туре	A 50 steel	-		
E_s	Young's modulus (MPa)	200000		
fy	Yield stress (MPa)	345		
V	Poisson's ratio	0.3		
γs	Density (kN/m ³)	78.6		

Table	1:	Material	properties	of	cross	heads
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	Table 2:	Material	properties	of deck slab
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	1 1	
	Concrete	
E_c	Young modulus (MPa)	27805
f_c	Compressive strength (MPa)	35
V	Poisson's ratio	0.2
γc	Density of reinforced concrete deck(kN/m ³)	24.5
	Steel	
Туре	A 50 steel	-
E_s	Young's modulus (MPa)	200000
f_v	Yield stress (MPa)	345
V	Poisson's ratio	0.3
γ_s	Density (kN/m ³)	78.6

Table 3: Section properties of tubular columns

A A	
Cross-sectional shape	Circular
diameter of cross section D (mm)	1200
Effective height of columns L1 (mm)	6000
Thickness of steel columns t (mm)	12.7
Area of cross section A (mm2)	4.74×104
Moment of inertia of cross section I (mm4)	83.5×108

Table 4: Section properties of cross heads.

Cross-sectional shape	Rectangle
width of cross section w (mm)	2000
depth of cross section H (mm)	1400
length L2 (mm)	11750
Area of cross section A (mm2)	28×105
Moment inertia of cross section about X axis (mm4)	4.57×1011
Moment inertia of cross section about Z axis (mm4)	9.33×1011
Moment inertia of cross section about X axis (mm4) Moment inertia of cross section about Z axis (mm4)	4.57×1011 9.33×1011

3. Overpass Loading

The overpass was designed to carry out the Iraqi standard loading made of military and uniform loads besides the abnormal loading of the British Standard (BS 5400) HB vehicle whichever is the greater. This, besides the earthquake loading from true actual earthquake response took place in Ali Al-Garbee in Iraq.

Ali AL-Gharbee earthquake, happened in Maysan province south of Iraq in 20 April 2013 with magnitude Mw= (4.9) with peak ground acceleration of 104.151 cm/sec2. Input ground motions (acceleration against time plot) for this study is shown in Figure (3). The earthquake ground motion is chosen for the following reasons:

- 1)It is up to date recorded Iraqi earthquake.
- 2)To check the typical Iraqi bridge structures to sustain real earthquakes happened in Iraq.

Earthquake loads have been applied to the overpass structure model in two perpendicular directions in the level of piers foundation. The first is in the x-direction which is parallel to the structure plane and the other is in y-direction which is perpendicular to the structure plane [2].



Figure 3: Ali Al-Garbee Acceleration-Time Plot

4. Finite Element Modeling

4.1 Concrete Modeling

For concrete simulation, 3D solid brick element is used. This element is used for the three-dimensional modeling of solids with or without reinforcing bars (rebar) as shown in Figure (4). The element is defined by eight nodes having three translational degrees of freedom at each node. The concrete filled was divided into longitudinal circular layers of 10 cm thickness and radial with 14.4° angle making 25 elements radially for each layer.



Figure 4: 3D Solid Brick Concrete Elements.

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4.2 Steel Tubular Column Modeling

For steel tubes simulation, shell element will be used for that purpose. It is suitable for analyzing thin to moderately-thick shell structures as shown in Figure (5). Shells have four nodes with six degrees of freedom at each node: a translation in the x, y, and z directions, and rotations about the x, y, and z axes. The steel tube is divided longitudinally into layers of 10 cm thickness following the concrete fill. In thickness, the steel is modeled using one shell element.



Figure 5: Shell Element for Steel Tubular Column

5. Analysis Results of Overpass Structure

From the loading combinations shown below, the worst loading case gave the bending moments and shear forces shown in Figure (6 to 9) for YZ planes of the overpass structure subjected to the original Ali Al-Gharbee earthquake ground-motion.

1) 1.1D + 1.3L +1.4E [ASCE 7-10]. 2) 1.3+1L +1E [ASCE 7-10]. 3) 1.2D +1E +1L + 0.2 SDL [ACI 343].



Figure 6: Bending Moment for Second (Middle) Piers in the XZ Plane about (Worst Case loading)



Figure 7: Bending Moment for First Pier in the YZ (Worst Case loading)



Figure 8: Shear Force in Second (Middle) Pier in the YZ Plane (Worst Case loading)



Figure 9: Shear Force in Third Pier in the YZ Plane (Worst Case loading)

6. Case under Study

In construction situations where concrete amount necessary to fill the full columns height is costly effective like the case under investigation herein, it is highly recommended to fill partially the columns without affecting their strength and ductility. The steel columns will be of 1200 mm in diameter and 12.7 mm in thickness. They will be filled with different heights of concrete ratios, namely (0%, 25%, 30%, 50% and 100%) of the total column height of 6m. These five columns models shown in Figures (10) & (11) will be checked under the shear forces and bending moments resulted from the bridge analysis. Steel grade used in these models is (A50) steel, Radius-thickness ratio parameter (R_t) is 0.134 and the slenderness ratio parameter (λ) is 0.101. These columns are considered cantilevered-type with fixed base conditions at the footing.



Figure 10: Concrete filling Ratio for the composite of Columns (hc/h) = (0%, 25%, 30%, 50% and 100%) for Modeling (a, b, c, d, and e) columns respectively

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Figure 11: Showing Steel Tubular Column Meshing in the Composite Column with Different Concrete Filling Ratio (0%, 25%, 30%, 50% and 100%).

From the analysis the following results are obtained

6.1 Stresses

The stresses are calculated at specific locations. Since the buckling will generally occur near the bottom of the tubular column or above the concrete filling level, five locations distributed vertically starting 200 mm above fixed base then in intervals of 100 mm giving total of five locations in the filling zone and another three locations above the concrete filling level starting 100 mm above concrete level. Tabulated stress results of analyses are shown in Tables (5) to (14). Normal stress distribution on the exterior face σ_{EF} and the normal distribution stress in the interior face σ_{IF} of composite steel columns and the shearing stresses are given. The maximum and minimum values of all types of stresses are illustrated in Figures (12) to (16).

The distribution of normal and shear stresses at the two faces (interior and exterior) of the steel column for all concrete filled levels are shown in Figures (17) to (20).

Table 5: Longitudinal Normal Stresses in the Interior and Exterior Face of Columns (MPa) With (hc/h= 0%).

No.	Location	σ_{IF}	σ_{EF}
1	200 mm above base	723.1	620.4
2	300 mm above base	588.6	612.8
3	400 mm above base	447.7	544.9
4	500 mm above base	252.7	317.7
5	600 mm above base	326.7	241.6

Table 6: Longitudinal Normal Stresses in the Interior and Exterior Face of Columns (MPa) With (hc/h= 25%).

LACOIN	Exterior ruce of columns (wird) with (herit 2576).					
No.	Location	σ_{IF}	σ_{EF}			
1	200 mm above base	412	434			
2	300 mm above base	392	404			
3	400 mm above base	366	378			
4	500 mm above base	345	365			
5	600 mm above base	323	357			
6	100 mm above concrete level	387	390			
7	200 mm above concrete level	368	376			
8	300 mm above concrete level	347	362			

Fable 7: 1	Longitud	linal N	Norm	al St	resses i	in Iı	nteri	or ai	nd
Exterior	Face of (Colun	nns (l	MPa)) With	(hc/	'h= 3	0%)).

		<u>``</u>	
No.	Location	σ_{IF}	σ_{EF}
1	200 mm above base	389	403
2	300 mm above base	377	390
3	400 mm above base	354	367
4	500 mm above base	331	343
5	600 mm above base	307	319
6	100 mm above concrete level	379	392
7	200 mm above concrete level	345	368
8	300 mm above concrete level	331	348

Table 8: Longitudinal Normal Stresses in the Interior and Exterior Face of Columns (MPa) With (hc/h= 50%).

Exterior ruce of columns (in d) with (ne/n 50/0).					
No.	Location	σ_{IF}	σ_{EF}		
1	200 mm above base	334	354		
2	300 mm above base	339	351		
3	400 mm above base	325	337		
4	500 mm above base	302	313		
5	600 mm above base	288	296		
6	100 mm above concrete level	322	331		
7	200 mm above concrete level	310	316		
8	300 mm above concrete level	291	307		

Table 9: Longitudinal Normal Stresses in the Interior and Exterior Face of Columns (MPa) With (hc/h=100%).

			/
No.	Location	σ_{IF}	σ_{EF}
1	200 mm above base	267	283
2	300 mm above base	259	270
3	400 mm above base	243	250
4	500 mm above base	232	245
5	600 mm above base	213	227

Table 10: Shear Streses values in the Interior and Exterior Face of Columns (MPa) With (hc/h= 0%).

No.	Location	$ au_{IF}$	$ au_{EF}$
1	200 mm above base	196.1	172.3
2	300 mm above base	87.1	85.7
3	400 mm above base	68.7	69.3
4	500 mm above base	63.2	73.3
5	600 mm above base	54.3	61.3

Table 11: Shear Stresses values in the Interior and ExteriorFace of Columns (MPa) With (hc/h= 25%).

No.	Location	$ au_{IF}$	$ au_{EF}$
1	200 mm above base	47.3	38.9
2	300 mm above base	40.7	33.5
3	400 mm above base	37.4	29.6
4	500 mm above base	29.8	23.5
5	600 mm above base	23.3	19.3
6	100 mm above concrete level	31	29
7	200 mm above concrete level	27	27
8	300 mm above concrete level	24	26

Table 12: Shear Stresses values in the Interior and ExteriorFace of Columns (MPa) With (hc/h= 30%).

No.	Location	$ au_{IF}$	$ au_{EF}$
1	200 mm above base	52.1	43.2
2	300 mm above base	45.2	37.3
3	400 mm above base	42.6	35.5
4	500 mm above base	35.1	31.6
5	600 mm above base	28.7	24.7
6	100 mm above concrete level	27	25
7	200 mm above concrete level	24	23
8	300 mm above concrete level	20	22

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3 400 mm above base 49.1 40.2 42.2 4 500 mm above base 36.1 5 600 mm above base 35.7 32.2 100 mm above concrete level 37 34 6 200 mm above concrete level 33 31 7 28 8 300 mm above concrete level 32 **Figure (11d):** Shear Stress in The Interior Face (τ_{IF}) (MPa) of Shell Elements With (hc/h= 0%). Table 14: Shear Stresses values in the Interior and Exterior Face of Columns (Mpa) With (hc/h=100%) No. Location τ_{IF} τ_{EF} 200 mm above base 71.1 63.2 2 300 mm above base 64.4 56.3 3 400 mm above base 61.9 51.7 4 500 mm above base 54 45.2 5 600 mm above base 46.3 42.5 W.İJSr. Figure (13a): Longitudinal Normal Stress in The Exterior Face (σ_{EF}) (MPa) of Shell Elements With (hc/h=25%). Figure (12a): Longitudinal Normal Stresses in The Exterior Face (σ_{EF}) (MPa) of Shell Elements With (hc/h= 0%). Figure (13b): Longitudinal Normal Stress in The Interior 1.20 0.96 Face (σ_{IF}) (MPa) of Shell Elements With (hc/h= 25%). 0.72 0.48 0.24 0.00 -0.24 -0.48 -0.72 -0.96 -1.20 -1.44 Figure (12b): Longitudinal Normal Stresses in The Interior Face (σ_{IF}) (MPa) of Shell Elements With (hc/h= 0%). **Figure (13c):** Shear Stress in The Exterior Face (τ_{EF}) (MPa)

Table 13: Shear Stresses values in the Interior and Exterior Face of Columns (MPa) With (hc/h= 50%).

 au_{IF}

58.4

52.7

 τ_{EF}

51.8

44.8

Location

200 mm above base

300 mm above base

No.

1

2





of Shell Elements With (hc/h= 25%).



Figure (12c): Shear Stress in The Exterior Face (τ_{EF}) (MPa) of Shell Elements With (hc/h=0%).

Figure (13d): Shear Stress in The Interior Face (τ_{IF}) (MPa) of Shell Elements With (hc/h=25%).

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Figure (14a): Longitudinal Normal Stress in The Exterior Face (σ_{EF}) (MPa) of Shell Elements With (hc/h= 30%).



Figure (15b): Longitudinal Normal Stress in The Interior Face (σ_{IF}) (MPa) of Shell Elements With (hc/h= 50%).



Figure (14b): Longitudinal Normal Stress in The Interior Face (σ_{IF}) (MPa) of Shell Elements With (hc/h= 30%).



Figure (15c): Shear Stress in The Exterior Face (τ_{EF}) (MPa) of Shell Elements With (hc/h= 50%).



Figure (14c): Shear Stress in The Exterior Face (τ_{EF}) (MPa) of Shell Elements With (hc/h= 30%).



Figure (15d): Shear Stress in The Interior Face (τ_{IF}) (MPa) of Shell Elements With (hc/h= 50%).



Figure (14d): Shear Stress in The Interior Face (τ_{IF}) (MPa) of Shell Elements With (hc/h= 30%).



Figure (16a): Longitudinal Normal Stress in The Exterior Face (σ_{EF}) (MPa) of Shell Elements With (hc/h= 100%).





Figure (16b): Longitudinal Normal Stress in The Interior Face (σ_{IF}) (MPa) of Shell Elements With (hc/h= 100%).

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Figure (16c): Shear Stress in The Exterior Face (τ_{EF}) (MPa) of Shell Elements With (hc/h= 100%).



Figure (16d): Shear Stress in The Interior Face (τ_{IF}) (MPa) of Shell Elements With (hc/h= 100%).



Figure (17) : Comparison Normal Stresses (MPa) in The Exterior Face of Column with Different Concrete Filling Ratio (hc/h) %.



Figure (18): Comparison Normal Stresses (MPa) in The Interior Face of Column with Different Concrete Filling Ratio (hc/h) %.



Figure (19): Shear Stresses (MPa) in The Exterior Face of Column with Different Concrete Filled Ratio (hc/h) %.



Figure (20): Shear Stresses (MPa) in The Interior Face of Column with Different Concrete Filled Ratio (hc/h) %.

6.2 Ductility Factor (µ)

It is an important parameter in assessing the ductility of columns. It is an index which helps in estimating the seismic behavior of CFST column. It can be found using Equation (1) [3] with the load-deformation curves. The Load-deformation curves for the five columns modeling are shown in Figures (21) to (25).



Figure (21): Load Deformation Curve of column with (hc/h=0%).



Figure (22): Load Deformation Curve of column with (hc/h= 25%).



Figure (23): Load Deformation Curve of column with (hc/h= 30%).

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Figure (24): Load Deformation Curve of column with (hc/h= 50%).



Figure (25): Load Deformation Curve of column with (hc/h= 100%).

$$\mu m = \frac{\delta u}{\delta y} \tag{1}$$

Depending on these relations between load and deformation; the ductility factor for each ratio of concrete filling can be found. The calculated values of the ductility factor μ_m are listed in Table (15) and plot of μ_m against the concrete-filling ratio is shown in Figure (26). As seen from the figure, the ductility factor μ_m considerably becoming higher when the height ratio of the fill-in concrete is increased. In other words, it can be seen from Figure (26) that, comparing with specimen with (hc/h=0%) without concrete infill, the ductility factors μ_m of specimen (hc/h=25%) increased by 6.5%, while in the case of specimen (hc/h=30%), the corresponding values increased by 10%, and in the case of specimen (hc/h=50%) increased by 20%. This can be attributed to the confinement of the internal concrete which prevented local buckling of steel plates forming the tubular column. Accordingly, some improvement of ductility capacity of the circular steel section due to the fill-in concrete can be expected even when a diaphragm is not provided on the top face of the concrete.

 Table 15: Values of ductility factor for different concrete filling ratios (hc/h)

(hc/h) Ratio	δy	би	μт
0%	35.5	107	3.014
25%	31.5	101	3.21
30%	29.3	97	3.31
50%	24.7	89	3.6
100%	23.1	71	3.02



Figure (26): Effect of filled-in concrete on ductility.

It is clear the effect of concrete filling on the ductility factor of the steel column. Filling concrete with hc/h = (30 to 50) % will give the best results in ductility and strength.

7. Conclusions

- 1) The partially (CFST) circular columns specimens (hc/h = 0.25), (hc/h = 0.3) and (hc/h = 0.50) showed considerably better deformation capacity than specimen (hc/h = 0.0) without encased concrete. However, for the specimen (hc/h = 0.25) and lowest ratios infill, showed that the filled-in concrete of an insufficient length made a very small contribution to the ultimate load since the load was hardly transmitted to the concrete through the frictional contact between the external plate with internal concrete. On the other hand, the specimens (hc/h = 0.3) and (hc/h = 0.5) possess a sufficient length of infill concrete, and the filled-in concrete was found to be effective in improving the strength and ductility of columns.
- 2) Using the composite circular steel columns with controlled height of concrete will reduce the total amount of concrete used for the pier columns which will effectively reduce the total cost of overpass.
- **3)** Ductility of the steel tubular columns case study shows improvement when the height of concrete fill is increased to (30 to 50) percent of the column height. This satisfies one of the main ideas underlying partially concrete-filled strengthening technique: namely to increase the ductility for resisting strong earthquakes without substantially increasing the strength.
- 4) Using circular steel columns will solve the problems rising during construction limited working areas such as in the case study where railway lines affected area availability for constructing the overpass. This also reduces total construct on period.
- 5) Local buckling of the steel tubular columns considerably reduces the stiffness and strength of circular thin-plate partially (CFST) columns and thus must be taken into account in the nonlinear analysis as carried out in this work.

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