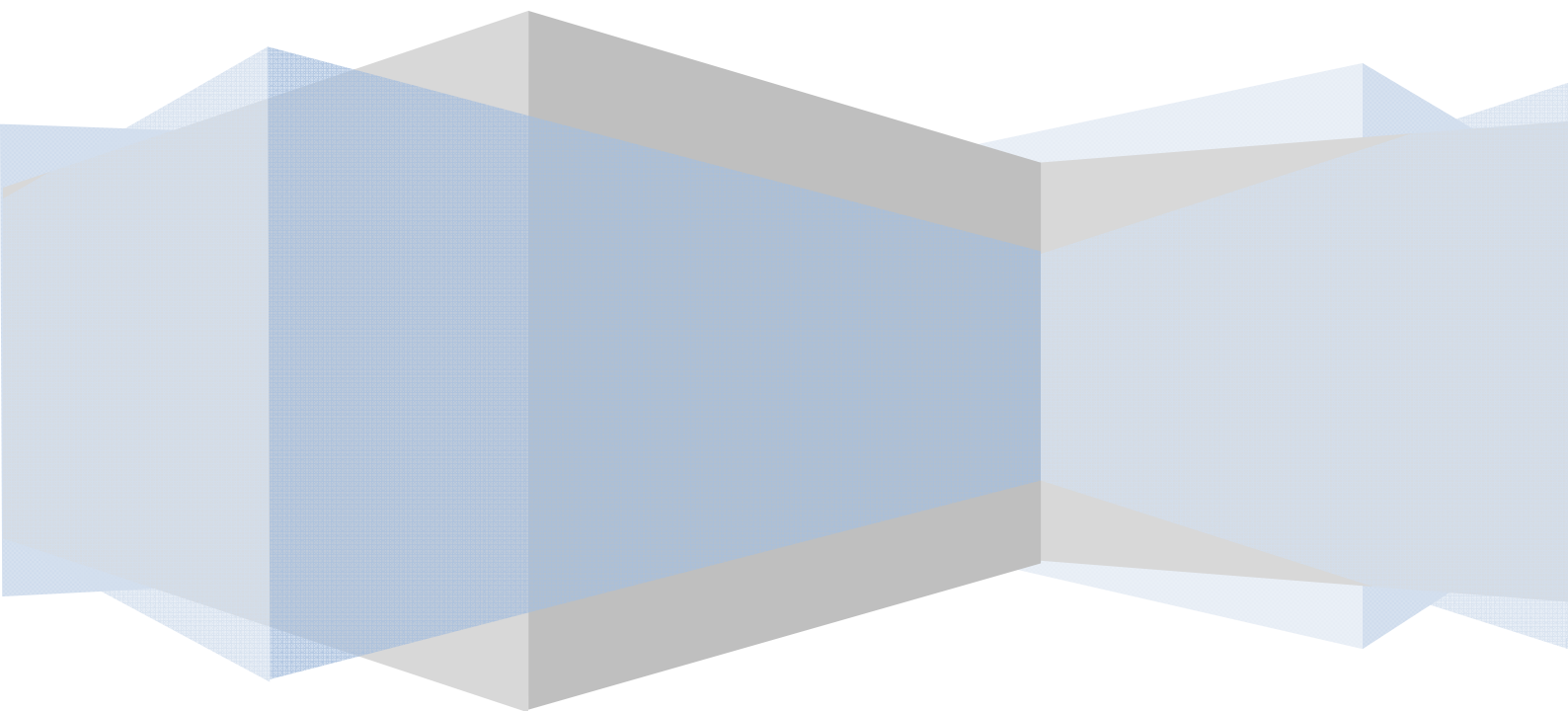


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Nonlinear Finite Element Analysis of CFT-to-Bracing Connections Subjected to Axial Compressive Forces

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ABSTRACT: The ABAQUS finite element program together with a nonlinear material constitutive model for concrete-filled tube (CFT) and steel gusset plate is used to analyze the behaviors of the gusset plate type CFT-to-Bracing connections subjected to axial compressive forces. It is found that the failure of CFT-to-Bracing connections occurs below the connecting area. The ultimate strengths of the CFT columns slightly increase with the increasing of the load ratio and the thickness of the gusset plate. The introduction of the cutouts on the gusset plates also slightly increases the ultimate strengths of the CFT columns.

Keywords: concrete-filled tube, bracing, axial force, finite element analysis.

1. Introduction

A concrete-filled tube (CFT) column consists of a steel tube filled with concrete. Due to the benefit of composite action of both materials, the CFT columns provide excellent seismic resistant structural properties such as high strength, high ductility and large energy absorption capacity. Therefore, CFT columns have gained popularity in supporting heavy loads in high-rise buildings, bridges and offshore structures. Various experimental and analytical studies have been performed on CFT columns [1-17] and special interests have been focused on the connection regions [18-24].

The aim of this investigation is to employ the nonlinear finite element program ABAQUS [25] to perform numerical simulations of the gusset plate type CFT-to-bracing connections (Fig. 1) subjected to axial compressive forces. To achieve this goal, proper material constitutive models for steel gusset plate, steel tube and concrete core are proposed. Then the proposed material constitutive models are verified against experimental data of Yang [26]. Finally, the influence of the thickness of the gusset plate, the opening of the gusset plate, and the type of loading on the behavior of CFT-to-Bracing connections are studied and discussed.

2. Material properties and constitutive models

The experiment for CFT-to-bracing connections subjected to axial compressive forces (Fig. 1) was carried out by Yang [26]. There are seventeen specimens and their dimensions are given in Table 1. These CFT columns are subjected to compressive forces P_a and P_g to the end of the columns and the gusset plates, respectively. They can be categorized into three groups. The first group of the CFT column is the S specimen which has no gusset plate at all and is subjected to axial compressive force P_a to the end only. The second group of the CFT columns is the P specimen. The first number after P is the thickness of the gusset plate (in mm), which could be 12, 24 and 36. The second number after P is the load ratio $P_a/(P_a + P_g)$ in term of percentage. For example, P24-67 specimen stands for CFT column having a gusset plate with 24 mm in thickness and a load ratio $P_a/(P_a + P_g) = 0.67$. The third group of the CFT columns is the PH specimen. Same as the P specimen, the first number after PH is the

thickness of the gusset plate and the second number after PH is the load ratio. However, the gusset plate of the PH specimen contains circular cutouts. The 24 mm gusset plate has 2 circular holes and the 36 mm gusset plate has 3 circular holes. The dimensions and positions of these holes in the gusset plate are given in Fig. 2.

2.1 Steel tube and steel gusset plate

In the analysis, the Poisson's ratio ν_s and the elastic modulus E_s of the steel tube and steel gusset plate are assumed to be 0.3 and 200 GPa, respectively. The uniaxial behavior of the steel tube and the steel gusset plate is modeled by a piecewise linear model and their stress-strain curves used in the analysis are shown in Fig. 3.

When the steel tube and the steel gusset plate are subjected to multiple stresses, a von Mises yield criterion is employed to define the initial yield surface, which is written as

$$F = \frac{1}{\sqrt{2}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} - \sigma_y = 0 \quad (1)$$

where σ_1 , σ_2 , and σ_3 are the principal stresses. Figure 4 shows the von Mises yield surface in the three-dimensional principal stress space. The response of the steel tube and the steel gusset plate is modeled by an elastic-hardening plastic theory with isotropic hardening rule and associated flow rule. When plastic deformation occurs, there should be a certain parameter to guide the expansion of the yield surface. A commonly used approach is to relate the multidimensional stress and strain conditions to a pair of quantities, namely, the effective stress f_s and effective strain ϵ_s , such that results obtained following different loading paths can all be correlated by means of the equivalent uniaxial stress-strain curve. In this investigation, the uniaxial stress-strain curves given in Fig. 3 are used as the equivalent uniaxial stress-strain curve for the steel tube and the steel gusset plate. As the result, the subsequent yield surface of the tub and the gusset plate can be written as

$$F = \frac{1}{\sqrt{2}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} - f_s = 0 \quad (2)$$

2.2 Concrete

The Poisson's ratio ν_c of concrete under uniaxial compressive stress ranges from 0.15 to 0.22, with a representative value of 0.19 or 0.20 [27]. In this study, the Poisson's ratio of concrete is assumed to be 0.2.

Let the uniaxial compressive strength and the corresponding strain of the unconfined concrete be f_c and ε_c (Fig. 5). The value of ε_c is usually around the range of 0.002 to 0.003. A representative value 0.002 is used in the analysis. When concrete is subjected to laterally confining pressure, the uniaxial compressive strength f_{cc} and the corresponding strain ε_{cc} (Fig. 5) are much higher than those of unconfined concrete. The relations between f_{cc} , f_c and between ε_{cc} , ε_c are estimated by the following equations [28]:

$$f'_{cc} = k_4 f'_c + k_1 f_l \quad (3)$$

$$\varepsilon'_{cc} = \varepsilon_c \left(1 + k_2 \frac{f_l}{f_c} \right) \quad (4)$$

where f_l represents the confining pressure around the concrete core. k_1 and k_2 are constants and can be obtained from experimental data. Meanwhile, the constants k_1 and k_1 are set as 4.1 and 20.5 based on the studies of Richart et al. [29]. The original form of Eq. (3) was proposed for concrete subjected to hydrostatic pressure [28] and did not contain the strength factor k_4 . For CFT subjected to bending moment, part of the concrete may be subject to tensile stress, which is different from the hydrostatic pressure condition. Therefore, a strength factor k_4 is introduced by the authors [15] with the limitation $0 \leq k_4 \leq 1$.

Because the concrete in the CFT columns is usually subjected to triaxial compressive stresses, the failure of concrete is dominated by the compressive failure surface expanding with increasing hydrostatic pressure. Hence, a linear Drucker-Prager yield criterion G (Fig. 6) is used to model the yield surface of concrete, which is expressed as

$$G = t - p \tan \beta - d = 0 \quad (5)$$

where

$$p = -(\sigma_1 + \sigma_2 + \sigma_3) / 3 \quad (6.a)$$

$$d = \left(1 - \frac{\tan \beta}{3} \right) f'_{cc} \quad (6.b)$$

$$t = \frac{\sqrt{3}J_2}{2} \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{\sqrt{3}J_2} \right)^3 \right] \quad (6.c)$$

$$r = \left[\frac{9}{2} (S_1^3 + S_2^3 + S_3^3) \right]^{1/3} \quad (6.d)$$

and S_1 , S_2 , and S_3 are principal stress deviators. The constants K and β are material parameters determined from experimental data. In the analysis, K and β are set to 0.8 and 20° , respectively [14,15].

The response of the concrete is also modeled by an elastic-hardening plastic theory with isotropic hardening rule and associated flow rule. The stress-strain relationship proposed by Saenz [30] has been widely adopted as the uniaxial stress-strain curve for concrete and it has the following form:

$$f_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2) \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right) - (2R - 1) \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^2 + R \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^3} \quad (7)$$

where

$$R = \frac{R_E (R_\sigma - 1)}{(R_E - 1)^2} - \frac{1}{R_E}, \quad R_E = \frac{E_c \varepsilon_{cc}}{f'_{cc}}$$

and $R_\sigma = 4$, $R_E = 4$ may be used [31]. The initial modulus of elasticity of concrete E_c is highly correlated to its compressive strength and can be calculated with reasonable accuracy from the empirical equation [32]:

$$E_c = 4700 \sqrt{f'_{cc}} \text{ MPa} \quad (8)$$

In the analysis, Eq. (7) is taken as the equivalent uniaxial stress-strain curve for concrete when the concrete strain ε_c is less than ε_{cc} (Fig. 4). When $\varepsilon_c > \varepsilon_{cc}$, a linear descending line is used to model the softening behavior of concrete. If k_3 is defined as the material degradation parameter, the descending line is assumed to be terminated at the point where $f_c = k_3 f_{cc}$ and $\varepsilon_c = 1.1 \varepsilon_{cc}$ [14,15].

3. Finite element model for CFT columns

The specimen for CFT-to-bracing connections subjected to axial compression force has been shown in Fig. 1. Due to symmetry, only one fourth of the CFT-to-bracing connection is analyzed (Fig. 7). Symmetric boundary conditions are enforced on the symmetric planes, which are $u = 0$ on the front surface of the element mesh and $v = 0$ on the top surface of the element mesh surface. To model the fixed end condition, $u = v = w = 0$ are set at the right surface of the element mesh. In addition, $u = v = 0$ are set at the left surface of the element mesh to simulate the boundary condition of the loading end.

In the finite element mesh, the concrete core, the steel tube and the steel gusset plate are all modeled by 8-node isoparametric solid elements (three degrees of freedom per node) with reduced integration rule. There are three interfaces existed in the finite element mesh, which are the concrete - tube interface, concrete - gusset plate interface and the tube - gusset plate interface. All these interfaces are modeled by pairs of contact surfaces. The nodes on these interfaces are connected through the contact surfaces that require matching meshes on the both sides of the interfaces. These contact surfaces can model infinitesimal sliding and friction [25]. The friction coefficient used in all the analyses is $\mu = 0.25$ [14,15]. As the results, the nodes on the interfaces are allowed to either contact or separate but not to penetrate each other.

Convergent studies of the finite element meshes have been done by the authors using various element sizes for CFT columns [33]. It has been shown that the numerical results are not sensitive to the element sizes and mesh refinements. All the finite element meshes used in the analysis can be seen in the deformation plots of the following section.

4. Numerical analysis

In order to carry out the numerical analysis, the

parameters f_l , k_3 and k_4 of the confined concrete should be provided to completely define the equivalent uniaxial stress-strain relation. Based on the suggestion by previous research [14], the following expression is used to calculate the lateral confining pressure of concrete f_l .

$$f_l / f_y = 0.043646 - 0.000832(D/t) \quad (9)$$

where $f_y = 308.5$ MPa is the yield stress of the steel tube. Since the D/t value of all the specimens is equal to 37.86, we obtain $f_l = 3.748$ MPa and this value is used in all the analyses.

The values of the parameters k_3 and k_4 apparently depend on the cross-sectional geometry and material properties. Consequently, their appropriate values are determined by matching the numerical results with experimental data via parametric studies in the following sections.

4.1 Simulation of the material degradation parameter k_3

To make a proper selection of the material degradation parameter k_3 , different values of k_3 are simulated for CFT specimens S, P24-00 and P24-100. The selected k_3 values are $k_3 = 0.4, 0.45, 0.6$ and 0.8 , respectively. Based on $k_4 = 0.75$, the load - displacement curves of the experimental data and the numerical results for those specimens are shown in Fig. 8. The vertical axes and the horizontal axes of the Fig. 8 are the total load $P = P_a + P_g$ and the shortening of the LVDT displacement transducer. The positions of the LVDT displacement transducers on the CFT columns in the experimental set up are shown in Fig. 9. From Fig. 8(a), we can observe that the numerical result with $k_3 = 0.6$ are in good agreement with the experimental data of specimen S. From Figs. 8(b) and 8(c), we can observe that the numerical results with $k_3 = 0.45$ are in good agreement with the experimental data of specimens P24-00 and P24-100. The reason that the value of k_3 for specimen S is greater than those for specimens P24-00 and P24-100 is because the S specimen does not have a gusset plate. Since there are no cutouts on the tube, specimen S would have a better confining effect than specimens P24-00 and P24-100. As the result, after the ultimate strength of the concrete has been reached, the concrete strength of the CFT column without gusset plate is stronger than those with gusset plate. Similar results are also observed for other CFT columns. Therefore, in the following analyses, $k_3 = 0.6$ is used for CFT column without gusset plate and $k_3 = 0.45$ is used for CFT column with gusset plate.

4.2 Simulations of the strength factor k_4

The strength factor k_4 varies between 0 and 1. To make a proper selection of the strength factor k_4 , different values of k_4 are used to simulate the behavior of CFT specimens S, P24-00 and P24-100 again. Those selected k_4 values are $k_4 = 0.5, 0.75$ and 1 , respectively. Based on the values of k_3 suggested in the previous section, the load - displacement curves of the experimental data and the numerical results for those specimens are shown in Fig.

10. We can observe that the numerical results with $k_4 = 0.75$ are in good agreement with the experimental data, even though the gusset plates and the loading patterns of these specimens are in quite different arrangements. This justifies the use of $k_4 = 0.75$ in previous section in determining the values of parameter k_3 . Similar results are also observed for other CFT columns. Therefore, the value $k_4 = 0.75$ is used through out the numerical analysis.

4.3 Influence of the friction coefficient

To study the influence of the friction coefficient between the contact surfaces on the behavior of the CFT columns, different values of μ are used to simulate the behavior of CFT specimens P24-00 and P24-100. Those selected values are $\mu = 0, 0.25, 0.5, 0.75$ and 1 , respectively. Based on the values of k_3 and k_4 suggested in the previous sections, the load - displacement curves of the experimental data and the numerical results for those specimens are shown in Fig. 11. We can observe that the numerical results with various μ are very close. Similar results are also observed for other CFT columns. This confirms that the value of the friction coefficient μ does not have significant influence on the behavior of the CFT columns.

4.4 Simulations of the S specimen

The S specimen does not have any gusset plate and is subjected to compressive force P_a at the end. The load - displacement curves of the experimental data and the numerical results for S specimen are shown in Fig. 12 and their ultimate loads are listed in Table 2. It can be seen that the result of numerical analysis is very close to the experimental data. In Fig. 12, the reaction forces contributed from concrete and steel tube at the fixed end are also shown. It is clear that the steel tube yields first. After the concrete reaches its ultimate strength, the entire CFT column starts to soften up to failure. Figure 13 shows the deformation shape of the CFT column close to the failure stage. It can be seen that the concrete core and steel tube still keep in contact to each other and no local buckling of the tube takes place.

4.5 Simulations of the P12 specimens

The P12 specimens have gusset plates with 12 mm in thickness. The load - displacement curves of the experimental data and the numerical results for P12-00, P12-33 and P12-67 specimens are shown in Fig. 14 and their ultimate loads are listed in Table 2. Generally, the results of the numerical analyses are in good agreement with the experimental data.

The P12-00 specimen is subjected to force P_g applied to the gusset plate only. Probably due to the stress concentration effect around the gusset plate area, part of the concrete core would fail before the yielding of the steel tube occurs (Fig. 14a). After the yielding of the steel tube takes place, the concrete can still carry out additional axial load up to its ultimate strength. The P12-33 and P12-67 specimens are subjected to forces P_a and P_g simultaneously. Similarly to the S specimen, the steel tube yields first (Fig. 14b and Fig. 14c). After the concrete reaches its ultimate strength, the

entire CFT column starts to soften up to failure. From Fig. 14 and Table 2 we can observe that the change of the load ratio, i.e. $P_a / (P_a + P_g)$, does not influence the ultimate strength of CFT columns significantly.

Figure 15 shows the deformation shapes of the CFT columns close to the failure stage. It can be seen that the deformation of the column below the gusset plate is greater than those of other places. Local buckling of the steel tube might take place in the areas close to the gusset plate and the fixed end.

4.6 Simulations of the P24 specimens

The P24 specimens have gusset plates with 24 mm in thickness. The load - displacement curves of the experimental data and the numerical results for P24-00, P24-33 P24-67 and P24-100 specimens are shown in Fig. 16 and their ultimate loads are listed in Table 2. Generally, the results of the numerical analyses are in good agreement with the experimental data.

In spite of the loading patterns, the steel tube of all the specimens yields first. After the concrete reaches its ultimate strength, the concrete as well as the entire CFT column starts to soften up to failure. From Fig. 16 and Table 2 we can observe again that the change of the load ratio does not influence the ultimate strength of CFT columns significantly. However, when the load ratio becomes large, the CFT column would exhibit more softening behavior after its ultimate strength been reached. In addition, we might conclude that the ultimate strengths of the CFT columns slightly increase with the increasing of the load ratio.

Figure 17 shows the deformation shapes of the CFT columns close to the failure stage. Again, it can be seen that the deformation of the column below the gusset plate is greater than those of other places. Local buckling of the steel tube might take place in the areas close to the gusset plate and the fixed end. Comparing Fig. 17 with Fig. 15, we can conclude that the increasing of the thickness of the gusset plate does not influence the failure patterns of CFT columns significantly.

4.7 Simulations of the P36 specimens

The P36 specimens have gusset plates with 36 mm in thickness. The load - displacement curves of the experimental data and the numerical results for P36-00, P36-33 and P36-67 specimens are shown in Fig. 18 and their ultimate loads are listed in Table 2. Generally, the results of the numerical analyses are in good agreement with the experimental data.

Similar to P24 specimens, the steel tube of all the specimens yields first. After the concrete reaches its ultimate strength, the concrete as well as the entire CFT column starts to soften up to failure. Again, the change of the load ratio does not influence the ultimate strength of CFT columns significantly. However, when the load ratio becomes large, the CFT column would exhibit more softening behavior after its ultimate strength been reached. This phenomenon is more prominent for CFT columns having thicker gusset plates than those having thinner gusset plates (Figs. 14, 16 and 18). From Table 2, we can observe that the increasing of the thickness of the gusset plate does slightly increase the ultimate strength of CFT columns. However, this increasing in the

ultimate load is not significantly.

Figure 19 shows the deformation shapes of the CFT columns close to the failure stage. Comparing Fig. 19 with Figs. 17 and 15, we can conclude that the increasing of the thickness of the gusset plate does not influence the failure patterns of CFT columns significantly.

4.8 Simulations of the PH24 specimens

The PH24 specimens have gusset plates with 24 mm in thickness and have two circular cutouts on the gusset plates (Fig. 2). The load - displacement curves of the experimental data and the numerical results for PH24-00, PH24-33 and P24-67 specimens are shown in Fig. 20 and their ultimate loads are listed in Table 2. Generally, the results of the numerical analyses are in good agreement with the experimental data.

Similar to the P24 specimens, the steel tube of all the specimens yields first. After the concrete reaches its ultimate strength, the concrete as well as the entire CFT column starts to soften up to failure. From Fig. 20 and Table 2, we can observe again that the change of the load ratio does slightly increase the ultimate strength of CFT columns. When the load ratio becomes large, the CFT column would exhibit more softening behavior after its ultimate strength been reached. Comparing the ultimate strengths of PH24 specimens with those of P24 specimens (Table 2), we can see that the introduction of the cutouts on the gusset plates does slightly increase the ultimate strengths of CFT columns. This is because when the concrete fills in the cutouts on the gusset plates, more contact area between concrete and steel gusset plate are generated. As the result, the entire connection areas are reinforced.

Figure 21 shows the deformation shapes of the CFT columns close to the failure stage. Again, it can be seen that the deformation of the column below the gusset plate is greater than those at other places. Local buckling of the steel tube takes place in the areas close to the gusset plate and the fixed end. Comparing Fig. 21 with Fig. 17, we could observe that the introducing of the cutouts on the gusset plate causes more local buckling modes on the CFT columns.

4.9 Simulations of the PH36 specimens

The PH36 specimens have gusset plates with 36 mm in thickness and have three circular cutouts on the gusset plates (Fig. 2). The load - displacement curves of the experimental data and the numerical results for PH36-00, PH36-33 and P36-67 specimens are shown in Fig. 22 and their ultimate loads are listed in Table 2. Generally, the results of the numerical analyses are in good agreement with the experimental data.

Similar to the P36 specimens, the steel tube of all the specimens yields first. After the concrete reaches its ultimate strength, the concrete as well as the entire CFT column starts to soften up to failure. From Fig. 22 and Table 2, we can observe again that the change of the load ratio does slightly increase the ultimate strength of CFT columns. When the load ratio becomes large, the CFT column would exhibit more softening behavior after its ultimate strength been reached. Comparing the ultimate strengths of PH36 specimens from numerical analyses with those of P36 specimens (Table 2), we can see that

the introduction of the cutouts on the gusset plates does slightly increase the ultimate strengths of CFT columns. However, this phenomenon is not observed in the experimental data and more investigation is needed.

Figure 23 shows the deformation shapes of the CFT columns close to the failure stage. Again, it can be seen that the deformation of the column below the gusset plate is greater than those at other places. Local buckling of the steel tube takes place in the areas close to the gusset plate and the fixed end. Comparing Fig. 23 with Fig. 19, we could observe that the introducing of the cutouts on the gusset plate causes more local buckling modes on the CFT columns.

5. Conclusions

In this paper, nonlinear finite element analyses of CFT-to-Bracing connections subjected to axial compressive force are performed. Based on the numerical results, the following conclusions may be drawn:

- (1) The value of the friction coefficient μ between the concrete - tube contact surface, the concrete - gusset plate contact surface and the tube - gusset plate contact surface does not have significant influence on the behavior of the CFT-to-Bracing connections.
- (2) When the load ratio becomes large, the CFT column with gusset plate would exhibit more softening behavior after its ultimate strength been reached. In addition, the ultimate strengths of the CFT columns slightly increase with the increasing of the load ratio.
- (3) The deformation of the CFT column below the gusset plate is greater than those of other places. Local buckling of the steel tube might take place in the areas close to the gusset plate and the fixed end.
- (4) The increasing of the thickness of the gusset plate does slightly increase the ultimate strength of CFT columns. However, this increasing in the ultimate load is not significantly. In addition, the increasing of the thickness of the gusset plate does not influence the failure patterns of CFT columns significantly.
- (5) The introduction of the cutouts on the gusset plates does slightly increase the ultimate strengths of CFT columns. However, when the cutout areas are increased (say PH36 specimens), this phenomenon is not observed in the experimental result and further investigation is needed.
- (6) The introducing of the cutouts on the gusset plate causes more local buckling modes on the CFT columns under its ultimate strength.

6. References

- [1] Furlong RW. Strength of steel-encased concrete beam-columns. *Journal of the Structural Division*, ASCE 1967;93(ST5):113-124.
- [2] Knowles RB, Park R. Strength of concrete filled steel tubular columns. *Journal of the Structural Division*, ASCE 1969;95(ST12):2565-2587.
- [3] Ge HB, Usami T. Strength of concrete-filled thin-walled steel box columns: experiment. *Journal of Structural Engineering*, ASCE 1992;118(1):3036-3054.
- [4] Boyd FP, Cofer WF, McLean D. Seismic performance of steel-encased concrete column

- under flexural loading. *ACI Structural Journal* 1995;92(3):355-365.
- [5] Bradford MA. Design strength of slender concrete-filled rectangular steel tubes. *ACI Structural Journal* 1996;93(2):229-235.
- [6] Shams M, Saadeghvaziri MA. State of the art of concrete-filled steel tubular columns. *ACI Structural Journal* 1997;94(5):558-571.
- [7] Schneider SP. Axial loaded concrete-filled steel tubes. *Journal of Structural Engineering*, ASCE 1998;124(10):1125-1138.
- [8] Roeder CW, Cameron B, Brown CB. Composite action in concrete filled tubes. *Journal of Structural Engineering*, ASCE 1999;125(5):477-484.
- [9] Uy B. Strength of concrete filled steel box columns incorporating local buckling. *Journal of Structural Engineering*, ASCE 2000;126(3):341-352.
- [10] Elchalakani M, Zhao XL, Grzebieta RH. Concrete-filled circular steel tube subjected to pure bending. *Journal of Constructional Steel Research* 2001;57(11):1141-1168.
- [11] Bradford MA, Loh HY, Uy B. Slenderness limits for filled circular steel tubes. *Journal of Constructional Steel Research* 2002;58(2):243-252.
- [12] Elremaily A, Azizinamini A. Behavior and strength of circular concrete-filled tube columns. *Journal of Constructional Steel Research* 2002;58(12):1567-1591.
- [13] Huang C-S, Yeh Y-K, Liu G-Y, Hu H-T, Tsai KC, Weng YT, Wang SH, Wu M-H. Axial load behavior of stiffened concrete-filled steel columns. *Journal of Structural Engineering*, ASCE 2002;128(9):1222-1230.
- [14] Hu H-T, Huang CS, Wu M-H, Wu Y-M. Nonlinear analysis of axially loaded CFT columns with confinement effect. *Journal of Structural Engineering*, ASCE 2003;129(10):1322-1329.
- [15] Hu H-T, Huang CS, Chen ZL. Finite Element Analysis of CFT Columns Subjected to Combined Axial Force and Bending Moment, *Journal of Constructional Steel Research* 2005;61(12):1692-1712.
- [16] Ellobody E, Young B, Lam D. Behaviour of normal and high strength concrete-filled compact steel tube circular stub columns. *Journal of Constructional Steel Research* 2006;62(7):706-715.
- [17] Lu FW, Li SP, Sun G. A study on the behavior of eccentrically compressed square concrete-filled steel tube columns. *Journal of Constructional Steel Research* 2007;63(7):941-948.
- [18] Schneider SP, Alostaz YM. Experimental behavior of connections to concrete-filled steel tubes. *Journal of Constructional Steel Research* 1998;45(3):321-352.
- [19] Elremaily A, Azizinamini A. Design provisions for connections between steel beams and concrete filled tube columns. *Journal of Constructional Steel Research* 2001;57(9):971-995.
- [20] Elremaily E, Azizinamini A. Experimental behavior of steel beam to CFT column connections. *Journal of Constructional Steel Research* 2001;57(10):1099-1119.
- [21] Johansson M. Composite action in connection

regions of concrete-filled steel tube columns. *Steel and Composite Structures* 2003;3(1):47-64.

[22] MacRae G, Roeder CW, Gunderson C, Kimura Y. Brace-beam-column connections for concentrically braced frames with concrete filled tube columns. *Journal of Structural Engineering, ASCE*, 2004;130(2):233-243.

[23] Wu LY, Chung LL, Tsai SF, Shen TJ, Huang GL. Seismic behavior of bolted beam-to-column connections for concrete filled steel tube. *Journal of Constructional Steel Research* 2005;61(10):1387-1410.

[24] Choi, SM, Jung, DS, Kim, DJ, Kim, JH. An evaluation equation of load capacities for CFT square column-to-beam connections with combined diaphragm. *Steel and Composite Structures* 2007;7(4):287-301.

[25] ABAQUS, Inc., ABAQUS Analysis User's Manuals and Example Problems Manuals, Version 6.7, Providence, Rhode Island, 2007.

[26] Yang CY. Behavior of CFT-to-Bracing Connections under Axial Compression, M.S. Thesis (in Chinese), Department of Construction Engineering, National Taiwan University of Science and Technology, Taipei, Taiwan, R.O.C., 2002.

[27] ASCE Task Committee on Concrete and Masonry Structure. State of the Art Report on Finite Element Analysis of Reinforced Concrete, ASCE, New York, 1982.

[28] Mander JB, Priestley MJN, Park R. Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering, ASCE* 1988;114(8):1804-1823.

[29] Richart FE, Brandtzaeg A, Brown RL. A Study of the Failure of Concrete under Combined Compressive Stresses, Bulletin 185, University of Illinois Engineering Experimental Station, Champaign, Illinois, 1928.

[30] Saenz LP. Discussion of "Equation for the stress-strain curve of concrete" by Desayi P, Krishnan S. *ACI Journal* 1964;61:1229-1235.

[31] Hu H-T, Schnobrich WC. Constitutive modelling of concrete by using nonassociated plasticity. *Journal of Materials in Civil Engineering, ASCE* 1989;1(4):199-216.

[32] ACI Committee 318. Building Code Requirements for Structural Concrete and Commentary (ACI 318-05), American Concrete Institute, Detroit, Michigan, 2005.

[33] Chen C-W, Numerical Analysis of CFT-to-Bracing Connections Subjected to Axial Compressive Forces, M.S. Thesis (in Chinese), Department of Civil Engineering, National Cheng Kung University, Tainan, Taiwan, R.O.C., 2003.

7. Table

Table 1 Dimensions and load ratios of CFT columns

Specimen	H (mm)	h (mm)	Numbers of holes on gusset plate	Load ratio $P_a / (P_a + P_g)$ (%)
S	795	-	0	100
P12-00	1590	12	0	0
P12-33	1590	12	0	33
P12-67	1590	12	0	67
P24-00	1590	24	0	0
P24-33	1590	24	0	33
P24-67	1590	24	0	67
P24-100	1590	24	0	100
P36-00	1590	36	0	0
P36-33	1590	36	0	33
P36-67	1590	36	0	67
PH24-00	1590	24	2	0
PH24-33	1590	24	2	33
PH24-67	1590	24	2	67
PH36-00	1590	36	3	0
PH36-33	1590	36	3	33
PH36-67	1590	36	3	67

For all specimens D = 265 mm, L = 320 mm, t = 7mm, D/t = 37.86.

Table 2 Results of numerical simulations

Specimen	Ultimate Load (MN)		Error (%)
	Experiment	Analysis	
S	3.876	3.834	1.08
P12-00	3.693	3.775	2.22
P12-33	3.897	3.743	3.95
P12-67	3.871	3.805	1.70
P24-00	3.755	3.773	0.48
P24-33	3.862	3.796	1.71
P24-67	3.921	3.803	3.01
P24-100	3.803	3.812	0.24
P36-00	3.865	3.770	2.46
P36-33	4.154	3.794	8.67
P36-67	4.014	3.806	5.18
PH24-00	3.847	3.782	1.69
PH24-33	3.920	3.803	2.98
PH24-67	4.041	3.810	5.72
PH36-00	3.742	3.802	1.60
PH36-33	4.052	3.811	6.32
PH36-67	4.051	3.814	5.85

8. Figures

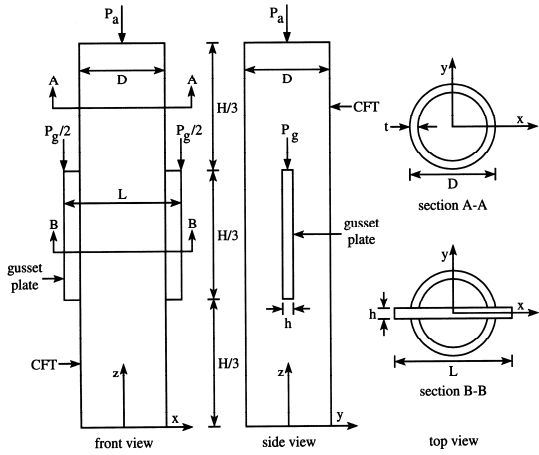


Fig. 1 Geometry and loading of gusset plate type CFT-to-Bracing connection

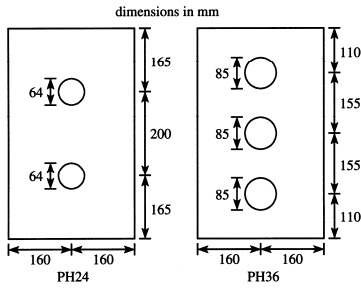


Fig. 2 Openings in the gusset plates

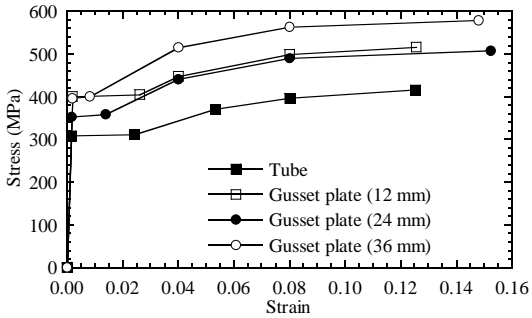


Fig. 3 Stress-strain curves of steel tube and gusset plate

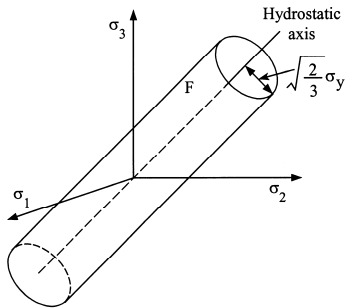


Fig. 4 von Mises yield surface in the three-dimensional principal stress space

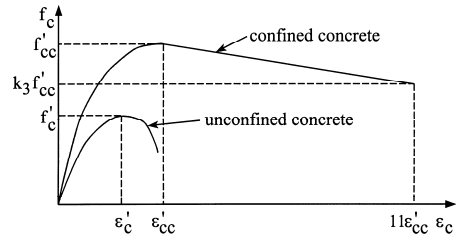


Fig. 5 Equivalent uniaxial stress-strain curve for concrete

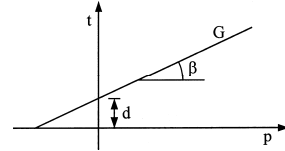


Fig. 6 Linear Drucker-Prager yield criterion for concrete

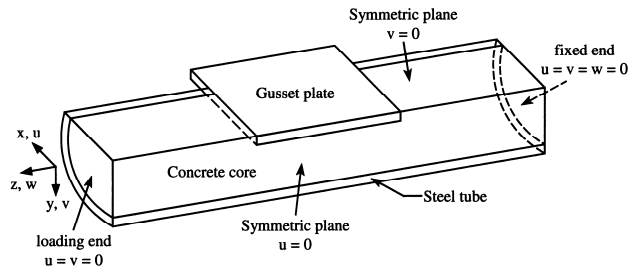


Fig. 7 Conceptual sketch and finite element modeling of CFT-to-bracing connection

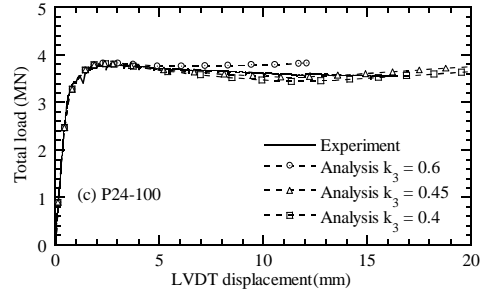
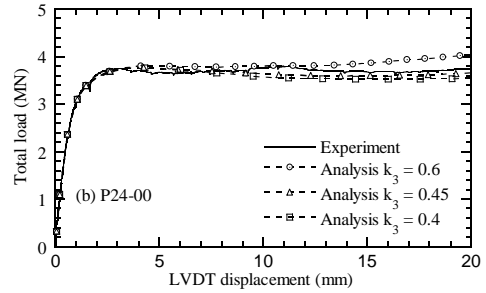
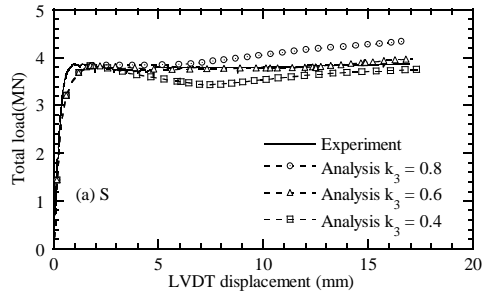


Fig. 8 Simulation of k_3 for specimens S, P24-00 and P24-100 ($k_4 = 0.75$)

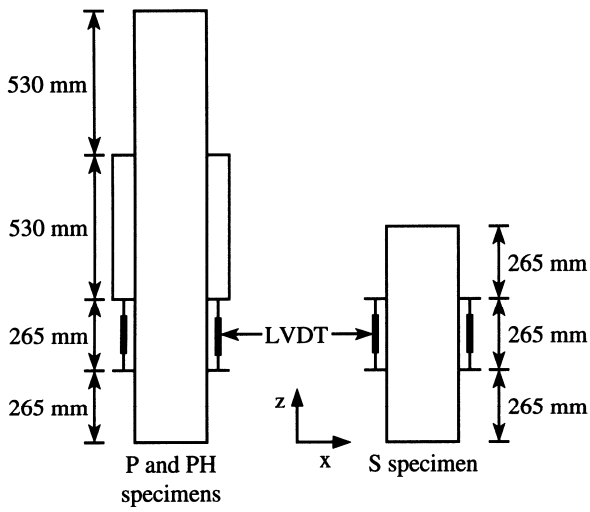


Fig. 9 Positions of LVDT for CFT specimens

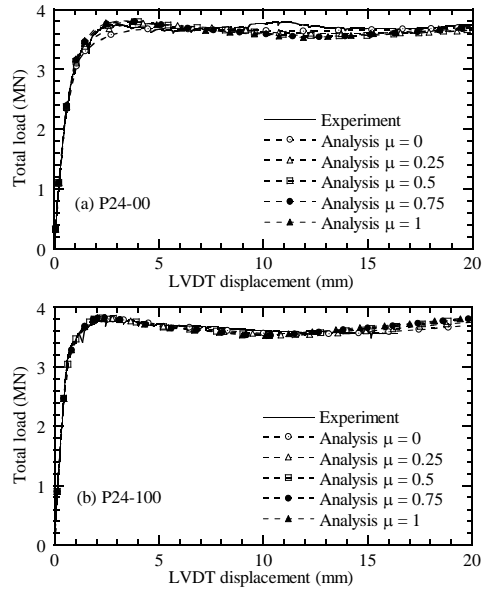


Fig. 11 Influence of the friction coefficient on the behavior of CFT columns

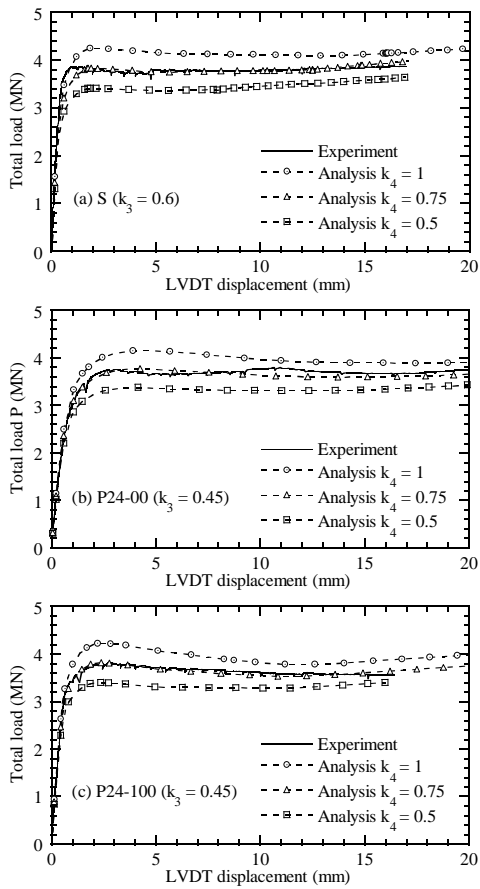


Fig. 10 Simulation of k_4 for specimens S, P24-00 and P24-100

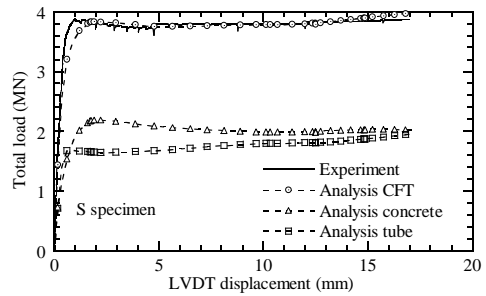


Fig. 12 Load - displacement curves of specimen S

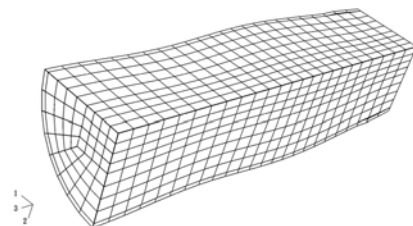


Fig. 13 Deformation shape of specimen S

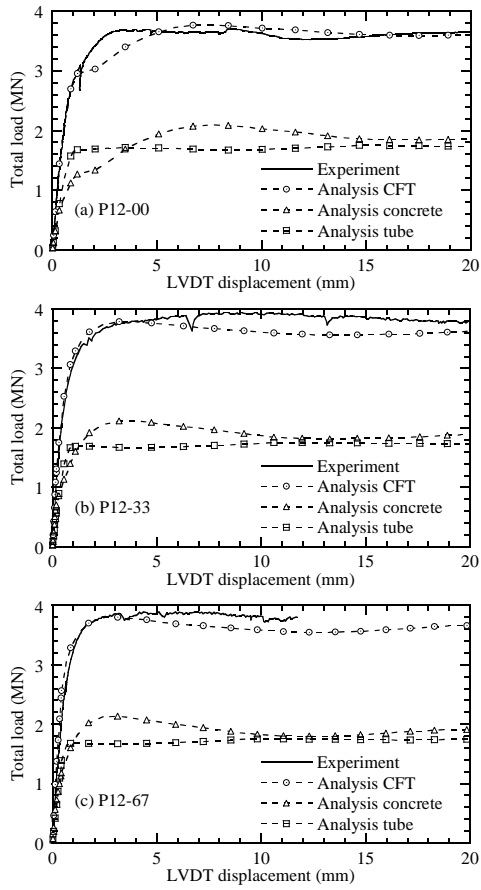


Fig. 14 Load - displacement curves of specimens P12-00, P12-33 and P12-67

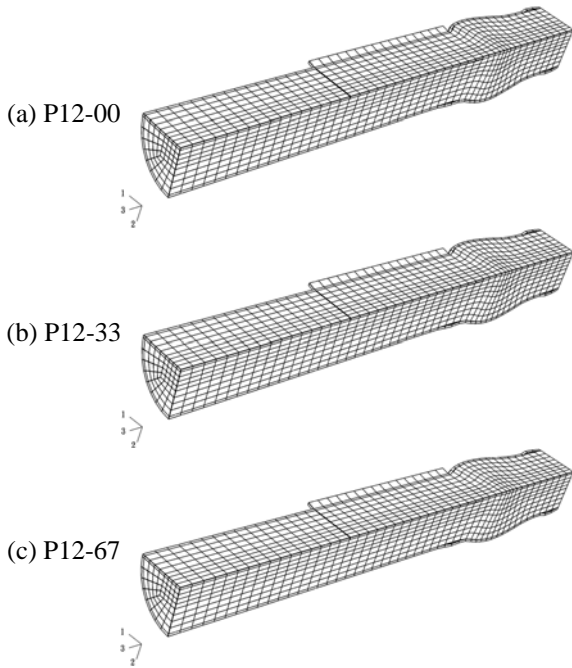


Fig. 15 Deformation shapes of specimens P12-00, P12-33 and P12-67

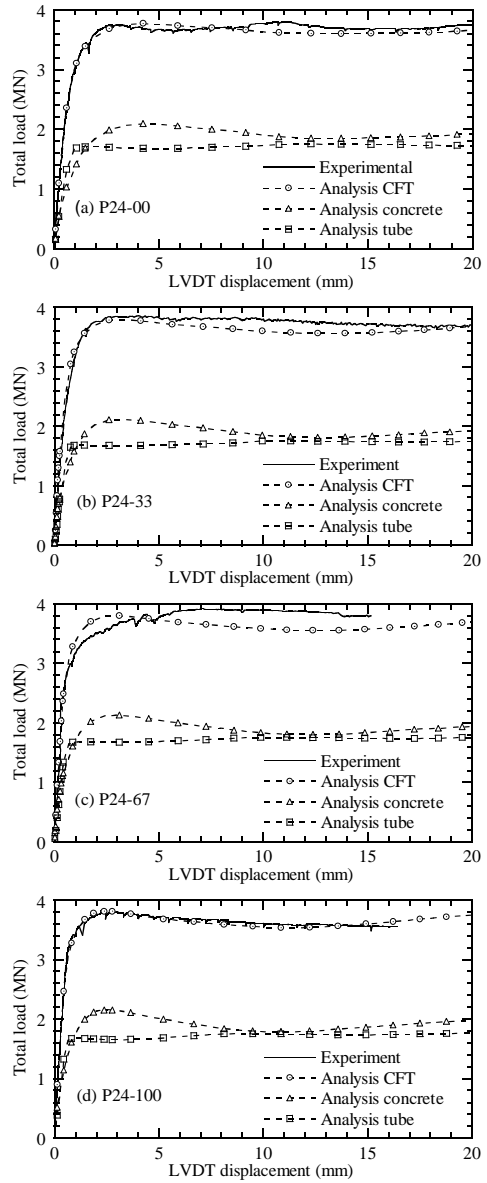


Fig. 16 Load - displacement curves of specimens P24-00, P24-33, P24-67 and P24-100

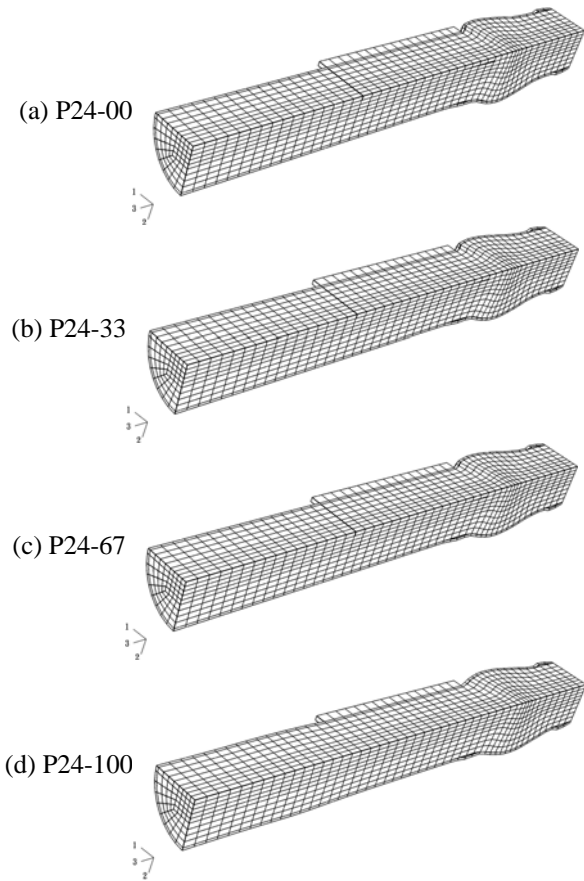


Fig. 17 Deformation shapes of specimens P24-00, P24-33, P24-67 and P24-100

P36-33 and P36-67

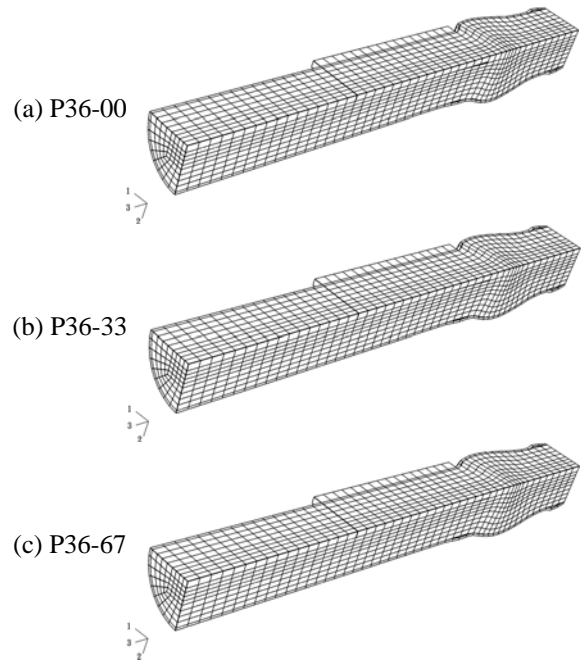


Fig. 19 Deformation shapes of specimens P36-00, P36-33 and P36-67

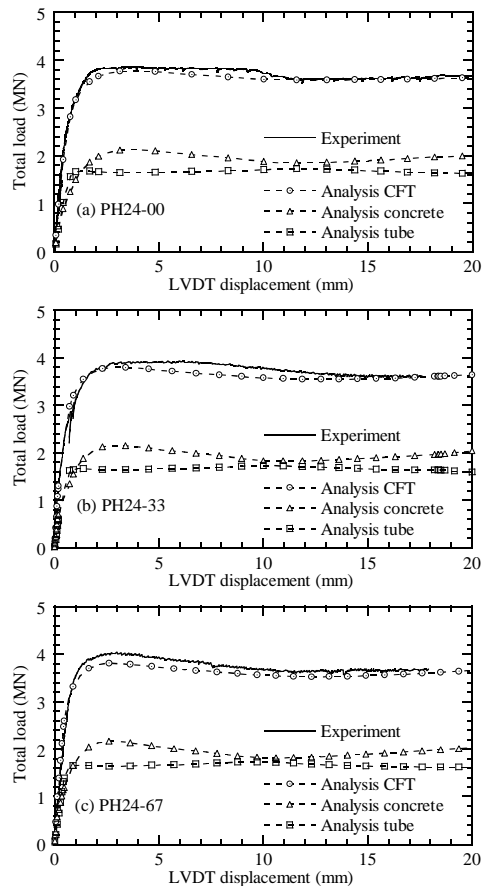
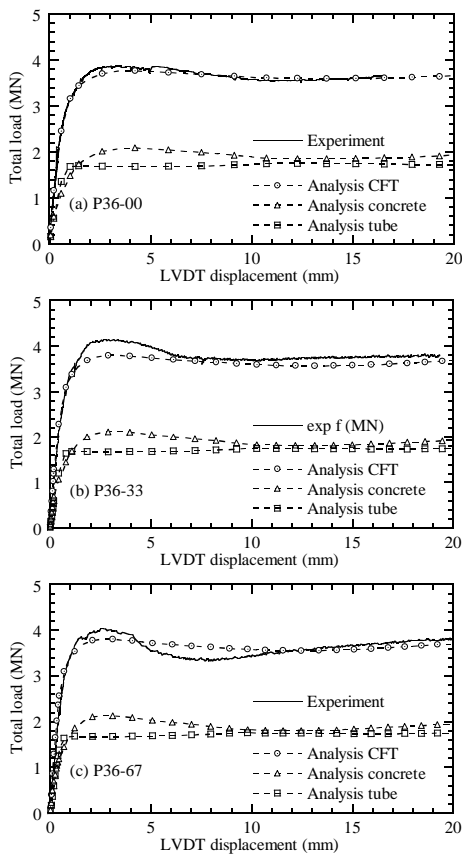


Fig. 18 Load - displacement curves of specimens P36-00,

Fig. 20 Load - displacement curves of specimens PH24-00, PH24-33 and PH24-67

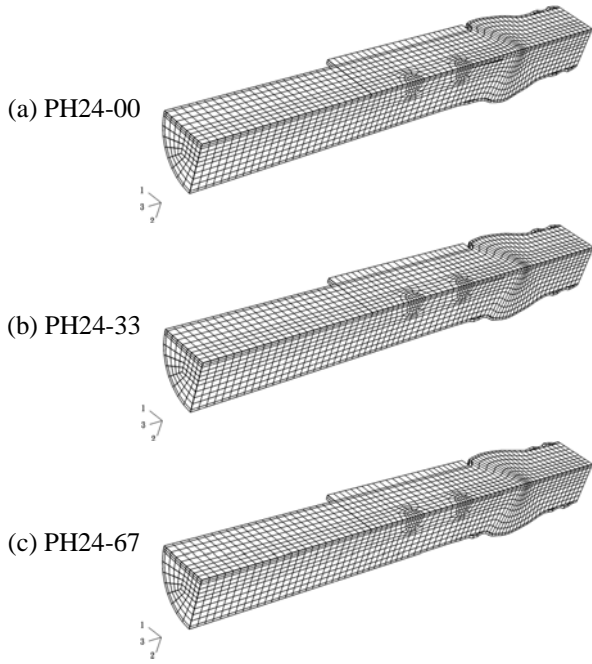


Fig. 21 Deformation shapes of specimens PH24-00, PH24-33 and PH24-67

PH36-00, PH36-33 and PH36-67

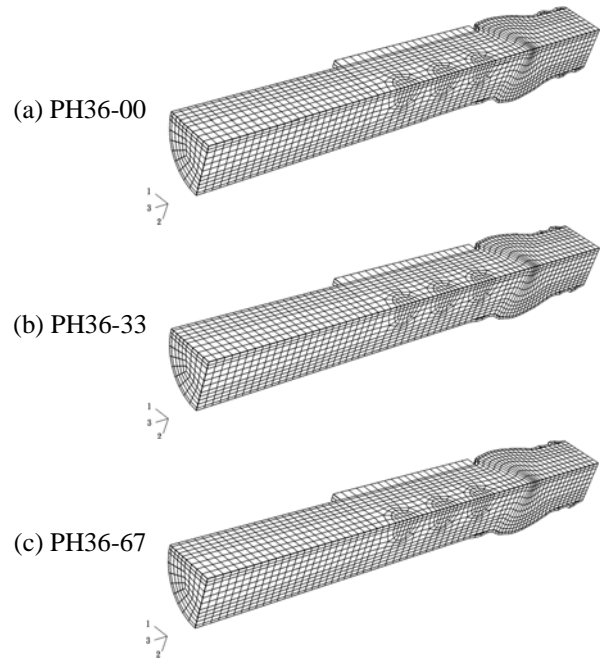


Fig. 23 Deformation shapes of specimens PH36-00, PH36-33 and PH36-67

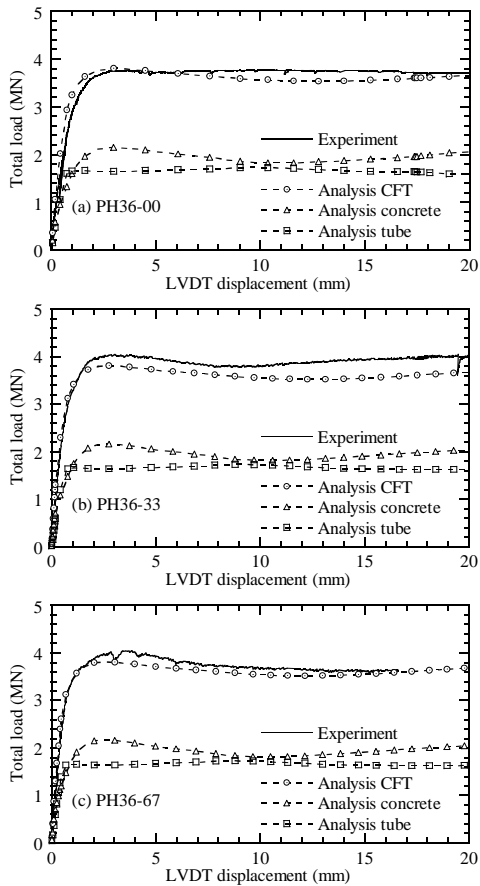


Fig. 22 Load - displacement curves of specimens