EXPERIMENTAL AND ANALYTICAL INVESTIGATIONS OF STEEL AND COMPOSITE TRUSSES

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ABSTRACT: The experimental and analytical investigations on bare and composite rectangular hollow sections (RHS) used as members of trusses are presented in this paper. The load resistances of the trusses consisted of steel and concrete-filled RHS tubes are compared to quantify the beneficial effects due to the in-filled concrete. The maximum loads on the trusses are also computed by the design method in Eurocode 3 (Steel member) and Eurocode 4 (Composite member) and compared with the test results. The results showed that the use of effective length method in linear analysis and design method is less convenient and accurate than the second-order analysis. The second-order analysis and design method not only gives a more accurate prediction than the linear analysis, but it also provides an efficient design as the assumption of effective length is not required to guess.

Keywords: Steel hollow sections, Concrete-filled steel hollow sections, Second-order analysis, Effective length method, Eurocode 3, Eurocode 4

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1. INTRODUCTION

Experimental studies on concrete-filled steel tubes under different end conditions have been extensive conducted and the works include those by Knowles and Park [1], Bridge [2], Shakir-Khalil [3] and Lu and Kennedy [4] who investigated the axial and flexural behavior of the sections. Most experiments were focused on the behavior of single member with ends restrained against lateral movement. In this paper, the end movements of the members were restrained by other connecting truss members. The end movements of the member induce the P-Δ effect its inclusion is important in the analysis and design.

Several commonly used design codes provide different design methods on composite members such as Eurocode 4 [5], BS5400 [6] and CoPHK [7]. These codes contain various design methods for several types of composite columns and these methods include the first order linear analysis and effective length method for member buckling strength check. The accuracy of those design methods depend heavily on the precision of determination of effective length factor which is not quite possible to estimate since the idealized assumption for simple end conditions like pin and rigid ends are unrealistic in most practical structures. In this paper, two effective length factors were used to predict the design load, and the results will be compared with test results.

As an alternative to the first order linear analysis with effective length assumption, the second-order analysis and design method for steel tube and concrete-filled steel tube members is recommended in many design codes such as Eurocode 4 [5] and CoPHK [7] as a preferred design method, especially when the elastic critical load factor is small. In the second-order analysis, the nonlinear effects such as P-δ, P-Δ effects and initial imperfection can be directly included in the analysis, and the estimation of the effective length is no longer required, and the member section capacity can be directly used for buckling strength check without modification factors. The individual member check is replaced by the section capacity check in a single equation in place of the approach requiring the use of several parameters embedded in the few checking equations such as the section capacity and the member buckling checks.

The pointwise equilibrium polynomial (PEP) element would be used in the paper for second-order analysis because of its simplicity and computational stability and efficiency allowing modeling one member by a single element. This modeling convenience reduces significantly the computational time and the process of separating the compressive and the tensile load cases is not needed because the matrix is valid for positive, negative and zero axial force. After modification of the PEP element by Chan and Zhou [8] and Zhou and Chan [9], not only the equivalent initial imperfection, which simulates the effect of geometric imperfection and residual stress, but also the semi-rigid joint at the ends of the member could be included in the PEP element. In the past few years, the second-order analysis and design method has been widely used for quite a few types of structures and the accuracy has been verified in many examples.

2. EXPERIMENTAL PROGRAM

2.1 Specimens

Two trusses were tested and their dimensions are shown in Figure 1. One truss was composed of RHS and square hollow section (SHS) steel tubes in all members and another truss was composed of concrete-filled RHS and SHS steel tube in compression members and SHS bare steel tube in tension members. Each three-dimensional truss consisted of 19 members which included two 50x30x3 RHS tubes and seventeen 60x60x3 SHS tubes. The two target failure members were 50x30x3 RHS tube and the remaining members were 60x60x3 SHS tube and the deflections of these target members are measured with full attention. The length of each truss member was 2m approximately and the tie members connecting the two plane trusses were 0.8m approximately. The ends of the members were connected rigidly by using 8mm butt weld.

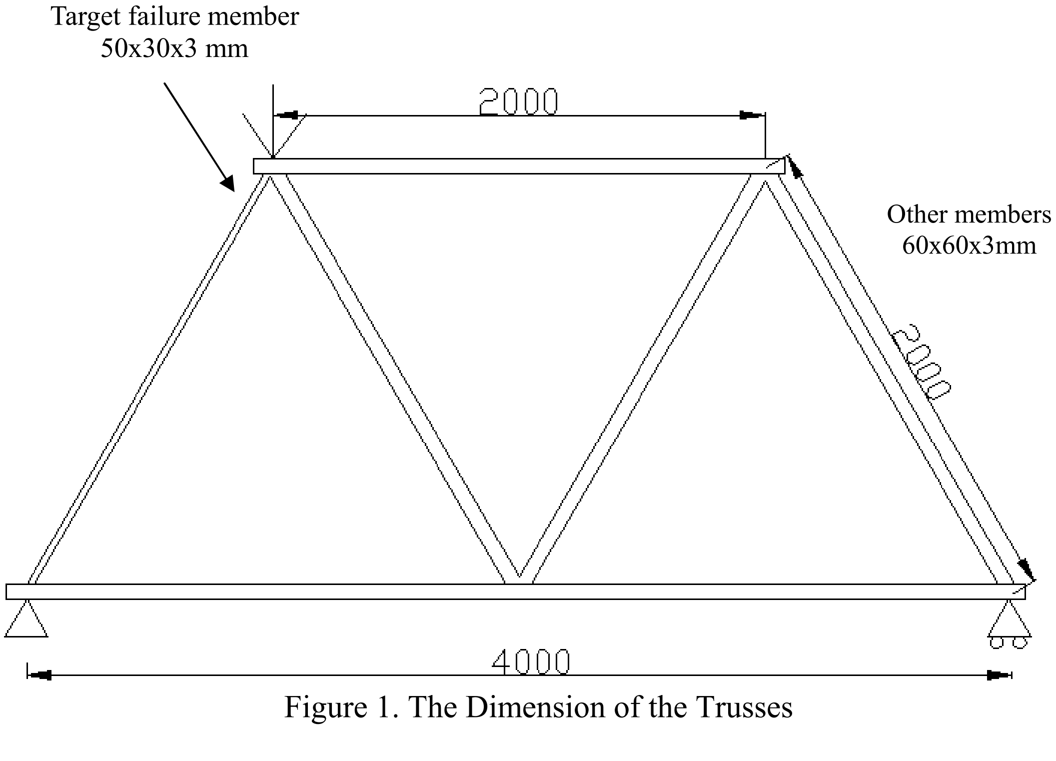

Figure 1. The Dimension of the Trusses

The average width, depth and thickness of both sections are listed in Table 1. The coupon test was carried out to determine the stress-strain curve of the steel section. The average yield stress (f_y), ultimate tensile stress (f_u) and the Young's modulus (E_s) are summarized in Table 1.

The high strength concrete was used in filling in the steel tubes The composition of concrete mix was water (238.1kg/m^3), Ordinary Portland Cement (479.5kg/m^3), coarse aggregate (862.5kg/m^3), fine aggregate (709kg/m^3) and Pulverized Fly Ash (205.5kg/m^3). The average compressive cube and cylinder strength were 91.65N/mm^2 and 89.87N/mm^2 respectively and the modulus of elasticity of concrete was 37.45kN/mm^2.

Table1. Material Properties of RHS Tube

Steel section	B (mm)	D (mm)	t (mm)	Yield stress (f_y) N/mm^2	Ultimate tensile stress (f_u) N/mm^2	Young's modulus (E_s) kN/mm^2
50x30x3	50.00	30.08	2.96	399.17	448.30	203.87
60x60x3	60.58	60.53	3.25	376.12	439.91	217.50

2.2. Test Results

The trusses were simply supported at the two ends and loaded by the hydraulic jack of capacity 400kN and placed between the pair of trusses as shown in Figure 1. Totally 12 displacement transducers were placed at the loading point and the top, bottom and middle of the target failure member to measure the deflection of the member and deflection of the truss. 18 strain gauges were placed at 3 locations at the top, middle and bottom of each target failure member and six strain gauges were mounted at each location. The detailed locations of displacement transducers and strain gauges are shown in Figure 2.

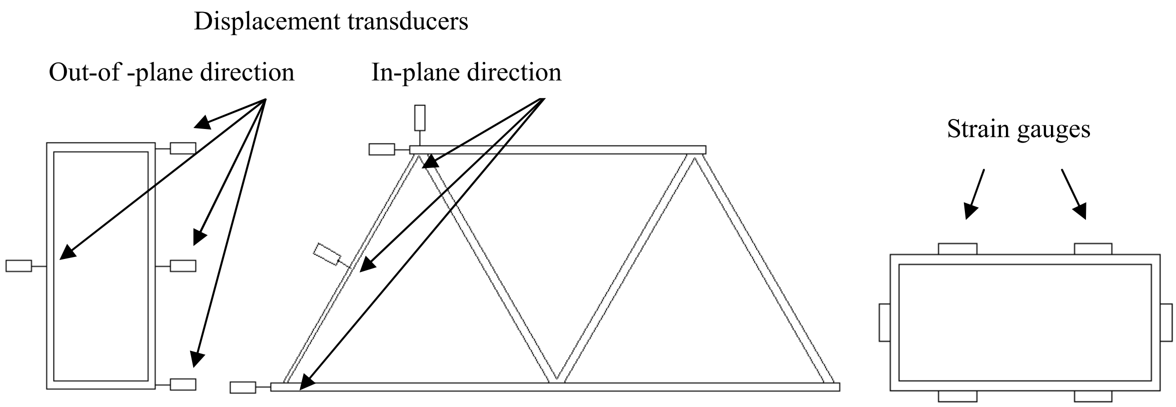

Figure 2. Displacement Transducers and Strain Gauges Location

The test results for both steel and concrete-filled steel tube are given in Table 2. The applied load at the top of the truss against mid-span in-plane deflection (the component normal to the truss member) of the failure members are plotted in Figure 3. For bare steel tube, the member deflection increased linearly with the applied load until it reached 36.85kN, and the load-deflection relationship then became non-linear. The maximum applied load on the steel truss was 76.61kN. For concrete-filled steel tube, the member deflection under applied load was similar to the steel tube in which linear relationship was observed before the applied load reaching 26.20kN. After this load, the deflection increased with applied force nonlinearly and the maximum applied load was found to be 90.00kN. The flexural buckling about the principal minor axis of the failure member took place and shown in Figures 4 and 5 for both trusses. The internet links of the videos are shown below.
http://www.nida-naf.com/index.php?option=com_content&view=article&id=121%3 .

Table2. Test Results

Truss member	Maximum Applied Force (kN) P_t
Steel Tube	76.61
Concrete-filled Steel Tube	90.00

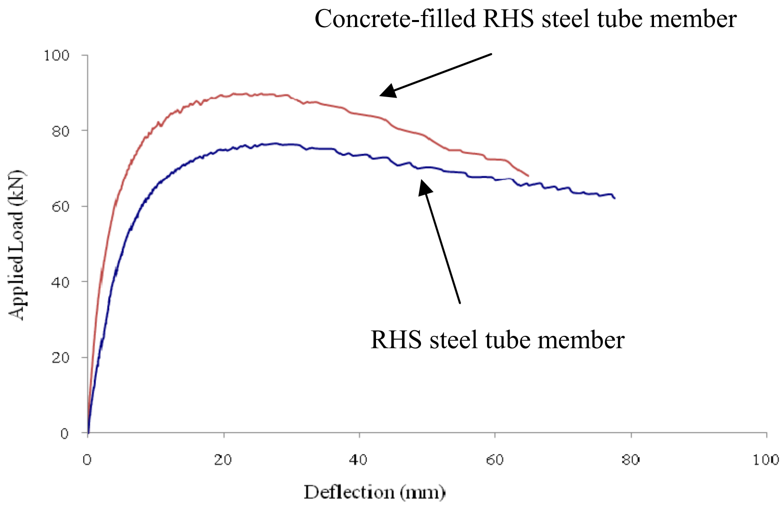

Figure 3. Load against in-plane Deflection of Failure Member

Figure 4. The Failure Shape of Steel Member Truss

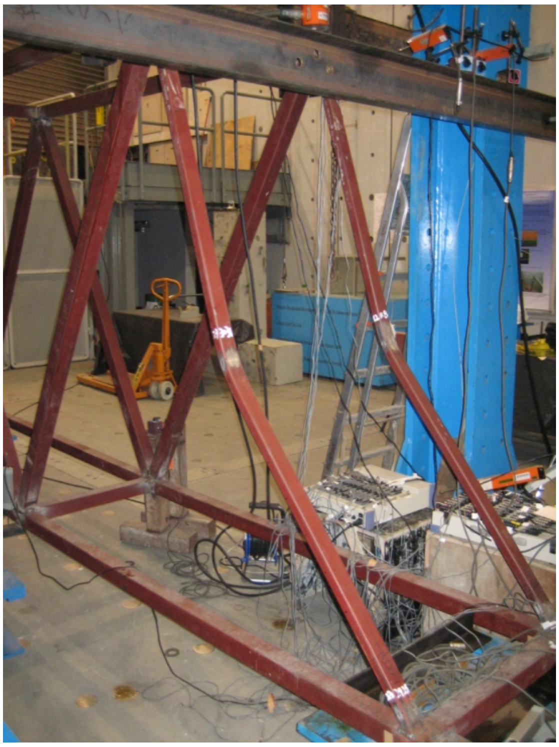

Figure 5. The Failure Shape of Composite Member Truss

The maximum load, which was taken by the member in composite truss, was 17.5% higher than the bare steel truss. The applied load against out-of-plane mid-span deflection of the failure members is also plotted in Figure 6. The curves showed that the out-of-plane deflection was small compared with the in-plane deflection at maximum applied load in both trusses.

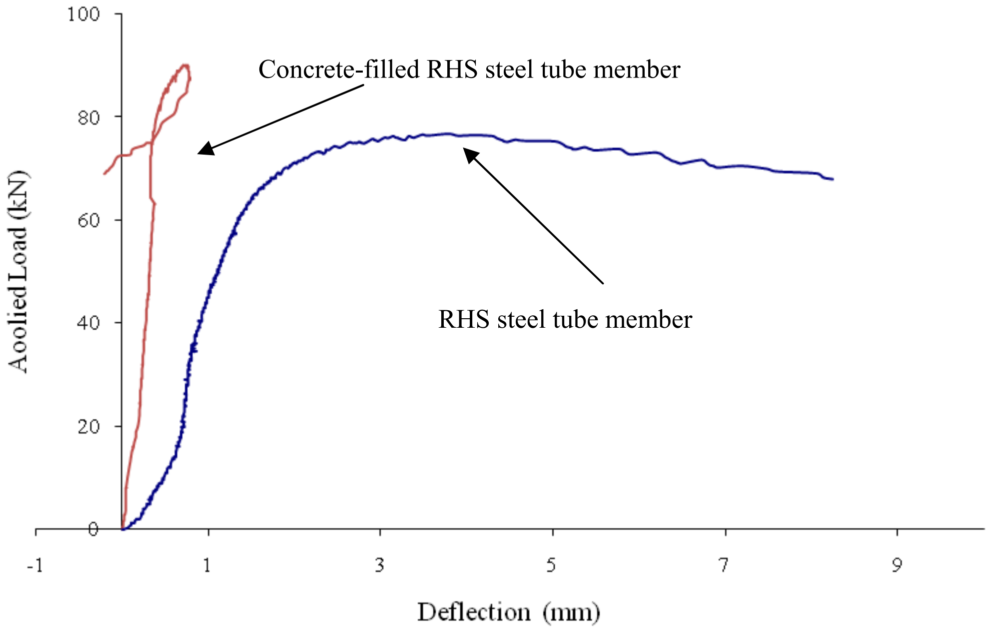

Figure 6. Load against Out-of-plane Deflection of Failure Member

Applied load against the strain plots at the mid-length of the failure members are shown in Figure 7. As expected, non-linear relationship was observed and the post-failure behaviour could be obverved in both RHS steel and concrete-filled RHS steel tubes. The RHS steel and concrete-filled RHS steel tube gave similar behaviour in strain as shown in Figures 7a and 7b. The variation on the strain along the top fiber (SG1 and SG2) and along the bottom fiber (SG4 and SG5) is small, hence the average strain in top and bottom was plotted against the applied load and large compressive strains was developed at bottom fiber which gave a consistent result with displacement transducers. The readings from SG6 and SG3 were identical up and close to the failure load for concrete-filled RHS steel tube and these readings are slightly different in RHS steel tube, due to the out-of straightness imperfections in major axis direction. The result implied that the out-of plane deflection was insignificant before failure load after which the out-of plane deflection increased significantly with decreasing load, hence the load-strain curve started to diverge.

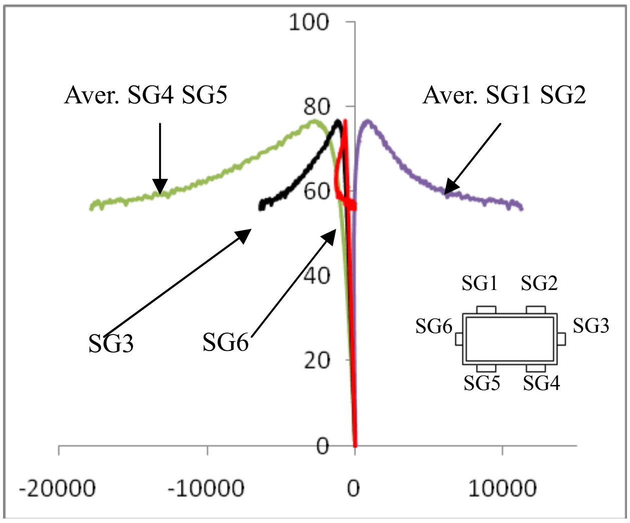

Figure 7a. Steel RHS Tube Member

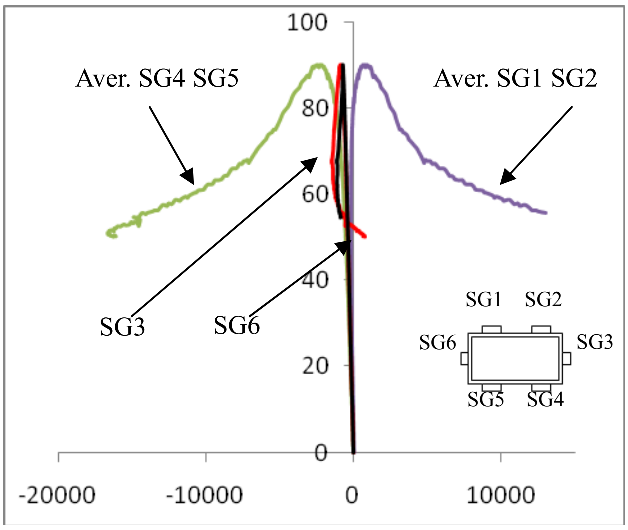

Figure 7b. Concrete-filled RHS Steel Tube Member

Figure 7. Load-Strain Cruve for the Failure Member

3. CODE AND SECOND-ORDER ANALYSIS AND DESIGN METHOD

3.1 Predicted Results from Eurocode 3 and Eurocode 4

The design method in Eurocode 4 [5] for concrete-filled steel tubular columns is briefly described here for clarity and completeness. The section capacity of concrete-filled steel tubular members is determined by simply adding the capacity of two components of concrete and steel tubes. The reduction factor χ, which is a function of the effective slenderness ratio and section type, is multiplied to the section capacity to consider the stability. Thus, the member resistance is obtained as,

$$P_{cp} = \chi(A_s f_{yd} + A_c f_{cd}) \tag{1}$$

in which A_s , A_c, f_{yd} and f_{cd} are the cross-sectional area and design strength of the steel and concrete respectively.

The reduction factor χ is given by

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \overline{\lambda}^2}} \qquad (2)$$

and

$$\phi = \frac{1}{2}\left[1 + \alpha(\overline{\lambda} - 0.2) + \overline{\lambda}^2\right] \qquad (3)$$

in which α is the imperfection factor and $\overline{\lambda}$ is the relative slenderness.

The predicted results using the design methods in Eurocode 3 [10] and Eurocode 4 [5] for RHS steel and concrete-filled RHS steel tube are summarized in Table 3. The effective length factor equal to 0.5 (L_e=1000) and 1.0 (L_e=2000) were assumed to simulate the fix end with fixed and free translations which are the upper and lower conditions to the actual behavior of the members in the truss. Predicted maximum applied loads according to the design codes were 31.77kN and 106.14kN for RHS steel tube, and 36.10kN and 130.49kN for concrete-filled RHS steel tube under these two different effective length factors. The ratios of tested to predicted load were about 0.7 and 2.4 for effective length factors (L_e/L) equal to 0.5 and 1.0. The results indicate that the true effective length of the members should be between these two values.

Table 3. Predicted Results by Design Codes

Truss member	Maximum Applied Force (kN)			Test Load/Predicted Load	
	Test result P_t	Results by Eurocode 3 & Eurocode 4 P_{ec}		P_t/P_{ec}	P_t/P_{ec}
		L_e=1000	L_e=2000	L_e=1000	L_e=2000
Bare Steel Tube	76.61	106.14	31.77	0.72	2.41
Concrete-filled Steel Tube	90.00	130.49	36.91	0.69	2.44

3.2 Second-order Design and Analysis Method

The use of second-order analysis and design method has been widely adopted for different types of structures because this analysis and design method not only simplifies the design process, but also gives an accurate result. The non-linear effects, which including member imperfection, P-δ and P-Δ second-order moments, are included in analysis and hence, the uncertain design process demonstrated in the last section requiring determination of effective length and buckling reduction factors is not required. The formulation of the element tangent stiffness and secant stiffness matrix for steel and composite members have been detailed by Chan and Zhou [11, 12] and Chan et al. [13] and will not be repeated here.

3.3 Section Capacity Check

In the second-order analysis and design method, the section capacity check equation is used. For steel member, the equation below is adopted.

$$\frac{P}{P_p}+\frac{M_y+P(\delta_y+\Delta_y)}{M_{py}}+\frac{M_z+P(\delta_z+\Delta_z)}{M_{pz}}=\phi\le 1 \quad (4)$$

For composite member, two section capacity equations are used for the two load conditions. When the applied force is larger than the section capacity of concrete section (i.e. $P>P_{pm}$), Eq. 5 will be used and it accounts for the effects of axial force and moments in the section capacity equation. When the applied force is not greater than the capacity of concrete section (i.e. $P\le P_{pm}$), only applied moments are considered since the axial force does not reduce the failure load and Eq. 6 is then used for section capacity check. These two sets of section capacity equations are given as follows.

For $P>P_{pm}$

$$\frac{P-P_{pm}}{P_{cp}-P_{pm}}+\frac{M_y+P(\delta_y+\Delta_y)}{M_{cpy}}+\frac{M_z+P(\delta_z+\Delta_z)}{M_{cpz}}=\phi\le 1 \quad (5)$$

For $P\le P_{pm}$

$$\frac{M_y+P(\delta_y+\Delta_y)}{M_{cpy}}+\frac{M_z+P(\delta_z+\Delta_z)}{M_{cpz}}=\phi\le 1 \quad (6)$$

in which P is the applied force, P_p, P_{pm}, P_{cp} is compressive capacities of steel, concrete and composite cross-section, M_y and M_z are the external moments about the y and z axis, $P(\delta_y+\Delta_y)$ and $P(\delta_z+\Delta_z)$ are the P-δ and P-Δ moments about the y and z axes, M_{py}, M_{cpy}, and M_{pz}, M_{cpz} are the moment capacities of composite cross-section about the y and z axes.

As shown in the section capacity check equations that the P-Δ and P-δ effects have been included such that the assumption of effective length is no longer required. Further, the inclusion of initial imperfection has been directly considered in analysis that the concept of section capacity check for imperfect columns can be applied directly in the integrated analysis and design model.

3.4 Numerical Procedure

The load control Newton Raphson method combined with the minimum residual displacement method [14] is used and the method is capable of tracing the path up to and beyond the limit point without numerical divergence.

3.5 Analysis Results

The analytical model and the deformed shape of the truss are shown in Figure 8. The average yield stress and Young's Modulus of steel and concrete from tested material were used in computer model. The initial imperfection of the member was taken as $L/300$, where L is the member length, according to Table 5.1 in Eurocode 3 [10] and Table 6.5 in Eurocode 4 [5] for steel and composite columns respectively. The center-to-center member length was used and rigid connection between each member was assumed. Two point loads were applied to the top of the truss on each side and load increment of 0.05kN was used in analysis until the section capacity factor was equal to 1. The analysis results were presented together with test results in Table 4. The failure loads of steel and composite truss were 77.80kN and 90.35kN respectively and the ratios of test to analysis result are 0.98 and 1.0 for RHS steel and concrete-filled RHS steel tube members. The analysis results show that the second-order analysis gives accurate results on prediction of resistance of the bare steel and concrete-filled steel tube members in a consistent manner.

Table 4. Predicted Results by Second-order Analysis and Design Method

Truss member	Maximum Applied Force (kN)		Test Load / Predicted Load
	Test result P_t	Second-order analysis and design method P_a	P_t/P_a
Bare Steel Tube	76.61	77.80	0.98
Concrete-filled Steel Tube	90.00	90.35	1.00

4. CONCLUSIONS

Experimental investigation on behavior of RHS steel and concrete-filled RHS steel tubes used as members in a truss was presented in the paper. The load capacity of the concrete-filled RHS steel tube member is 17.5% higher than the RHS steel tube member. The results by the Eurocode 3 [10] and Eurocode 4 [5] show that, with the assumption of effective length factor to be 0.5, the code over-estimates the resistance of the failure member which leads to an unsafe design. On the other hand, when the assumption of the effective length factor as 1.0 is adopted, the code under-estimates the member resistance and the design is uneconomical. The second-order analysis and design method without assumption of effective length with non-linear buckling effects are directly included in analysis gives results much closer to the test results and this indicates clearly the superior performance of the second-order analysis for design of trusses made of steel and composite RHS sections. When dealing with the design of practical steel structures, the second-order analysis further improves the efficiency by skipping time for approximating a correct effective length for each member under different load case.

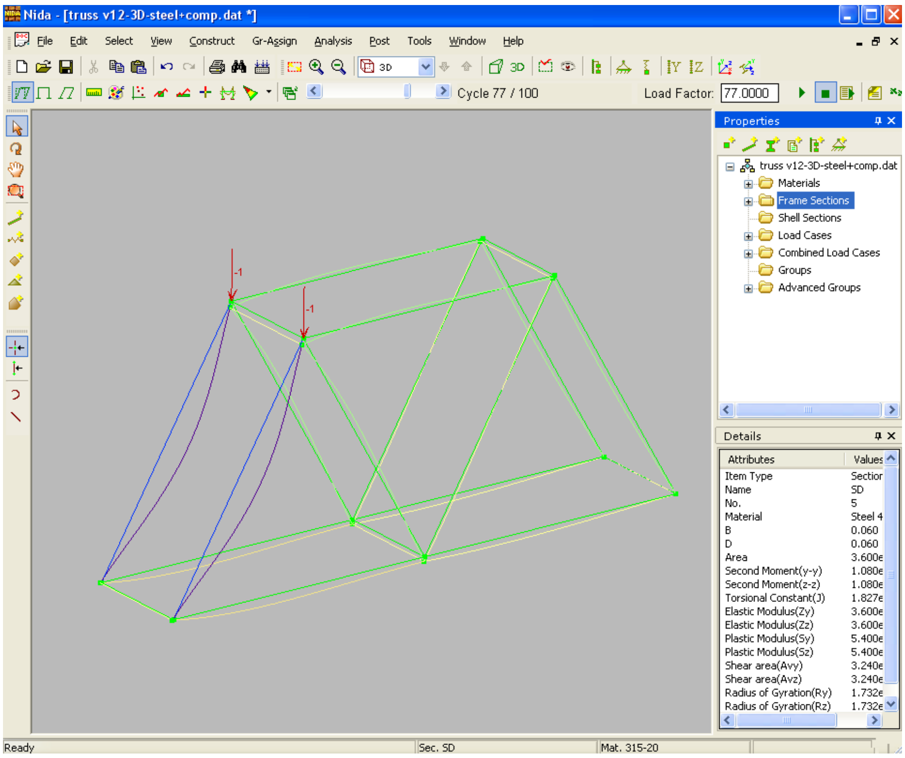

Figure 8. The Analytical Model of the Truss

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