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An investigation into structural behaviour of modular steel scaffolds

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Abstract. This paper presents a study on the structural behaviour of modular steel scaffolds through both experimental and numerical investigations. Three one-storey and three two-storey modular steel scaffolds were built and tested to failure in order to examine the structural behaviour of typical modular steel scaffolds. Details of the tests and their test results were presented in this paper. Moreover, an advanced non-linear analysis method was employed to evaluate the load carrying capacities of these scaffolds under different support conditions. Comparisons between the experimental and the numerical results on the structural behaviour of these modular steel scaffolds were also presented. Moreover, the restraining effects of external supports in practical situations were also studied through finite element methods. The predicted load carrying capacities and deformations at failure of these models under partially restrained conditions were found to be close to the experimental results. A codified design method for column buckling with modified slenderness ratios was adopted for practical design of modular steel scaffolds.

Key words: modular steel scaffolds; structural instability; partially restrained supports; modified slenderness ratios; column buckling design.

1. Introduction

Modular steel scaffolds are temporary structures and commonly used as supporting scaffolds in building construction. The modular units are typically fabricated from slender members made from high strength cold-formed steel tubes. The advantages of modular steel scaffolds are easy fabrication, installation and dismantling. However, there are a significant number of collapses involved with these scaffolds from time to time due to inadequate design against axial buckling, poor workmanship with insufficient bracing, and over-loads on sites. Failure of supporting scaffolds causes not only work delays, but also injuries and casualties. The failure cases of modular steel scaffolds were studied in the past and the most of collapses were found to be occurred during construction (Peng *et al*. 1996).

Some manufacturers may provide safe load carrying capacities of modular steel scaffolds according to their test data on scaffolding frames. A number of researchers executed systematic experiments to measure the load carrying capacities of modular steel scaffolds (Peng *et al*. 1997, Weesner and Jones 2001). In order to study the structural behaviour of these scaffolds, advanced structural analysis programs were often employed to carry numerical analyses on modular steel scaffolds using different approaches.

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1.1. Modular steel scaffolds

In order to erect modular steel scaffolds efficiently in building construction, different types of modular steel scaffolds are available in the market with various typical member configurations. In practice, door type modular steel scaffolds have been widely used in Japan as temporary supporting scaffolds, and these modular steel scaffolds are now commonly used throughout South East Asia. It is interesting to note that there is an extensive research program on structural bamboo (Chung and Yu 2002) and bamboo scaffolds (Yu *et al*. 2003) which are commonly used as assess scaffolds in Asia, in particular, in Hong Kong and the Southern China. The structural instability of bamboo members and bamboo scaffolds was studied extensively to provide practical design rules to assess their load carrying capacities in practical applications.

In order to study the structural behaviour of the joints between steel scaffolding frames, spiral spring connections are often used in finite element models (Peng 1994), and it is necessary to evaluate the stiffness of these spiral springs from experiments. For simplicity, the joints between steel scaffolds may be considered as continuous joints, and thus, rigid connections between steel scaffolds are adopted in the finite element models. Moreover, wooden shores or steel shores at the top of modular steel scaffolds are highly susceptible to buckle as they are not designed to resist moment. The instability of wooden shores (Peng 1994) is described in details by Peng and thus not included in this study. Experimental investigations (Chung and Lau 1999, Wong and Chung 2002, Chung and Lawson 2000) on semi-rigid connections are also reported in the literature.

1.2. Stability analysis

Modular steel scaffolds are typically defined as slender structures with significant instability problems, thus second-order analysis is often required to investigate their structural behaviour. A structural analysis software GMNAF was used to perform a three-dimensional second-order elastic analysis of steel scaffolds (Peng *et al*. 1997). Furthermore, an advanced non-linear analysis program NIDA (Chan and Zhou 1998) using one-element-per-member formulation was also employed to evaluate the load carrying capacities of modular steel scaffolds. The method has been extended to elasto-plastic nonlinear analysis of slender frames (Zhou and Chan 2004, Chan and Zhou 2004), trusses (Chan *et al*. 2002) and pre-stressed truss (Chan *et al*. 2002). The predicted failure loads were the applied loads at first yield of column members in the presence of initial geometrical imperfection. Another commercial finite element program ANSYS was also employed to predict the elastic buckling loads of modular steel scaffolds (Weesner and Jones 2001). It is interesting to note that extensive numerical investigations on steel connections (Chung and Ip 2000, 2001) are also reported using ANSYS.

In general, the effects of initial geometrical imperfections are considered to be important to the structural instability of modular steel scaffolds, and the initial geometrical imperfections of the modular steel scaffolds may be simulated by applying a horizontal force equal to 0.1% of the vertical applied loads at the mid-height of the scaffolds (Peng *et al*. 1997). However, it is somehow difficult to justify the magnitudes of the notional forces and also the locations of application of notional forces. Therefore, the initial geometrical imperfection of the modular steel scaffolds was conveniently assumed to take up the first eigenmodes in this study, and the magnitude of the out-of-straightness is taken as 0.001 of the height of scaffolding frames.

Researchers may have different interpretations of the boundary conditions at both the top and the bottom of the scaffolds. It is argued that the boundary conditions should be considered as pinned at the bottom due to the extension of screw jacks, but certain rotational restraints are provided by jack bases when the modular steel scaffolds are loaded. In practice, it is difficult to quantify the restraining effects at both the top and the bottom of the scaffolds, and thus the advanced non-linear finite element model is employed to evaluate the load carrying capacities of these scaffolds under different support conditions.

2. Objectives

At present, there is little design guidance available in the literature on the design of modular steel scaffolds. In order to minimize work hazards, a proper design of modular steel scaffolds is required to establish their structural adequacy. This paper aims to investigate the structural behaviour of these modular steel scaffolds through both experimental and numerical investigations. Three one-storey and three two-storey modular steel scaffolds were built and tested to failure in order to provide data for comparison.

An advanced non-linear analysis method was adopted to evaluate the load carrying capacities of modular steel scaffolds under different support conditions. The finite element models with partially restrained conditions at both the top and the bottom of modular steel scaffolds were analyzed to investigate the restraining effects offered from supports. Practical values of positional restraints at the top and rotational restraints at the bottom were evaluated. A design method for column buckling with modified slenderness ratios for practical applications was also suggested.

3. Experimental investigations

In order to examine the structural behaviour of modular steel scaffolds, six full-scale tests were executed in order to establish their buckling resistances against axial compression. Three one-storey one-bay modular steel scaffolds (MSS1) and three two-storey one-bay modular steel scaffolds (MSS2) with similar geometrical dimensions were tested, and each of the test specimens comprised of scaffolding frames of 1930 mm high and 1219 mm wide metal scaffolds. All the scaffolding frames were positioned at 1819 mm apart with cross-bracings at two planes, as shown in Fig. 1. The specimens were mounted onto a loading frame and set in a vertical position. Both the jack extensions at the bases and the U-heads were adjusted so that the overall heights of the test specimens were 2025 and 3985 mm for MSS1 and MSS2 respectively. An axial compression load was applied progressively with a 50 ton hydraulic jack until unloading occurs. It should be noted that the loading attachments have provided a considerable level of positional restraint to the test specimens during loading.

It should be noted that at failure, significant lateral displacements of the test specimens were observed in the plane of the bracing members. Typical deflected shapes of both MSS1 and MSS2 at failure are shown in Fig. 2, and the vertical members buckled in single and double curvatures respectively. The applied loads and the vertical deflections were measured continuously during the tests till unloading occurred. Both the dimensions and the maximum applied loads at failure of the test specimens are summarized in Table 1 together with the measured yield strengths of the buckled members obtained from coupon tests. It was found that the average failure load per leg are 63.4 kN and 53.4 kN for MSS1 and MSS2 respectively. The resistance ratios which are defined as the ratios of the load carrying capacities of modular steel scaffolds against buckling to their full capacities are found to be 0.46 and 0.39 for MSS1 and MSS2 respectively.

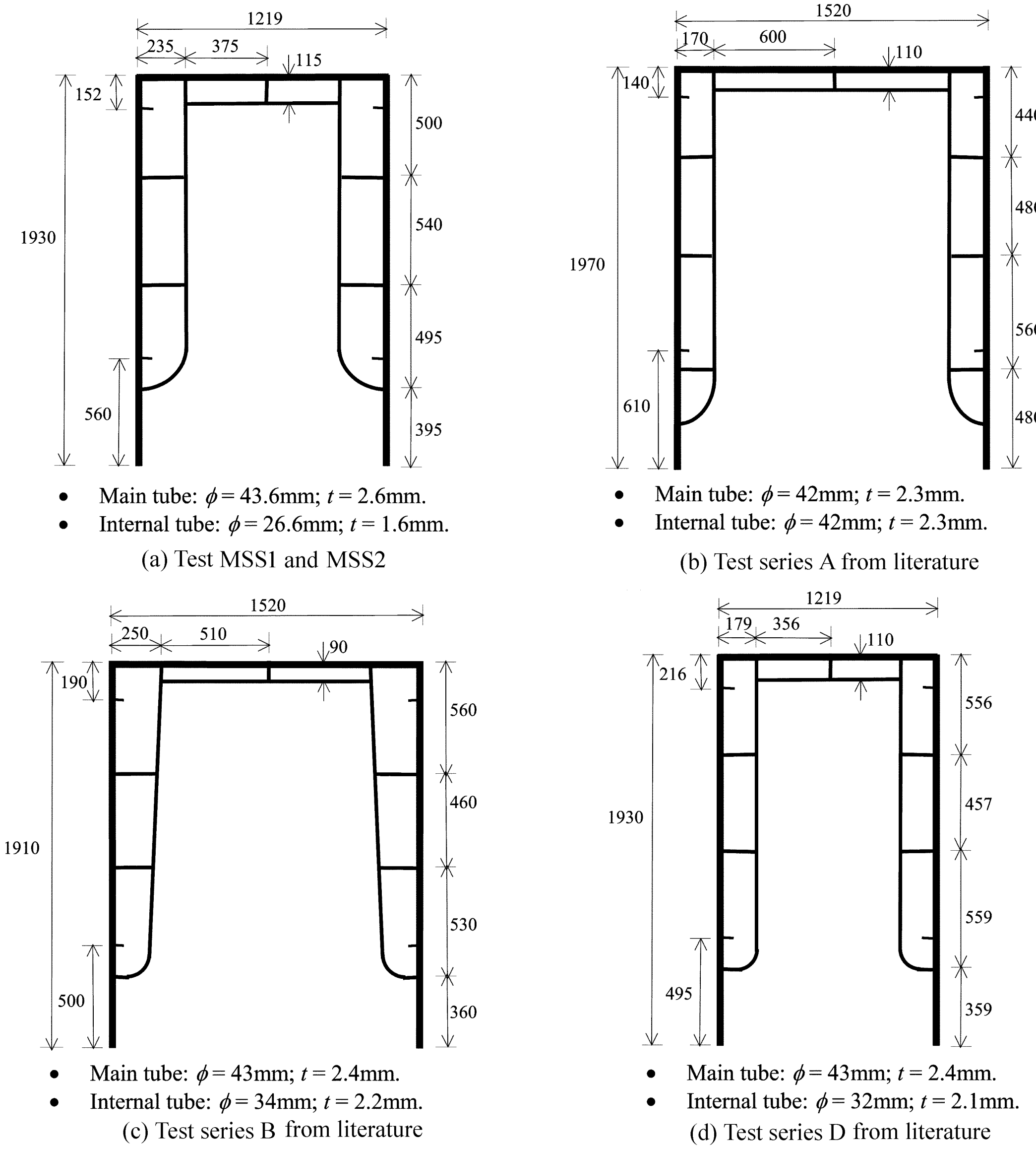
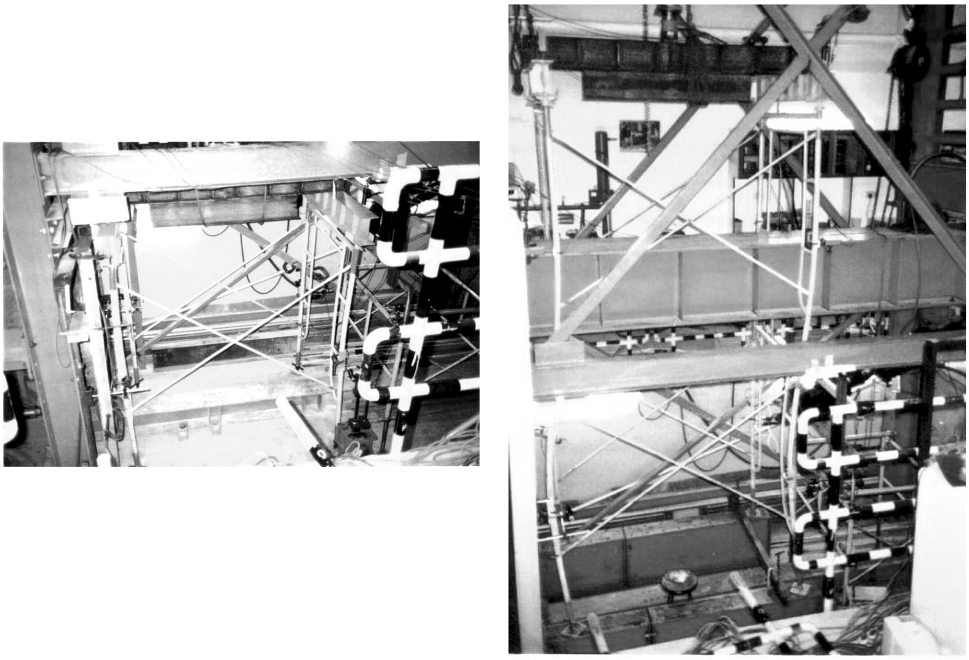


Fig. 1 Geometry of typical scaffolding frames in various test series

4. Numerical investigations

Finite element models were established using the computer software NIDA (Yu *et al.* 2002, Chung and Yu 2003, Yu *et al*. 2004) to investigate the structural behaviour of these modular steel scaffolds. In the present study, initial geometrical imperfections in the form of the first eigenmodes of these scaffolds were provided in the models in order to provide realistic evaluation on the load carrying capacities of modular steel scaffolds. The magnitudes of the maximum out-of-straightness were assigned as 0.001 of the height of the modular units. It should be noted that the connection joints between the modular units were assumed to be rigid in the finite element models. The predicted failure loads were the applied



(a) Deflected shape of MSS1 at failure (b) Deflected shape of MSS2 at failure

Fig. 2 Testing of modular steel scaffolds

Table 1 Test results

Test		Diameter D (mm)	Thickness t (mm)	Area A (mm^2)	Maximum applied load per leg P_{test} (kN)	Maximum compressive strength p_c (N/mm^2)	Measured yield strength p_y (N/mm^2)	Resistance ratio p_c / p_y	Average
	1	43.20	2.63	335.2	60.6	181	412	0.44	
MSS1	2	43.30	2.63	336.0	63.5	189	399	0.47	0.46
	3	43.35	2.76	351.9	66.1	189	409	0.46	
	1	43.18	2.67	339.8	48.7	143	380	0.38	
MSS2	2	43.49	2.87	366.2	55.4	151	377	0.40	0.39
	3	43.25	3.26	409.6	56.2	137	344	0.40	

loads at first yield of the column members in the presence of initial geometrical imperfection. For details of the finite element formulation, refer to Chan and Zhou (1998).

The nominal external diameter and the nominal thickness of the steel tubes are 43.6 and 2.6 mm respectively. As high strength steel tubes are commonly used in practice for modular steel scaffolds, the actual yield strength of steel tubes is adopted to be 406 N/mm^2 in the finite element models while the Youngs modulus is assumed to be 205 kN/mm^2.

4.1. Support conditions

In order to investigate the load carrying capacities of MSS1 and MSS2 under test conditions, the finite element models with different support conditions were analyzed. It should be noted that the

Table 2 Numerical results with various support conditions

Support condition	Model	Failure load per leg P_{NIDA} (kN)	Resistance ratio p_c / p_y	Effective length coefficient k_e
Pinned-Fixed	MSS1	69.7	0.51	**0.93**
	MSS2	62.8	0.46	**1.01**
	MSS3	46.0	0.34	**1.25**
Pinned-Pinned	MSS1	66.3	0.49	**0.97**
	MSS2	38.7	0.29	**1.39**
	MSS3	41.7	0.31	**1.33**
Free-Fixed	MSS1	54.8	0.40	**1.11**
	MSS2	35.0	0.26	**1.48**
	MSS3	32.2	0.24	**1.55**
Free-Pinned	MSS1	32.5	0.24	**1.54**
	MSS2	31.5	0.23	**1.57**
	MSS3	31.1	0.23	**1.58**

Notes: p_y is measured yield strength and equal to 406 N/mm^2.

boundary conditions with various degrees of positional and rotational restraints were classified into four catalogues, namely Pinned-Fixed, Pinned-Pinned, Free-Fixed and Free-Pinned, where the first condition refers to the positional restraint provided at the top of the scaffold while the second condition refers to the rotational restraint provided at the bottom of the scaffold respectively. Table 2 summarizes the results obtained from NIDA while the predicted deformations at failure of both MSS1 and MSS2 are illustrated in Fig. 3.

In order to assess the structural efficiency of modular steel scaffolds, the resistance ratio defined in Section 3 is adopted which may be re-presented as p_c / p_y where p_c is the compressive buckling strength of column member at failure and p_y is the yield strength of column member. For MSS1, the resistance ratios of the finite element models under Free-Fixed and Pinned-Pinned conditions were found to be 0.40 and 0.49 respectively, comparing favourably with the resistance ratio of 0.46 from tests. For MSS2, the resistance ratios of the finite element models under Pinned-Pinned, Free-Fixed and Free-Pinned conditions were all below 0.30. The resistance ratio of the finite element model under Pinned-Fixed condition was 0.46 which was significantly higher than the resistance ratio of 0.39 from tests.

4.2. Partially restrained support conditions

It is interesting to note that the predicted deformation at failure of MSS2 under Pinned-Pinned support condition was similar to the experimental observation though the predicted load carrying capacities of MSS2 were not close to the experimental results. This was mainly due to the partial rotational restraint provided by the base jack at the bottom of the scaffolds. It should be noted that the loading frames attached at the top of the scaffolds provided considerable level of positional restraints. Thus, the support condition of the finite element model was modified accordingly as Spring-Fixed, i.e., partially restrained in position at the top while fixed at the bottom of the scaffolds (Yu *et al*. 2002). It was shown that a value of extensional stiffness, k_p, at 100 kN/m might be applied at the top of the one-storey and the two-storey scaffolds to achieve good correlation with the test results.

In general, it was difficult to determine the values of restraints provided at both the top and the bottom

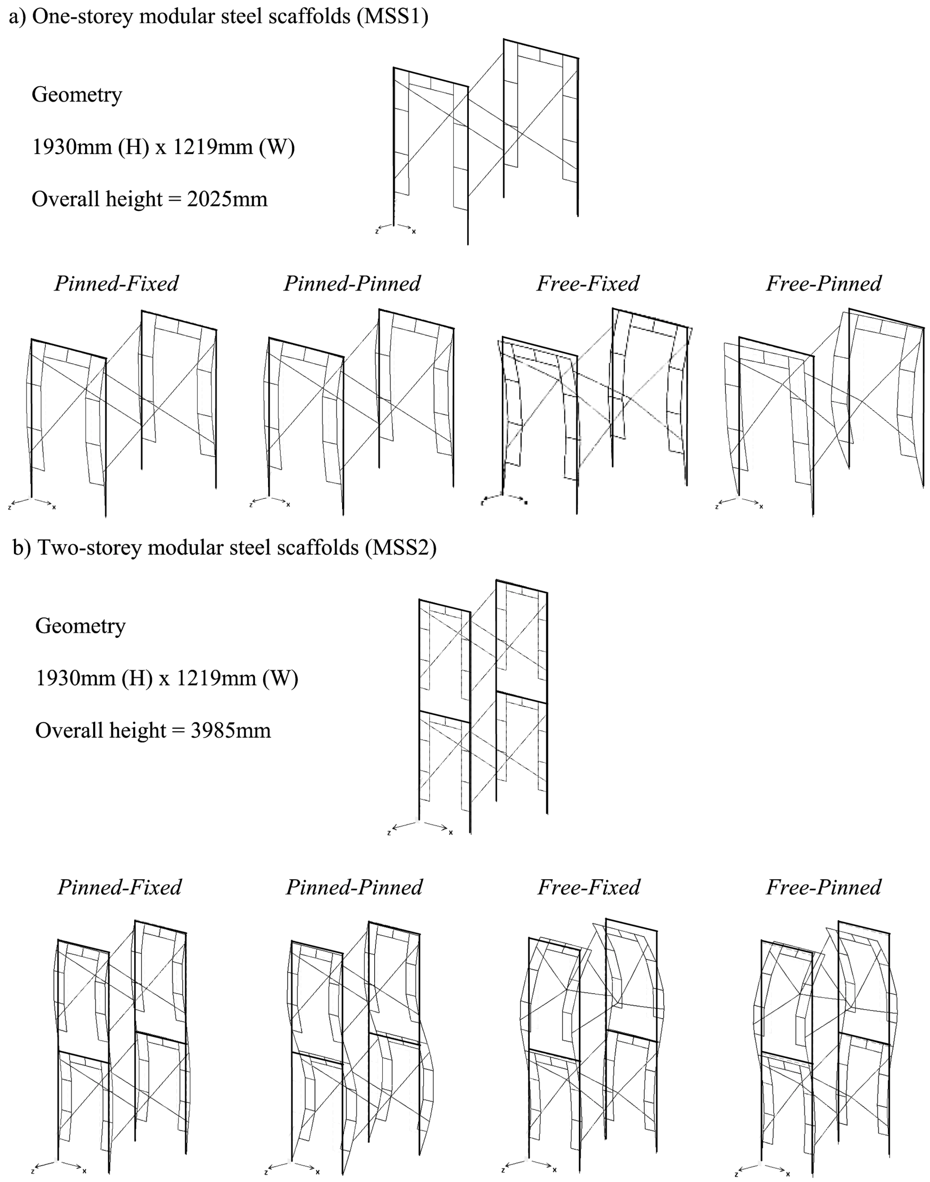


Fig. 3 Predicted deformation at failure from finite element models

of the scaffolds in practice. In order to improve the finite element models for general situations, the support condition of the model was then modified where the rotations were partially restrained at the bottom of the scaffolds. This was classified as Translational Spring - Rotational Spring condition. It should be noted that the first condition refers to positional restraint provided at the top of the scaffold with an extensional stiffness, k_p, and the second condition refers to the rotational restraint provided at the bottom of the scaffold with an rotational stiffness, k_r, respectively. Table 3 summarizes the load carrying capacities of MSS1 and MSS2 with different values of rotational stiffness, k_r. It was found that the predicted load carrying capacities and the predicted deformations at failure of the models under

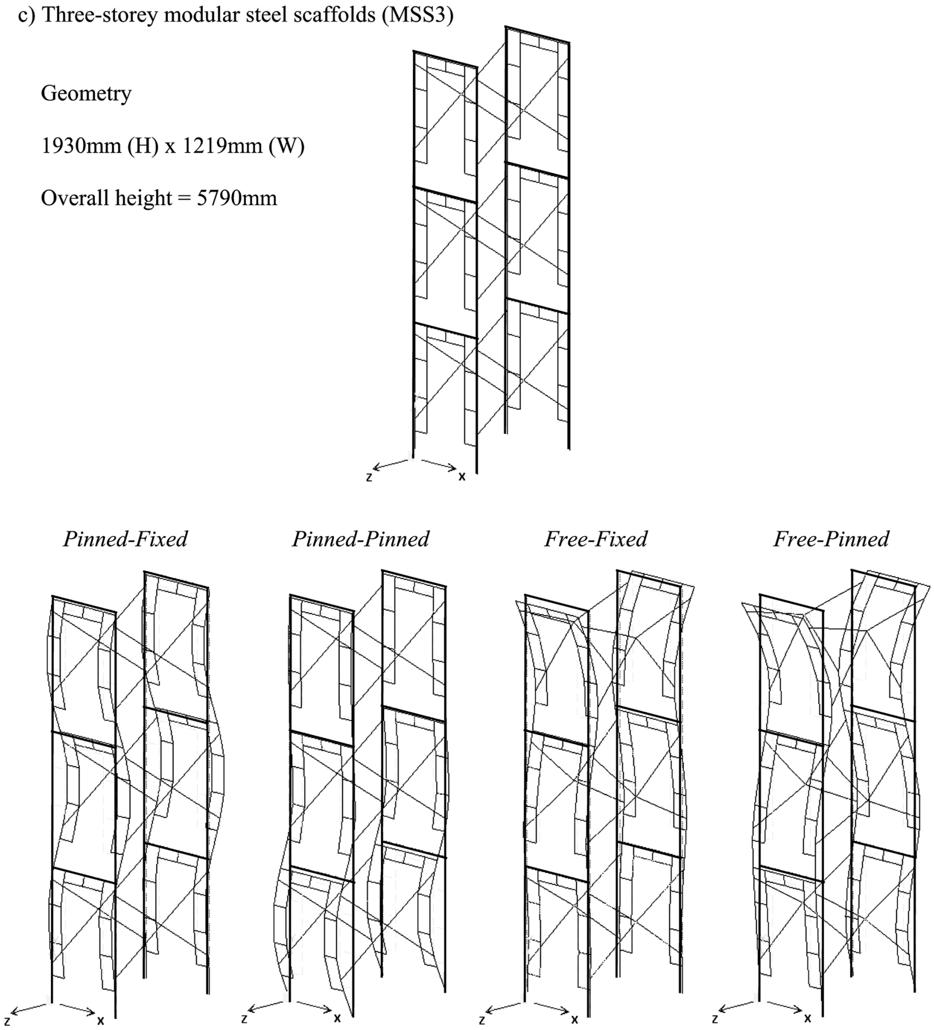


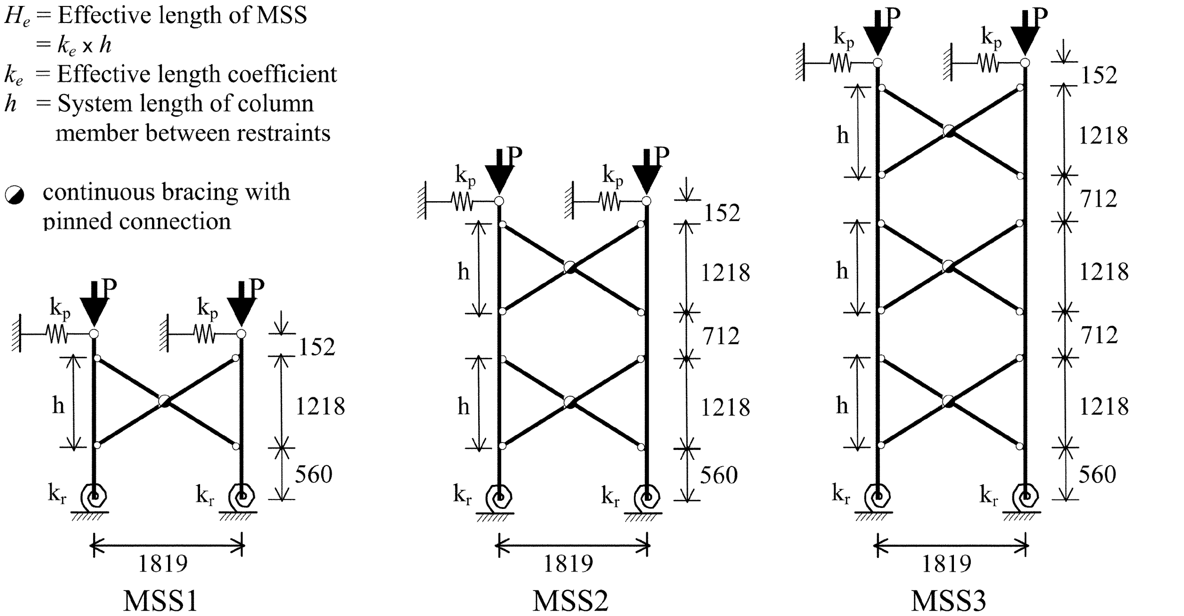
Fig. 3 Predicted deformation at failure from finite element models (Continued)

Translational Spring - Rotational Spring condition were close to the experimental results.

It is also shown in Table 3 that for the finite element models with k_r equal to 100 kNm/rad, the resistance ratios of both MSS1 and MSS2 are found to be 0.44 and 0.37 respectively, which are close to the test results of 0.46 and 0.39 respectively. It is believed that the rotational restraints at the bottom are relatively large when the modular steel scaffolds is loaded, especially when the jack extension is small as in the tests. It is also shown that a large variation in the value of k_r will only result in a small change in the resistance ratios for both MSS1 and MSS2. It is thus argued that the value of k_r may be assigned to be 10kNm/rad conservatively for both MSS1 and MSS2, and thus, the resistance ratios become 0.39 and 0.32 respectively. A maximum value of k_r at 100 kNm/rad may be used if substantial restraints are provided at the bottom of the scaffolds, and the resistance ratios may be increased to 0.44 and 0.37 for MSS1 and MSS2 respectively.

Table 3 Numerical results with partially restrained supports

	Extensional stiffness k_p (kN/m)	Rotational stiffness k_r (kNm/rad)	Failure load per leg P_{NIDA} (kN)	Resistance ratio p_c/p_y	Effective length coefficient k_e
	100	100	59.2	0.44	**1.06**
MSS1	100	10	53.6	0.39	**1.13**
	100	1	50.4	0.37	**1.18**
	100	100	51.0	0.38	**1.17**
MSS2	100	10	43.2	0.32	**1.30**
	100	1	38.3	0.28	**1.40**
	100	100	45.6	0.34	**1.26**
MSS3	100	10	44.2	0.33	**1.28**
	100	1	40.8	0.30	**1.35**



5. Three-storey modular steel scaffolds

A test series of three-storey modular steel scaffolds (Weesner and Jones 2001) was reviewed in order to provide test data for further comparison. While four different types of modular steel scaffolds were tested, these door-type scaffolds in test series A, B and D in reference 3 are found to be similar to those scaffolds in the present test series. It should be noted that the overall height of all the test specimens is approximately equal to 5700 mm. Shoring-type scaffolds were used in test series C and thus not included in this study. The basic configurations of test specimens in test series A, B and D are presented in Fig. 1. The nominal values of both external diameters and thicknesses are presented in Table 4 together with the test results. It should be noted that the nominal yield strength and the Young's modulus are assumed to be 350 N/mm^2 and 205 kN/mm^2 respectively. Lateral displacements with significant buckling were observed in the plane of the bracing members in all tests. The minimum failure loads per leg were found to be 48.8, 46.1 and 45.2 kN for test series A, B and D respectively; the

Table 4 Summary of three-storey modular steel scaffolds

	Diameter D (mm)	Thickness t (mm)	Area A (mm^2)	Maximum applied per leg P_{test} (kN)	Resistance ratio p_c/p_y
A1	42	2.3	290	48.8	0.42
A2	42	2.3	290	50.5	0.43
B1	43	2.2	306	46.1	0.37
B2	43	2.2	306	47.5	0.39
D	43	2.4	306	45.2	0.37

	Support condition					
	Pinned-Pinned		Translational spring - Rotational spring*		Translational spring - Rotational spring**	
	P_{NIDA} (kN)	p_c/p_y	P_{NIDA} (kN)	p_c/p_y	P_{NIDA} (kN)	p_c/p_y
A1	48.7	0.42	57.2	0.49	48.4	0.42
A2						
B1	44.8	0.36	48.4	0.39	47.4	0.39
B2						
D	41.6	0.34	45.2	0.37	44.2	0.36

Notes: P_{NIDA} is failure load per leg with initial imperfection simulated by eigenvalue analysis and the magnitude equal to 0.001 of unit height,
p_y is material yield strength and equal to 402 N/mm^2,
D, t are nominal external diameter and thickness of steel tube respectively,
A is the measured cross-sectional area of steel tube, and
* k_p at the top equal to 100 kN/m and k_r at the bottom equal to 100 kNm/rad,
** k_p at the top equal to 100 kN/m and k_r at the bottom equal to 10 kNm/rad.

corresponding resistance ratios are 0.42, 0.37 and 0.37 respectively.

Finite element models using the computer software NIDA were established to evaluate the load carrying capacities of these three-storey modular steel scaffolds for comparison. It should be noted that the actual yield strength is assumed to be 1.15 times the nominal value, i.e., $1.15 \times 350 = 402$ N/mm^2, which is employed in all finite element analyses. The finite element models under Translational Spring - Rotational Spring condition were then adopted with the extensional stiffness of 100 kN/m at the top and the rotational stiffness of 100 kNm/rad at the bottom of the scaffolds. It should be noted that out-of-plane buckling was predicted at failure, but the failure loads were found to be slightly higher than the test results. It was argued that not sufficient rotational restraints were provided at the bottom of the scaffolds due to the 152 mm jack extension at the base, and thus, the rotational stiffness might be reduced correspondingly to 10 kNm/rad. Subsequent analyses showed that the resistance ratios were close to the test results as shown in Table 4.

A parametric study of three-storey models (MSS3) similar to those of MSS1 and MSS2 was also carried out under different support conditions. The predicted resistance ratios of MSS3 under Pinned-Pinned, Free-Fixed and Free-Pinned conditions were found to be close to those of MSS2, and the results were also presented in Table 2 for easy comparison. The models for MSS3 under Translational Spring - Rotational Spring condition were also applied with the extensional stiffness of 100kN/m at the top and the rotational stiffness of 100 kNm/rad at the bottom of the scaffolds. It should be noted that the load carrying capacity of MSS3 under Translational Spring - Rotational Spring condition was 45.60 kN, and the corresponding resistance ratio was 0.34. The results obtained from NIDA are also presented in Table 3 for easy comparison with the numerical results of MSS1 and MSS2. The resistance ratios of the models with Restrained-Restrained condition were found to be 0.44, 0.38 and 0.34 for MSS1, MSS2 and MSS3 respectively. It should be noted that the mode shapes are slightly different for MSS2 with the rotational stiffness of 10 and 1 kNm/rad. Therefore, the resistance ratio of MSS2 is smaller than that of MSS3.

6. Column buckling design

The steel column buckling design method given in BS5950: Part 1: 2000 (British Standards Institution BS5950 2000) is adopted to assess the load carrying capacities of modular steel scaffolds based on modified slenderness ratios of the column members. The compressive buckling strength of modular steel scaffolds is expressed as a reduced compressive strength of the column members, and the design method is presented as follows:

i) Basic section properties of the column members are evaluated first:

Area, $A = \frac{\pi}{4}(D_e^2 - D_i^2)$;

Second moment of area, $I = \frac{\pi}{64}(D_e^4 - D_i^4)$.

where D_e and D_i are the external and the internal diameters of the tubular column members respectively.

The slenderness ratio, $\lambda = \frac{H_e}{r}$ where r is the radius of gyration equal to $\sqrt{\frac{I}{A}}$

The effective length of the column members, H_e, is given by

$$H_e = k_e \times h$$

where k_e is the effective length coefficient;

h is the system length of column members between restraints.

ii) The modified slenderness ratio, $\bar{\lambda}$, is given by:

$$\bar{\lambda} = \sqrt{\frac{p_y}{p_{cr}}} = \frac{\lambda}{\lambda_Y}$$

where $\lambda_Y = \pi\sqrt{\frac{E}{p_y}}$;

E is the Young's modulus and equal to 205 kN/mm^2 and

p_y is the yield strength of the cold-formed steel tubes;

iii) The strength reduction factor due to compressive buckling of column members, $\bar{\psi}$, is given by:

$$\bar{\psi} = \frac{p_c}{p_y} = \frac{1}{\bar{\phi} + \sqrt{\bar{\phi}^2 - \bar{\lambda}^2}}$$

where $\bar{\phi} = 0.5(1 + \eta + \bar{\lambda}^2)$

η $= 0.001a(\lambda - 0.2\lambda_Y)$ is an imperfection factor whose value depends on the material of the column members, and the initial out-of-straightness of the column members allowed for.

$a = 5.5$ for column members as given in Clause 4.7.5 of BS5950: Part 1: 2000.

6.1. Calibration of design method

Back analysis against test data was carried out to calibrate the proposed design method and the results are summarized in Table 5. Both the measured dimensions and the yield strengths were used in the back analysis. In order to assess the structural efficiency of modular steel scaffolds, a model factor, MF, is established and defined as follows:

$$\mathrm{MF} = \frac{\bar{\psi}_{test}}{\bar{\psi}}$$

where ψ_{test} is the resistance ratio of the modular steel scaffolds obtained from tests, and

ψ is the strength reduction factor of the modular steel scaffolds obtained from design.

The model factors for the proposed design method of the column buckling against the test data of the modular steel scaffolds up to three-storey high are also presented in Table 5. The averaged model factors are found to be 1.13, 1.04 and 1.29 for one-storey, two-storey and three-storey modular steel scaffolds respectively. The non-dimesionalized column buckling curve is plotted in Fig. 4 for direct comparison with test results, and it is shown that the proposed design method is structurally adequate.

Table 5 Back analysis of test data

		Failure load per leg P_{test} (kN)	Area A (mm^2)	Maximum compressive strength $p_{c,test}$ (N/mm^2)	Yield strength p_y (N/mm^2)	System length h_e (mm)
One-storey	1	60.6	335	180.8	412	1218
	2	63.5	336	189.0	399	1218
	3	66.1	352	187.8	409	1218
Two-storey	1	48.]7	340	143.3	380	1218
	2	55.4	366	151.3	377	1218
	3	56.2	410	137.2	344	1218
Three-storey	A1	48.8	290	168.3	402.5	1220
	A2	50.5	290	174.1	402.5	1220
	B1	46.1	306	150.7	402.5	1220
	B2	47.5	306	155.2	402.5	1220
	D	45.2	306	147.7	402.5	1219

		Effective length coefficient k_e	Effective length H_e (mm)	Modified slenderness ratio $\bar{\lambda}$	Resistance ratio $\bar{\psi}_{test}$	Strength reduction factor $\bar{\psi}$	Model factor MF
One-storey	1	1.1	1340	1.33	0.44	0.40	1.10
	2	1.1	1340	1.31	0.47	0.41	1.16
	3	1.1	1340	1.32	0.46	0.40	1.14
Two-storey	1	1.2	1462	1.40	0.38	0.37	1.02
	2	1.2	1462	1.39	0.40	0.37	1.08
	3	1.2	1462	1.34	0.40	0.39	1.03
Three-storey	A1	1.3	1586	1.60	0.42	0.30	1.40
	A2	1.3	1586	1.60	0.43	0.30	1.44
	B1	1.3	1586	1.56	0.37	0.31	1.19
	B2	1.3	1586	1.56	0.39	0.31	1.23
	D	1.3	1585	1.55	0.37	0.31	1.17
						Average	**1.18**

Notes: Resistance ratio, $\overline{\psi}_{test} = p_c / p_y$ based on test data;

Model factor, $MF = \overline{\psi}_{test} / \overline{\psi}$.

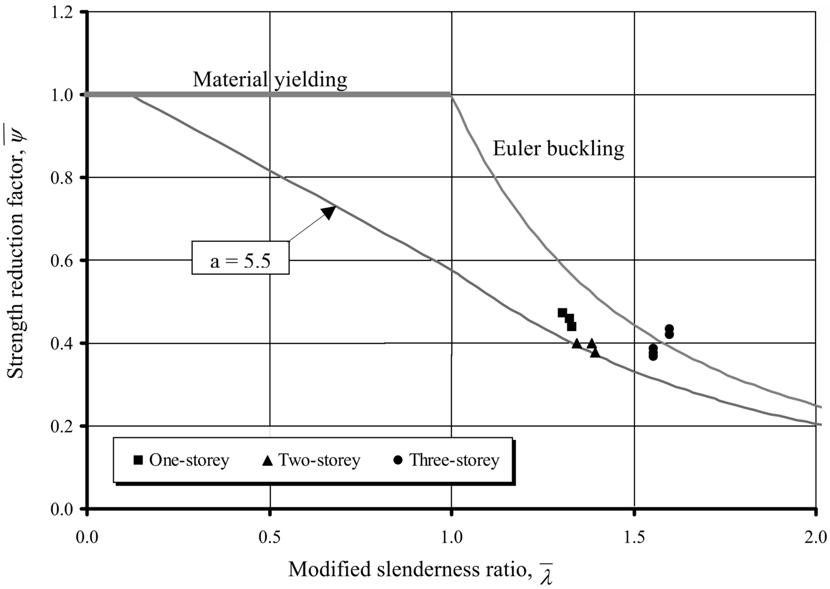


Fig. 4 Column buckling curve for modular steel scaffolds

6.2. Effective length coefficients

It is interesting to re-interpret the findings using the proposed design method. The effective length coefficients of all the finite element models with various support conditions are presented in Tables 2 and 3 for easy comparison. It is shown in Table 2 that the effective length coefficients of the scaffolds up to three-storey height may be assigned conservatively to be 1.60. In test conditions, the effective lengths of MSS1, MSS2 and MSS3 may be taken as 1.10, 1.20 and 1.30 respectively of the column heights between bracing members, *h*, as shown in Table 3. It also demonstrates the effectiveness of the bracing members in suppressing overall column buckling of the scaffolds.

7. Practical considerations

Attention should be drawn to the following for practical uses and erection of modular steel scaffolds.

7.1. Support restraints in modular steel scaffolds

In general, the load carrying capacities of modular steel scaffolds depend significantly on the restraint provided at the top and the bottom of the scaffolds. Conservatively, the effective length coefficient may be assigned to be 1.6 for self-standing modular steel scaffolds up to three-storey high. The rotational restraint at the bottom of the scaffolds may affect the buckling mode shapes of the lowest storey, together with significant changes in the load carrying capacities in one-storey and two-storey modular steel scaffolds. It should be noted that the support conditions at the bottom of the scaffolds may be controlled by the jack extension. For jack extensions larger than 150 mm, the rotational stiffness should be reduced to 10 kNm/rad or the effective length coefficients should be increased by 15%. In all cases, the extension of screw jack at the base should be less than 600 mm.

7.2. Initial out-of-plumbness

In order to achieve high structural efficiency of modular steel scaffolds, quality control and site supervision should be carried out to ensure proper use of modular steel scaffolds. Screw jacks at the base should be adjusted to maintain the modular steel scaffolds in a vertical position, and also to ensure that the loads are evenly distributed among the four legs. It is recommended that all column members should be plumb within 10mm over a modular unit height while the maximum displacement from the vertical should be less than 15mm. It is also important to limit the maximum initial out-of-plumbness of both beam and column members to 5mm in order to ensure that the proposed design method is applicable.

8. Conclusions

A theoretical and experimental investigation to the structural behaviour of modular steel scaffolds under different support conditions is presented. It is demonstrated that the load carrying capacities of the modular steel scaffolds are very sensitive to the positional restraint, k_p, and the rotational restraint, k_r, provided at the top and the bottom of the scaffolds respectively. It is important to incorporate the effects of these restraints in assessing the structural behaviour of the scaffolds under both experimental and numerical investigations.

Finite element models with partially restrained conditions at both the top and the bottom are employed to model these scaffolds. Based on the results of the advanced non-linear analyses, practical values of effective length coefficients are also provided. Conservatively, the effective length coefficient may be assigned to be 1.6 of the scaffolds up to three-storey height. After calibration against test data, the codified design method for steel column buckling is shown to be structurally adequate, and it may be used effectively to design modular steel scaffolds against column buckling in both test and practical conditions. Structural engineers are thus encouraged to design modular steel scaffolds rationally to achieve enhanced structural economy and safety.

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