Steel and Composite Structures, Vol. 4, No. 6 (2004) 471-487
DOI: http://dx.doi.org/10.12989/scs.2004.4.6.471

Prediction on load carrying capacities of multi-storey door-type modular steel scaffolds

W. K. Yu† and K. F. Chung‡

Department of Civil and Structural Engineering, The Hong Kong Polytechnic University, Hong Kong SAR, China

(Received September 24, 2004, Accepted November 22, 2004)

Abstract. Modular steel scaffolds are commonly used as supporting scaffolds in building construction, and traditionally, the load carrying capacities of these scaffolds are obtained from limited full-scale tests with little rational design. Structural failure of these scaffolds occurs from time to time due to inadequate design, poor installation and over-loads on sites. In general, multi-storey modular steel scaffolds are very slender structures which exhibit significant non-linear behaviour. Hence, secondary moments due to both P-δ and P-Δ effects should be properly accounted for in the non-linear analyses. Moreover, while the structural behaviour of these scaffolds is known to be very sensitive to the types and the magnitudes of restraints provided from attached members and supports, yet it is always difficult to quantify these restraints in either test or practical conditions. The problem is further complicated due to the presence of initial geometrical imperfections in the scaffolds, including both member out-of-straightness and storey out-of-plumbness, and hence, initial geometrical imperfections should be carefully incorporated.

This paper presents an extensive numerical study on three different approaches in analyzing and designing multi-storey modular steel scaffolds, namely, a) *Eigenmode Imperfection Approach*, b) *Notional Load Approach*, and c) *Critical Load Approach*. It should be noted that the three approaches adopt different ways to allow for the non-linear behaviour of the scaffolds in the presence of initial geometrical imperfections. Moreover, their suitability and accuracy in predicting the structural behaviour of modular steel scaffolds are discussed and compared thoroughly. The study aims to develop a simplified and yet reliable design approach for safe prediction on the load carrying capacities of multi-storey modular steel scaffolds, so that engineers can ensure safe and effective use of these scaffolds in building construction.

Key words: modular steel scaffolds; initial geometrical imperfections; notional load approach; eigenmode imperfection approach; critical load approach.

1. Introduction

Modular steel scaffolds are commonly used as supporting scaffolds in building construction, and traditionally, the load carrying capacities of these scaffolds are obtained from limited full-scale tests with little rational design. In recent years, there is a growing concern about the validity of these measured load carrying capacities of the scaffolds in real construction applications as these capacities are obtained from tests with specific support conditions in laboratories which are often very different to those found on construction sites. Structural failure of multi-storey modular steel scaffolds occurs from

†Research Associate
‡Associate Professor, Corresponding Author, E-mail: cekchung@polyu.edu.hk

time to time due to inadequate design, poor installation and over-loads on sites as reported by Lightfoot and Oliveto (1977), Holmes and Hindson (1979), Gylltoft and Mroz (1995), Peng *et al*. (1996) and Godley and Beale (1997). Fig. 1 presents the general arrangement of multi-storey modular steel scaffolds.

In general, multi-storey modular steel scaffolds are highly susceptible to global and local instability, and buckling of column members in both out-of-plane and in-plane of the scaffolding frames are possible, depending on steel grades, member configurations and dimensions, as well as loading and support conditions. A comprehensive study on the structural behaviour of standard scaffolding structures in bridge construction was reported by Godley and Beale (2001) and Milokovic *et al*. (2002). Moreover, Peng *et al*. (1996) and Huang *et al*. (2000a & 2000b) reported separate investigations into the structural behaviour of high clearance scaffolding systems in building construction where both physical tests and theoretical studies were carried out.

In typical reinforced concrete and steel structures where the structural behaviour is predominately linear, moments and forces in members are generally determined from simple linear analysis, and the members are then checked against section capacities and member resistances according to relevant design rules. However, in typical modular steel scaffolds with very slender members, lateral deformations of these scaffolds are often apparent well before failure. Hence, non-linear analysis is normally required to predict the structural behaviour of these scaffolds under significant secondary moments induced by both P-δ and P-Δ effects. The problem is further complicated due to the presence of initial geometrical imperfections in the scaffolds, including both member out-of-straightness and storey out-of plumbness.

Various attempts on non-linear finite element analyses of modular steel scaffolds were reported in the literature. Among all, the first systematic second order numerical investigation on modular steel scaffolds using cubic Hermite elements were reported by Peng. It was found that in most cases, the finite element models using cubic Hermite elements gave results close to those obtained by finite elements based on the classical stability function. Moreover, the commercial finite element software ANSYS was employed by Huang *et al*. (2000a) and Weesner and Jones (2001) to predict the load carrying capacities of modular steel scaffolds. Both classical eigenvalue analyses and elastic geometrically non-linear analyses were performed to estimate the elastic critical buckling loads of these scaffolds. However, initial geometrical imperfection and material yielding were not incorporated in these models.

In order to incorporate storey out-of-plumbness in multi-storey modular steel scaffolds, both Liew *et al*. (1994) and Peng (1994) employed notional force approach in their non-linear analyses. It was found that for unbraced scaffolds, storey notional loads (for storey out-of-plumbness) were more important than member notional loads (for member out-of-straightness) in predicting both stiffness and strength of these scaffolds. In contrast, the load carrying capacities of braced scaffolds were significantly affected by member notional loads rather than storey notional loads. It was recommended that a combination of both storey and member notional loads should be used, whenever appropriate, to provide realistic lower bound estimation of the load carrying capacities of these scaffolds.

More recently, a systematic study on the structural behaviour of multi-storey door-type modular steel scaffolds through both experimental and numerical investigations was reported by Yu *et al*. (2004). Three one-storey and three two-storey modular steel scaffolds (MSS1 and MSS2) were built and tested to failure in order to examine the structural behaviour of typical scaffolds. Test data from literature on three-storey door-type modular steel scaffolds (MSS3) with similar member configuration and geometrical dimension were also adopted for investigation. An advanced non-linear finite element model (Yu *et al*. 2002) with high performance beam-column elements (Chan and Zhou 1998) was established to evaluate the load carrying capacities of these scaffolds under idealized boundary conditions. Moreover, in order to provide practical design guidance on the safe and effective use of multi-storey door-type modular steel

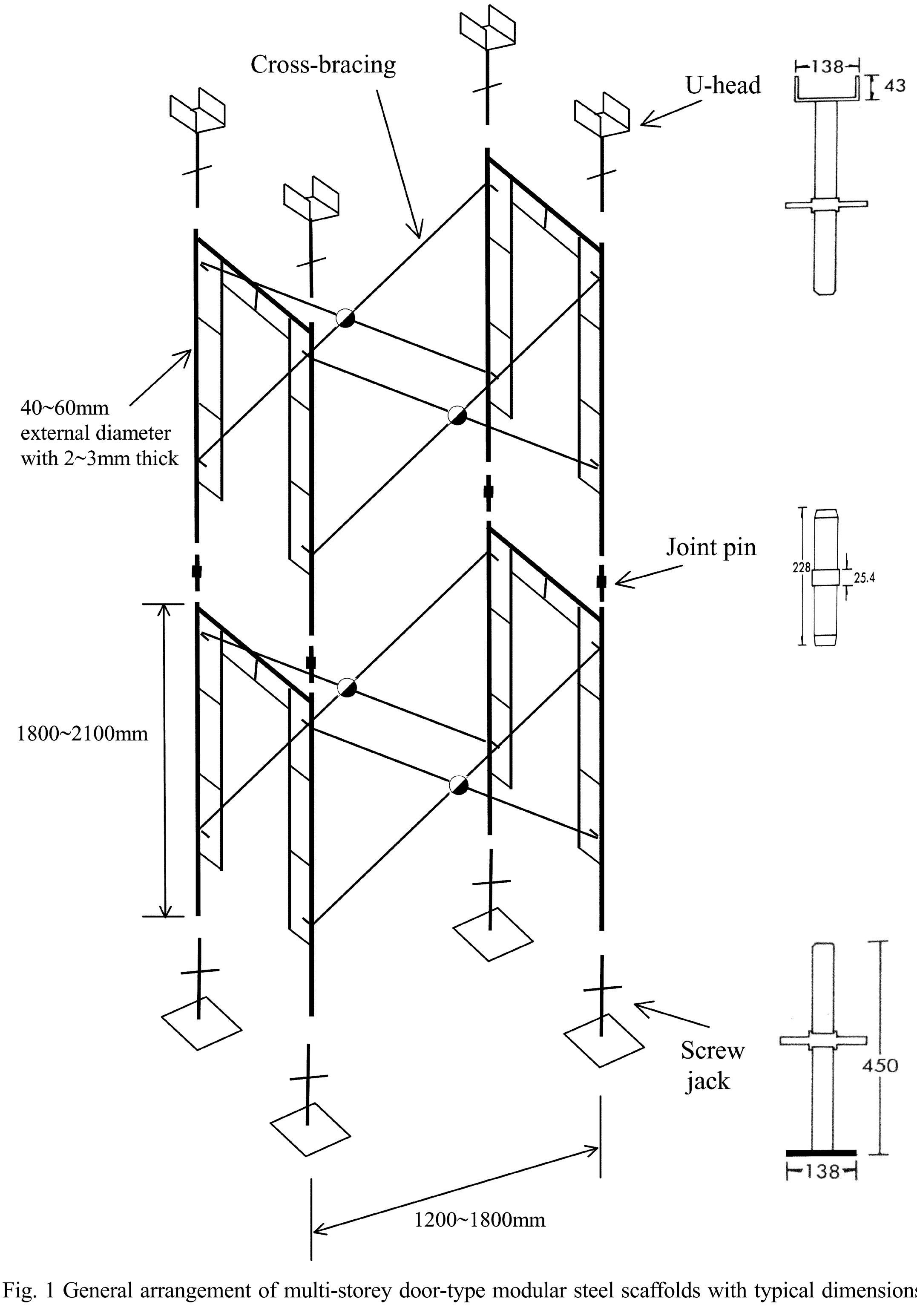


Fig. 1 General arrangement of multi-storey door-type modular steel scaffolds with typical dimensions

scaffolds, the structural behaviour of these scaffolds under a wide range of positional and rotational restraints at the top and the bottom of the scaffolds is also examined through an extensive parametric study.

It was demonstrated that the load carrying capacities of these scaffolds were sensitive to both the types and the magnitudes of restraints provided at the top and the bottom of the scaffolds, and also to the presence of cross-bracings. Hence, the load carrying capacities of two-storey and three-storey modular steel scaffolds were found to be only 85 and 80% of those of one-storey modular steel scaffolds respectively. Moreover, depending on the boundary conditions provided at the top and the bottom of the modular steel scaffolds, the load carrying capacities were found to vary from 50% to 120% of those obtained from tests. Engineers were thus urged to design, specify and erect multi-storey door-type modular steel scaffolds with caution. Careful interpretation on test results and safe load data is essential to the safe use of these scaffolds on sites, in particular, for those scaffolds with boundary conditions significantly different from those adopted in tests.

2. Scope of work

This paper presents an extensive numerical study (Yu 2004a) on three different approaches in analyzing and designing multi-storey modular steel scaffolds, namely,

a) *Eigenmode Imperfection Approach,*
b) *Notional Load Approach*, and
c) *Critical Load Approach*.

These three approaches adopt different ways to allow for the non-linear behaviour of the scaffolds in the presence of initial geometrical imperfections. Moreover, their suitability and accuracy in predicting the structural behaviour of typical modular steel scaffolds are discussed and compared systematically. The study aims to develop a simplified and yet reliable design approach for safe prediction on the load carrying capacities of multi-storey modular steel scaffolds.

For easy reference, a summary of all the test data of one-storey and two-storey modular steel scaffolds (MSS1 and MSS2) reported by Yu *et al*. (2004) and a total of seven three-storey modular steel scaffolds (MSS3) reported by Weesner and Jones (2001) is presented in this paper. Fig. 2 illustrates the member configurations and the geometrical dimensions of all the modular steel scaffolds while Table 1 summarizes all the test data of test series MSS1, MSS2 and MSS3.

3. Eigenmode Imperfection Approach

In order to predict the structural behaviour of multi-storey modular steel scaffolds with full consideration of P-δ and P-Δ effects, an advanced finite element model with high performance beam-column elements was established, and elastic geometrically non-linear analyses were performed to predict the load carrying capacities of the scaffolds. Fig. 3 illustrates the finite element models of all the scaffolds in test series MSS1, MSS2 and MSS3. It should be noted that the failure loads of the scaffolds are equal to the applied loads at which the stresses of extreme fibers of the cross-sections reach the design strength under combined compression and bending. In order to predict realistic load carrying capacities and buckling modes of these scaffolds under test conditions for rational comparison, an assumed boundary condition with the extensional stiffness of 100 kN/m at the top and the rotational stiffness of 10 kNm/rad at the bottom of the scaffolds (Chung and Yu 2003, Yu 2004b) is employed in all analyses.

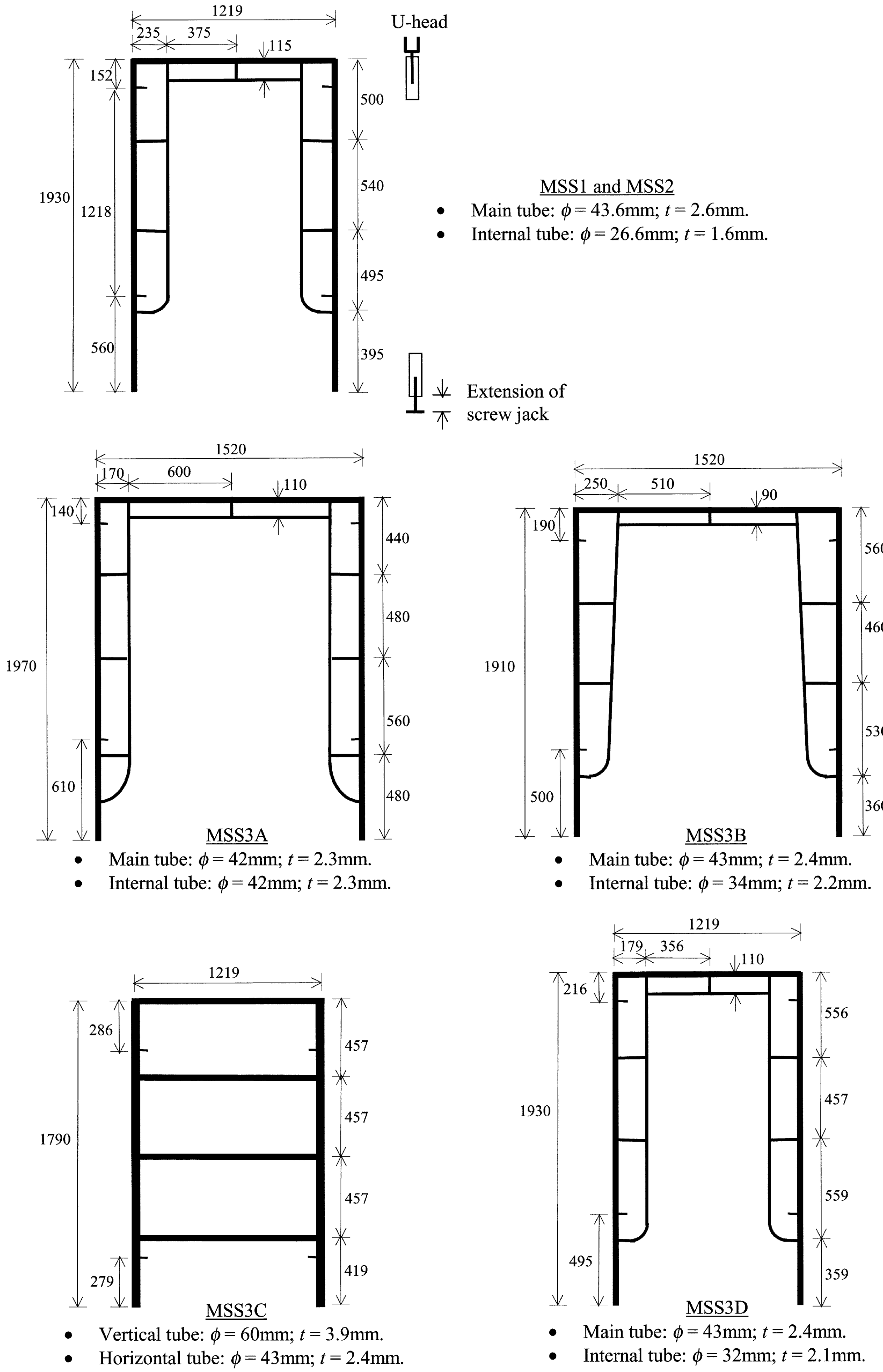


Fig. 2 Typical dimensions of scaffolding frame units

Table 1 Summary of test data and results

Test		Diameter D (mm)	Thickness t (mm)	Area A (mm^2)	Maximum failure load per leg (kN)	Averaged failure load per leg P_{test} (kN)	Strength utilization ratio χ_{test}
MSS1	1	43.2	2.63	335.2	60.6	63.4	**0.46**
	2	43.3	2.63	336.0	63.5		
	3	43.4	2.76	351.9	66.1		
MSS2	1	43.2	2.67	339.8	48.7	53.4	**0.39**
	2	43.5	2.87	366.2	55.4		
	3	43.3	3.26	409.6	56.2		
MSS3	A1	42	2.3	290	48.8	49.7	**0.43**
	A2	42	2.3	290	50.5		
	B1	43	2.2	306	46.1	46.8	**0.38**
	B2	43	2.2	306	47.5		
	C1	60	3.9	687	125.6	128.5	**0.46**
	C2	60	3.9	687	131.3		
	D	43	2.4	306	45.2	45.2	**0.37**

Notes: D, t are the nominal external diameter and the nominal thickness of steel tubes respectively.
A is the nominal cross-sectional area of steel tubes.
p_y is the measured yield strength and equal to 406 N/mm^2 for MSS1 and MSS2.
is the nominal yield strength and equal to 350 N/mm^2 for MSS3.
the actual yield strength is assumed to be equal 1.15×350 N/mm^2=402.5 N/mm^2.

As the effects of initial geometrical imperfections are considered to be important to the structural behaviour of multi-storey modular steel scaffolds, the initial geometrical imperfections are directly imposed onto the geometry of the scaffolds in the present approach, and hence, there are member out-of-straightness in all the members and storey out-of-plumbness in the scaffolds. While there are many ways to generate suitable initial geometrical imperfections, the eigenmodes corresponding to the lowest eigenvalues of the scaffolds are adopted because such eigenmodes are distinctively defined and easily obtained from classical eigenvalue analyses. This approach is widely employed in non-linear analyses of thin-walled steel plate and shell structures because of its simplicity and rationality.

It should be noted that the eigenmodes define the complete geometry of the scaffolds explicitly, coupling up both the member out-of-straightness and the storey out-of-plumbness. Thus, the magnitude of the initial geometrical imperfection of imperfect models, δ, may be readily assigned according to fabrication and erection tolerances in practice. Two practical limiting values of δ are assigned in the present study for comparison:

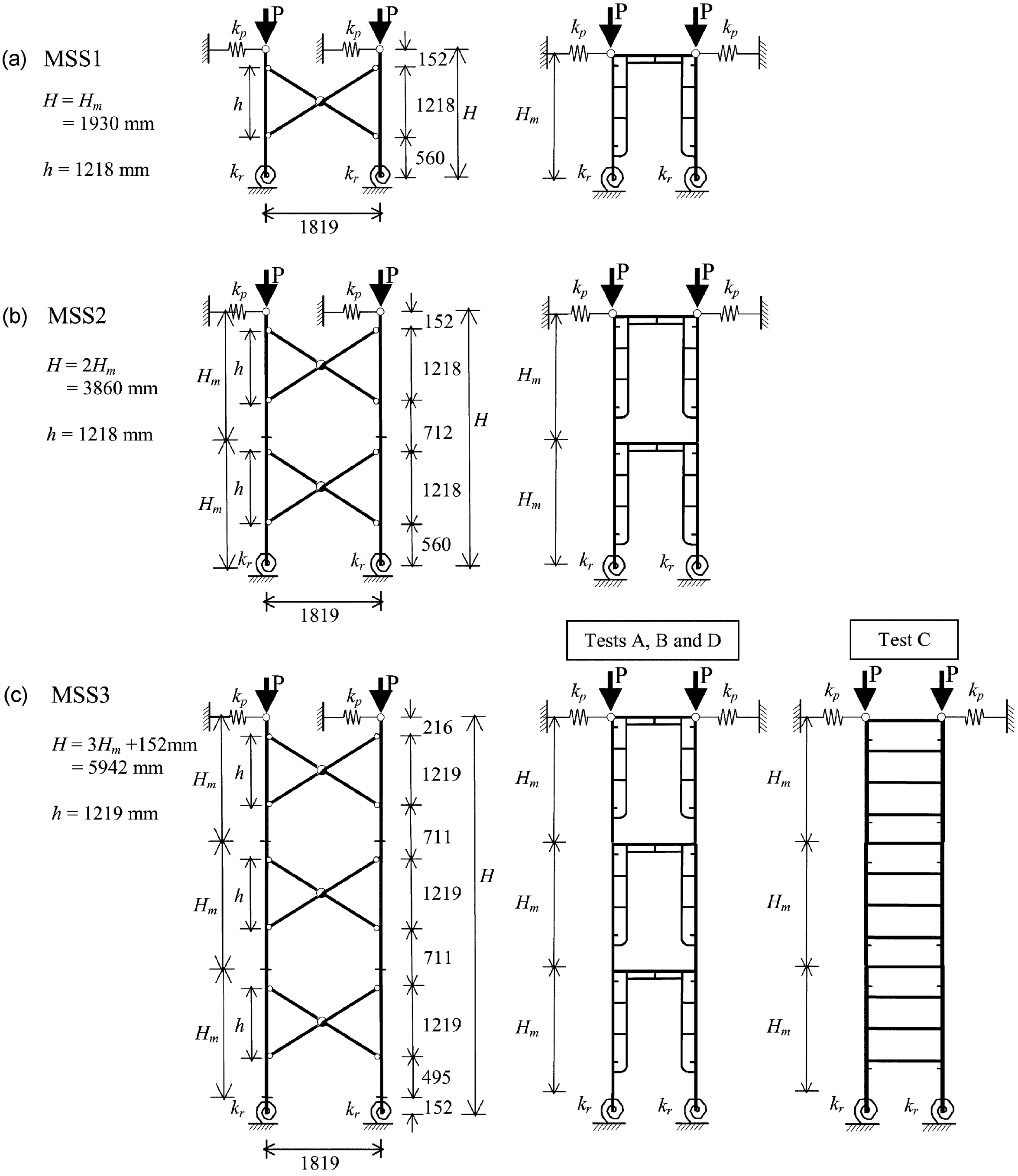
$$\delta = \delta_L = H_m/1000 \qquad \text{(lower bound value)} \tag{1a}$$

$$= \delta_H = H/1000 \qquad \text{(upper bound value)} \tag{1b}$$

where

H_m is the modular height of scaffolding frames, and

H is the overall height of modular steel scaffolds.



H is the overall height of modular steel scaffolds

H_m is the modular height and equal to 1930 mm

h is the system height of column member between restraints

k_p is the positional restraint at the top of the scaffolds and equal to 100 kN/m

k_r is the rotational restraint at the bottom of the scaffolds and equal to 10 kNm/rad

continuous bracing with pinned connection

Fig. 3 Finite element models

Table 2 Failure loads of modular steel scaffolds obtained from Eigenmode Imperfection Approach (EIA)

Test		Failure load per leg obtained from tests	Failure load per leg obtained from analyses		Strength utilization ratio		
			δ_L=H_m/1000	δ_H=H/1000			
		P_{test} (kN)	$P_{EIA\text{-}\delta_L}$ (kN)	$P_{EIA\text{-}\delta_H}$ (kN)	χ_{test}	$\chi_{EIA\text{-}\delta_L}$	$\chi_{EIA\text{-}\delta_H}$
MSS1		63.4	53.6	-	0.46	0.39	-
MSS2		53.4	43.3	41.0	0.39	0.31	0.30
MSS3	A	49.7	48.6	43.2	0.43	0.42	0.37
	B	46.8	47.3	43.0	0.38	0.38	0.35
	C	128.5	136.7	127.0	0.46	0.49	0.46
	D	45.2	44.1	40.0	0.37	0.36	0.32

Notes: a) The measured yield strength of steel tubes is 406 N/mm^2 for MSS1 and MSS2.
b) The yield strength of steel tubes is assumed to be 402.5 N/mm^2 for MSS3.
c) The lowest eigenmodes of the scaffolds under the assumed boundary condition are adopted as the initial geometrical imperfection.
d) The magnitude of initial out-of-straightness, δ, is taken as either H_m/1000 or H/1000.

Table 2 presents the predicted failure loads of multi-storey modular steel scaffolds in test series MSS1, MSS2 and MSS3 based on both δ_L and δ_H together with test results for direct comparison. Moreover, all the results are also presented in terms of strength utilization ratios for easy comparison. Typical modes of failure of all the scaffolds are illustrated in Figs. 4, 5 and 6 for test series MSS1, MSS2 and MSS3, respectively.

It is shown that the predicted failure loads based on δ_L are found to be close to the test results with similar buckling mode shapes for test MSS1, MSS2 as well as MSS3-A, -B and -D. However, for test MSS3-C where the member configuration is significantly different from those of MSS3-A, -B and -D, the predicted failure load based on δ_L is found to be slightly larger than the test result. Hence, it is necessary to adopt δ_H in predicting the failure load of test MSS3-C; the failure mode shape is shown to be bi-axial buckling. In general, the failure loads of the modular steel scaffolds are found to be reduced by 10% when δ_H is used instead of δ_L.

Hence, the proposed approach is applicable to multi-storey modular steel scaffolds with δ equal to δ_L in most cases. For steel scaffolds with member configurations significantly different from MSS3-A, -B and -D, further investigation is necessary.

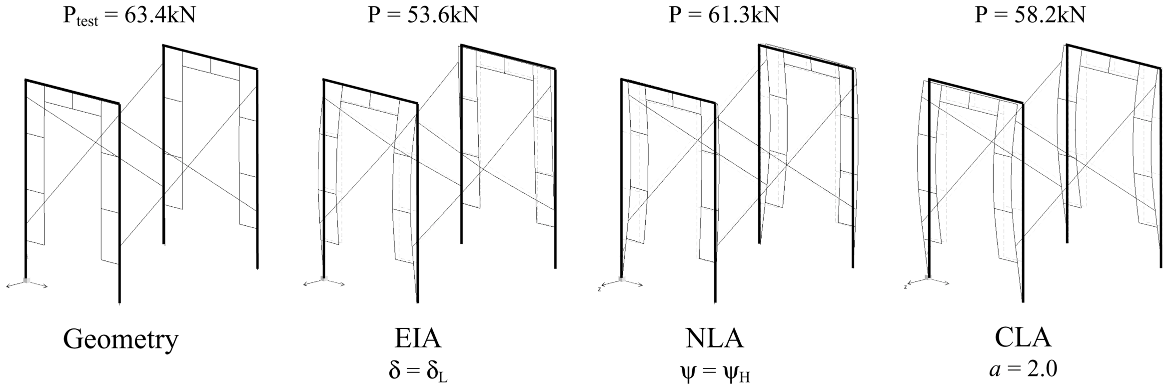
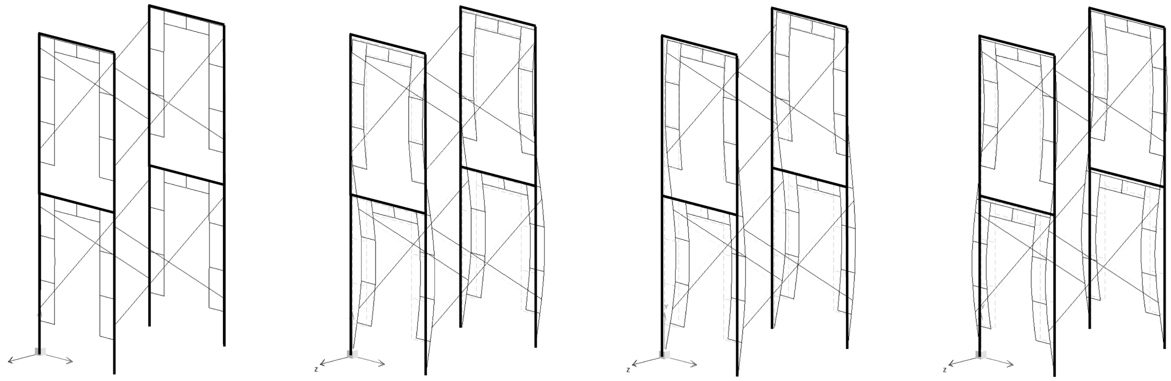


Fig. 4 Predicted failure modes of MSS1

P_{test} = 53.4kN P = 43.3kN P = 44.9kN P = 42.0kN



Geometry

EIA $\delta = \delta_L$

NLA $\psi = \psi_H$

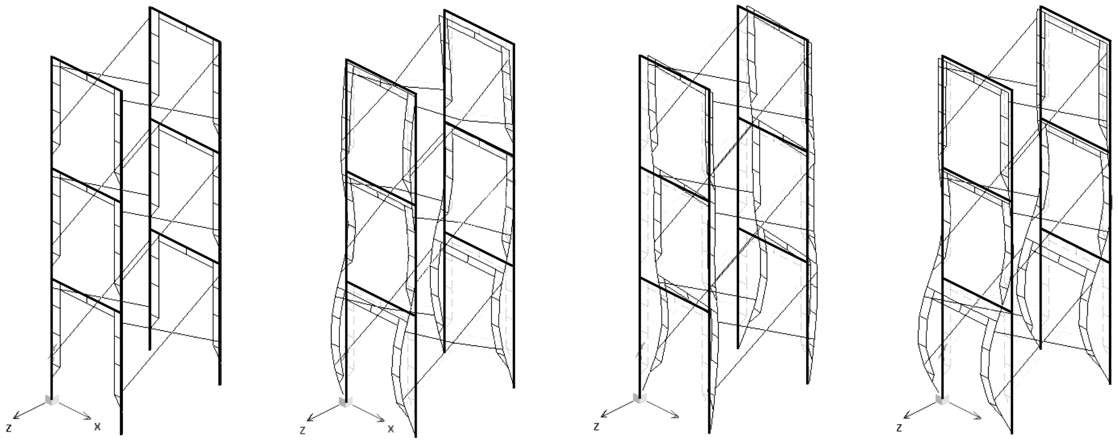
CLA $a = 2.0$

Fig. 5 Predicted failure modes of MSS2

4. Notional Load Approach

In multi-storey multi-bay steel frames, the effect of initial geometrical imperfection on the structural behaviour of the frames is conventionally allowed for through the use of notional loads. However, contrary to the Eigenmode Imperfection Approach where both member out-of-straightness and storey out-of-plumbness are incorporated, only storey out-of-plumbness is accounted for indirectly in this approach through the deformations caused by the notional loads during structural analysis. The effect of member out-of-straightness is incorporated in member design through the use of column buckling curves of imperfect columns.

P_{test} = 49.7kN P = 48.6kN P = 49.8kN P = 49.0kN



Geometry

EIA $\delta = \delta_L$

NLA $\psi = \psi_H$

CLA $a = 2.0$

Fig. 6(a) Predicted failure modes of MSS3-A

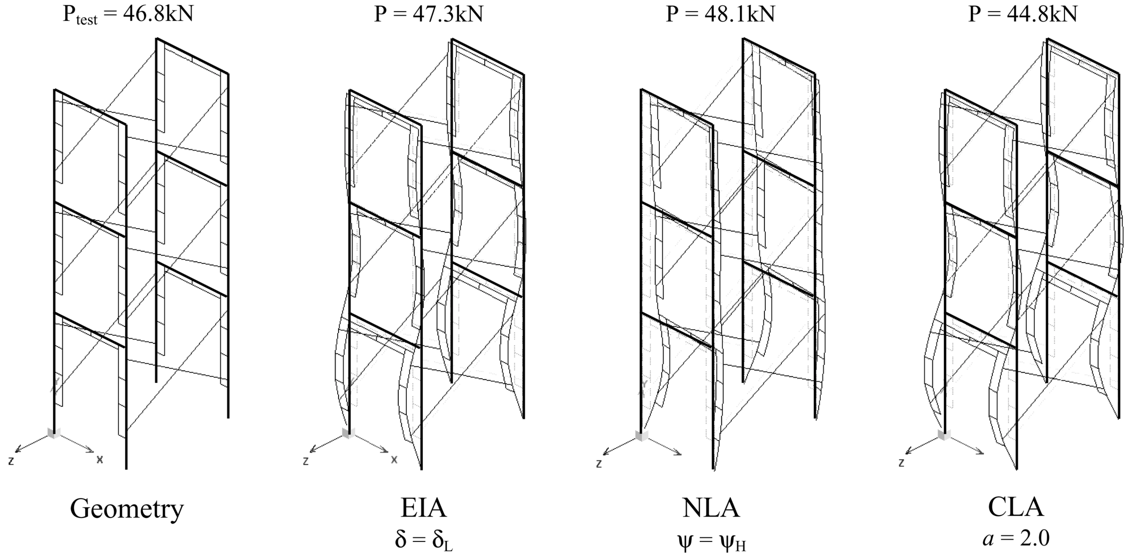


Fig. 6(b) Predicted failure modes of MSS3-B

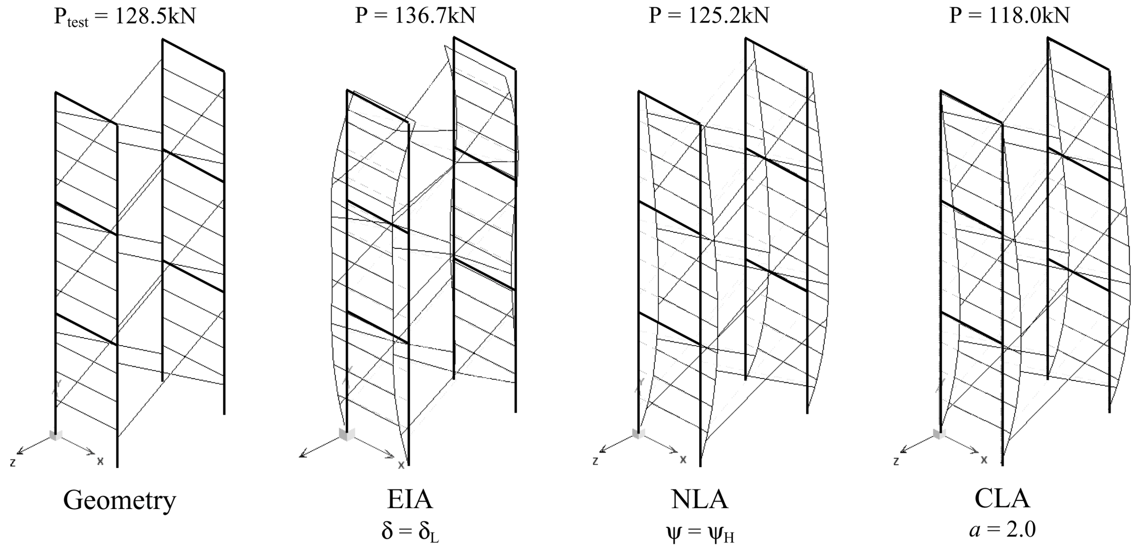


Fig. 6(c) Predicted failure modes of MSS3-C

In general, the storey out-of-plumbness is modeled by an equivalent geometrical imperfection ψ in elastic linear analysis of framed structures, and hence, an equivalent notional load, $N=\psi P$, is adopted, as shown in Fig. 7. In Eurocode 3: Part 1.1 (1993 & 2002), the magnitude of the notional load is defined as a function of the member configuration of a frame, and the notional load should be applied at each floor and roof level of the frame at any one direction at a time. Table 3 presents the recommended values of the notional forces in typical beam-column framed buildings. However, BS5950: Part 1 (2000) recommends a notional load to be applied at any one direction at one time at each loaded level of a frame, and the magnitude of the notional load should be equal to 0.5% of the total factored vertical

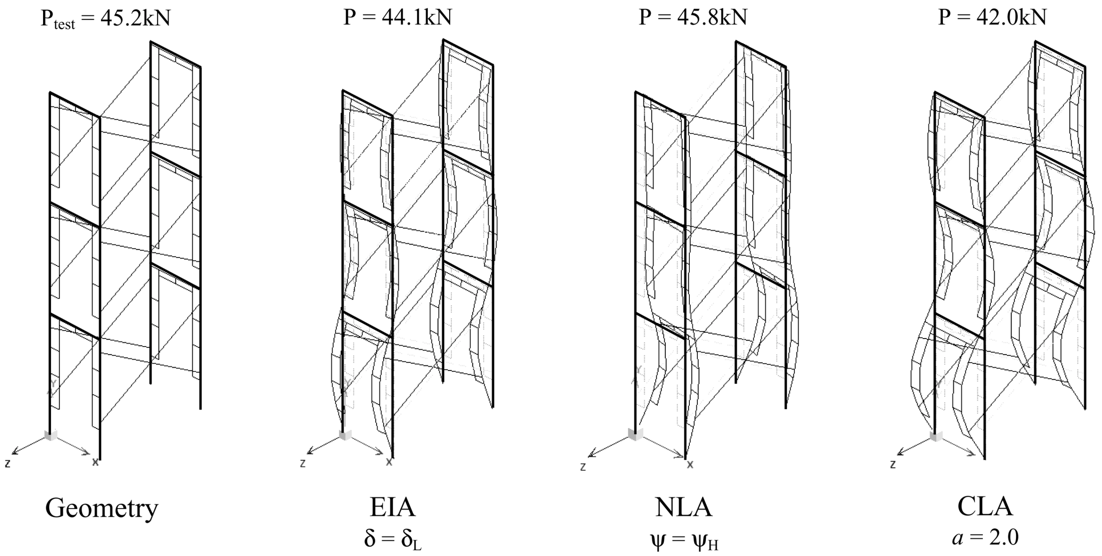


Fig. 6(d) Predicted failure modes of MSS3-D

(dead and imposed) loads. However, when examining the structural behaviour of two-storey modular steel scaffolds where both the top and the bottom of the scaffolds were pinned, Peng *et al*. (1997) adopted a notional load at 0.1% of the total vertical loads to be applied to the mid-height of the scaffolds instead.

In order to determine the appropriate magnitude of notional load, two values of ψ are assigned in the present study for comparison:

$$\psi = \psi_L = 0.001 \qquad \text{(lower bound value)} \tag{2a}$$
$$= \psi_H = 0.005 \qquad \text{(upper bound value)} \tag{2b}$$

It should be noted that the notional loads are applied in two directions, i.e., parallel and perpendicular to the plane of the frames, and also applied to all levels at a modular height apart, as shown in Fig. 8. This loading arrangement is considered to be essential in analyzing modular steel scaffolds where the

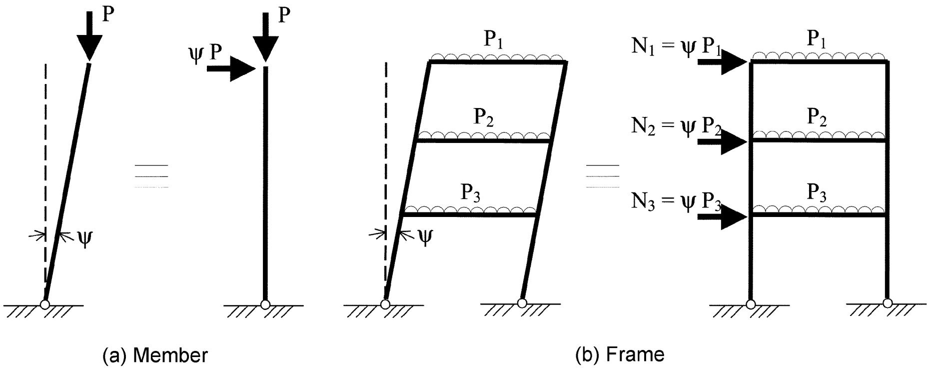


Fig. 7 Initial imperfection and equivalent notional loads

Table 3 Notional forces recommended by Eurocode 3: Part 1 (2003)

Number of storey	Number of bay		
	1	2	4
1	0.43	0.41	0.39
2	0.43	0.41	0.39
3	0.35	0.33	0.32
5	0.29	0.27	0.26
10	0.29	0.27	0.26

Notes: $N=0.01\times\eta\times\Sigma P$

where $\eta = \alpha_h\ \alpha_m\ \psi_o$

$\psi_o=0.5$

$\alpha_h=\dfrac{2}{\sqrt{h}}$ but $\dfrac{2}{3}\leq\alpha_h\leq 1.0$

$\alpha_m=\sqrt{0.5(1+1/m)}$

h is the height of structure which is taken as 2m in this case

m is the number of columns

critical plane of buckling is either difficult to be determined without detailed calculation or sensitive to initial geometrical imperfection. Moreover, in the finite element models, all the members are straight, i.e., without member out-of-straightness, and the entire scaffolds are plumbed, i.e., without storey out-of plumbness. All the multi-storey modular steel scaffolds are analyzed with elastic geometrically non-linear analyses.

Table 4 presents the predicted failure loads of multi-storey modular steel scaffolds in test series MSS1, MSS2 and MSS3 based on both ψ_L and ψ_H together with test results for direct comparison. Moreover, all the results are also presented in terms of strength utilization ratios for easy comparison. Typical modes of failure of all the scaffolds are illustrated in Figs. 4, 5 and 6 for test series MSS1, MSS2 and MSS3, respectively.

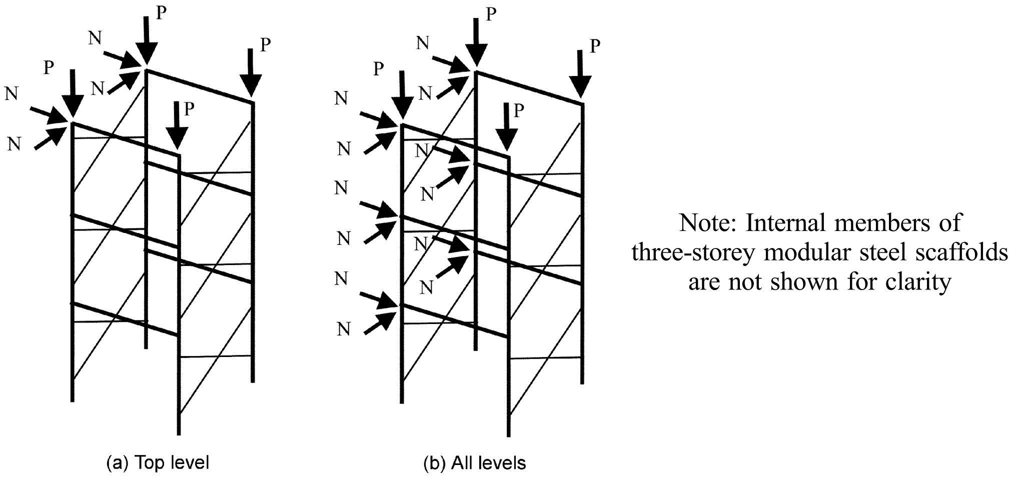


Fig. 8 Notional loads for storey out-of-plumbness

Table 4 Failure loads of modular steel scaffolds obtained from Notional Load Approach (NLA)

Test		Failure load per leg obtained from tests	Failure load per leg obtained from analyses		Strength utilization ratio		
			ψ_L=0.001	ψ_H=0.005			
		P_{test} (kN)	$P_{NLA\text{-}\psi_L}$ (kN)	$P_{NLA\text{-}\psi_H}$ (kN)	χ_{test}	$\chi_{NLA\text{-}\psi_L}$	$\chi_{NLA\text{-}\psi_H}$
MSS1		63.4	64.7	61.3	0.46	0.47	0.44
MSS2		53.4	46.1	44.9	0.39	0.33	0.32
MSS3	A	49.7	56.2	49.8	0.43	0.48	0.43
	B	46.8	51.1	48.1	0.38	0.41	0.39
	C	128.5	135.3	125.2	0.46	0.49	0.45
	D	45.2	56.3	45.8	0.37	0.46	0.37

Notes: a) The measured yield strength of steel tubes is 406 N/mm^2 for MSS1 and MSS2.
b) The yield strength of steel tubes is assumed to be 402.5 N/mm^2 for MSS3.
c) The notional loads are applied at all levels in both directions.
d) The assumed boundary condition to the scaffolds are adopted in the non-linear analyses.

It is shown that the predicted failure loads based on ψ_H are always found to be conservative when compared with the test results. Hence, the proposed approach is applicable to multi-storey modular steel scaffolds with an equivalent geometrical imperfection ψ equal to 0.005.

5. Critical Load Approach

In order to provide a simple method to assess the load carrying capacities of multi-storey modular steel scaffolds, it is proposed to design multi-storey modular steel scaffolds with a method similar to those of real steel columns. It should be noted that in most modern steel codes, the member resistances of steel columns are determined as a function of their elastic buckling loads and their cross-section capacities through a set of semi-empirical column buckling curves, depending on axes of buckling, section types, and magnitude of initial imperfections. Thus, it is proposed in the present approach that the failure loads of the scaffolds should be determined as a function of the elastic critical loads and the cross-section capacities of the *entire* scaffolds through a Perry-Robertson interaction formula. It should be noted that while the Robertson constant, a, in BS5950: Part 1 is equal to 5.5 for cold-formed steel tubes, it is necessary to determine the appropriate value for modular steel scaffolds in the present approach. Moreover, contrary to the other two approaches presented above, no member out-of-straightness and storey out-of-plumbness are incorporated in the geometry of the finite element models as these effects are considered in the Perry-Robertson interaction formula explicitly.

Table 5 presents the predicted failure loads of multi-storey modular steel scaffolds in test series MSS1, MSS2 and MSS3 based on both elastic and real columns together with test results for direct comparison. Moreover, all the results are also presented in terms of strength utilization ratios for easy comparison. Typical modes of failure of all the scaffolds are illustrated in Figs. 4, 5 and 6 for test series MSS1, MSS2 and MSS3, respectively.

It is shown that the predicted failure loads based on elastic columns are found to be un-conservative in some cases when compared with the test results. However, the predicted failure loads based on real columns with the Robertson constant, a, equal to 2.0 are always found to be conservative instead. Hence, the proposed approach is applicable to multi-storey modular steel scaffolds with a Roberson constant at 2.0. The structural adequacy of the proposed approach is illustrated in Fig. 9 where the

Table 5 Failure loads of modular steel scaffolds obtained from Critical Load Approach (CLA)

Test		Failure load per leg obtained from tests	Failure load per leg obtained from analyses		Strength utilization ratio		
			Elastic column	Real column			
		P_{test} (kN)	P_{CLA-E} (kN)	P_{CLA-R} (kN)	χ_{test}	χ_{CLA-E}	χ_{CLA-R}
MSS1		63.4	66.7	58.2	0.46	0.48	0.42
MSS2		53.4	46.4	42.0	0.39	0.34	0.30
MSS3	A	49.7	56.2	49.0	0.43	0.48	0.42
	B	46.8	50.4	44.8	0.38	0.41	0.36
	C	128.5	135.6	118.0	0.46	0.49	0.43
	D	45.2	46.9	42.0	0.37	0.38	0.34

Notes: a) The measured yield strength of steel tubes is 406 N/mm^2 for MSS1 and MSS2.
b) The yield strength of steel tubes is assumed to be 402.5 N/mm^2 for MSS3.
c) The lowest eigenvalues of the scaffolds under the assumed boundary condition are adopted as the elastic critical buckling load.
d) In order to allow for material yielding and initial geometrical imperfection, a Perry-Robertson inter action curve is adopted where the Robertson constant is taken as 2.0.

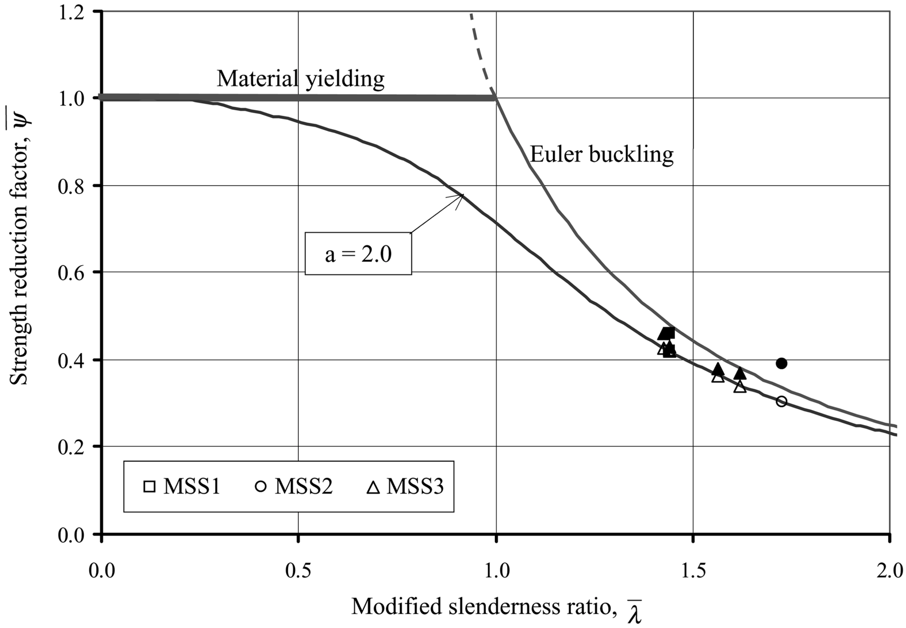


Fig. 9 Column buckling curve for modular steel scaffolds

Notes: ■●▲ denote test data

□○△ denote design values

$$\overline{\lambda} = \sqrt{\frac{P_c}{P_E}} \text{ and } \overline{\psi} = \frac{P}{P_c}$$

where P_c is the compression capacity of the scaffold;

P_E is the elastic critical load of the scaffold obtained from classical eigenvalue analysis;

P is the load carrying capacity of the scaffold.

proposed buckling curve of multi-storey modular steel scaffolds is plotted together with the test results of tests MSS1, MSS2 and MSS3.

6. Other considerations

6.1. Boundary conditions

It should be noted that the load carrying capacities of multi-storey modular steel scaffolds are very sensitive to i) internal restraints such as bracing members, and ii) external restraints such as lateral supports at the top as well as rotational restraints at the bottom of the scaffolds. Although it is demonstrated that all the three approaches are capable to predict the load carrying capacities of the scaffolds satisfactorily, accurate prediction relies heavily on the correct definition of the boundary conditions. It should be noted that it is always conservative to assume the scaffolds to be pin supported at the bottom while free at the top, and this tends to give fairly low load carrying capacities when compared with the test results. For further details on the structural behaviour of multi-storey modular steel scaffolds with different boundary conditions at the top and the bottom of the scaffolds with non-linear finite element analyses, refer to Yu *et al*. (2004).

6.2. Erection tolerances

In order to use modular steel scaffolds with high structural efficiency on construction sites, erection tolerances should be compiled in practice. According to good practice adopted among builders, the verticality of the entire modular steel scaffolds should be plumbed within a tolerance of 25 mm while the maximum out-of-plumbness between any two storeys of scaffolds should not exceed 5 mm. Moreover, the maximum out-of-straightness of each beam or column member should not exceed 5 mm. For further details on erection tolerances, refer to Clause 7.3.2.4 of BS5975 (1996).

7. Conclusions

This paper presents an extensive numerical study on three different approaches in analyzing and designing multi-storey modular steel scaffolds, namely,

a) *Eigenmode Imperfection Approach*,
b) *Notional Load Approach*, and
c) *Critical Load Approach*.

All these three approaches adopt different ways to allow for the non-linear behaviour of the scaffolds in the presence of initial geometrical imperfections, and the study aims to develop a simplified and yet reliable design approach for safe prediction on the load carrying capacities of multi-storey modular steel scaffolds.

It is demonstrated that all these three approaches are able to predict the structural behaviour of multi-storey modular steel scaffolds with satisfactory accuracy when compared with the results of 13 full scale tests. Moreover, it is interesting to note that the adequacy of each of the approaches depends primarily on different parameters:

a) the initial geometrical imperfection of imperfect models, δ, in the Eigenmode Imperfection Approach
b) the equivalent geometrical imperfection, ψ, in the Notional Load Approach, and
c) the Robertson constant, a, in the Perry-Robertson interaction formula in the Critical Load Approach.
The values of all these parameters are determined after careful calibration against test data.

It is important to emphasize that although the proposed approaches have been carefully calibrated against test data, they are only applicable to those modular steel scaffolds with essentially similar member configurations and geometrical dimensions to those given in Fig. 2, and similar mechanical properties to those given in Table 1. Nevertheless, the approaches provide alternate simple and yet effective means for the strength prediction of multi-storey modular steel scaffolds, and their general applicability should be further verified with more test data.

Acknowledgements

This research project leading to the publication of this paper is supported by the Research Committee of the Hong Kong Polytechnic University (Research Project No. G-V849).

References

British Standards Institution. BS5950: Structural use of steelwork in building. Part 1: Code of practice for design – Rolled and welded sections, 2000.

British Standards Institution. BS5975: Code of practice for falsework, 1996.

Chan, S.L. and Zhou, Z.H. (1998), "On the development of a robust element for second-order Non-linear integrated design and analysis (NIDA)", *J. of Constructional Steel Research*, **47**(I-II), 169-190.

Chung, K.F. and Yu, W.K. (2003), "Experimental and theoretical investigations on modular steel scaffolds", *Proc. of Technical Seminar on Metal Scaffolding (Falsework) - Design, Construction & Safety*, the Hong Kong Institute of Steel Construction, Hong Kong, 13-23.

European Standard. Eurocode 3: Design of steel structures. Part 1-1: General rules and rules of buildings, 1993 & 2003.

Godley, M.H.R. and Beale, R.G. (1997), "Sway stiffness of scaffold structures", *The Structural Engineer*, **75**(1), 4-12.

Godley, M.H.R. and Beale, R.G. (2001), "Analysis of large proprietary access scaffold structures", *Proc. of the Institution of Civil Engineers, Structures and Buildings*, **146**, 31-39.

Gylltoft, K. and Mroz, K. (1995), "Load carrying capacity of scaffolds", *Struct. Eng. Int.*, **1**, 37-42.

Holmes, M. and Hindson, D. (1979), "Structural behaviour of load bearing falsework", *Proc. of Institution of Civil Engineers*, **67**, 721-741.

Huang, Y.L., Chen, H.J., Rosowsky, D.V. and Kao, Y.G. (2000a), "Load-carrying capacities and failure modes of scaffold-shoring systems, Part I: Modeling and experiments", *Struct. Eng. Mech.*, **10**(1), 53-66.

Huang, Y.L., Kao, Y.G. and Rosowsky, D.V. (2000b), "Load-carrying capacities and failure modes of scaffold-shoring systems, Part II: An analytical model and its closed-form solution", *Struct. Eng. Mech.*, **10**(1), 67-79.

Liew, J.Y.R., White, D.W. and Chen, W.F. (1994), "Notional-load plastic-hinge method for frame design", *J. of Struct. Eng.*, **120**(5), 1434-1454.

Lightfoot, E. and Oliveto, G. (1977), "The collapse strength of tubular steel scaffold assemblies", *Proc. of Institution of Civil Engineers*, **63**, 311-329.

Milokovic, B., Beale, R.G. and Godley, M.H.R. (2002), "Determination of the factors of safety of standard scaffolds structures", *Proc. of the Third Int. Conf. on Advances in Steel Structures*, **1**, 303-310.

Peng, J.L. (1994), "Analysis models and design guidelines for high-clearance scaffold systems", Ph.D. Dissertation, School of Civil Engineering, Purdue University.

Peng, J.L., Pan, A.D.E., Chen, W.F., Yen, T. and Chan, S.L. (1997), "Structural modeling and analysis of modular falsework systems", *J. of Struct. Eng.*, **123**(9), 1245-1251.

Peng, J.L., Pan, A.D., Rosowsky, D.V., Chen, W.F., Yen, T. and Chan, S.L. (1996), "High clearance scaffold systems during construction I. Structural modeling and modes of failure", *Eng. Struct.*, **18**(3), 247-257.

Weesner, L.B. and Jones, H.L. (2001), "Experimental and analytical capacity of frame scaffolding", *Eng. Struct.*, **23**(6), 592-599.

Yu, W.K. (2004a), "Structural behaviour of tubular skeletal framework", Ph.D Thesis, the Hong Kong Polytechnic University.

Yu, W.K. (2004b), "An Investigation into structural behaviour of modular steel scaffolds", *J. of Steel and Composite Structures*, **4**(3), 211-226.

Yu, W.K., Chung, K.F. and Chan, S.L. (2002), "Structural stability of modular steel scaffolding systems", *Proc. of the Second Int. Conf. on Stability and Dynamic,* Singapore, 418-423.

Yu, W.K., Chung, K.F. and Chan, S.L. (2004), "Structural instability of multi-storey door-type modular steel scaffolds", *Eng. Struct.*, **26**(7), 867-881.

SC