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## Initial Geometrical Imperfections in Three-Storey Modular Steel Scaffolds

W.K. Yu<sup>1</sup>, K.F. Chung<sup>2</sup> and S.L. Chan<sup>3</sup>

### Abstract

Modular steel scaffolds are commonly used as supporting scaffolds in building construction. They are highly susceptible to global and local instability, 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. Initial geometrical imperfections are considered to be very important to the structural behaviour of multi-storey modular steel scaffolds. This paper presents an extensive numerical investigation on three different approaches in analyzing and designing multi-storey modular steel scaffolds, namely, a) *Notional Load Approach*, b) *Eigenmode Imperfection Approach*, and c) *Critical Load Approach*. It should be noted that all these three approaches adopt different ways to allow for the presence of initial geometrical imperfections in the scaffolds when determining their load carrying capacities. Moreover, their suitability and accuracy in predicting the structural instability of typical modular steel scaffolds are presented and discussed in details.

### Keywords

Modular steel scaffolds, initial geometrical imperfections, Notional load approach, Eigenmode imperfection approach, Critical load approach.

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## Introduction

Modular steel scaffolds are temporary structures and they are commonly used as supporting scaffolds in building construction. They comprise of scaffolding frames (modular units) which are linked-up with cross-bracing members, and 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, they are highly susceptible to global and local instability, and both out-of-plane and in-plane buckling of the scaffolding frames are possible, depending on steel grades, member dimensions and configurations, as well as loading and boundary conditions. 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 the load carrying capacities of these scaffolds in practical conditions as the real boundary conditions are very different from those provided in test conditions. Structural failure of these scaffolds occurs from time to time due to inadequate design, poor installation and over-loads on sites (Peng et al., 1997; Chung & Yu, 2003).

In conventional design, moments and forces in members are determined from linear analysis and the members are then checked against section capacities and member resistances according to relevant specifications. However, in modular steel scaffolds with very slender members, the column members are highly susceptible to both global and member buckling while their structural instability is difficult to be assessed due to complicated member configurations. Moreover, as lateral deformations of these scaffolds are often apparent well before failure, non-linear analysis is normally required to predict the structural behaviour of these scaffolds to allow for additional moments induced by both  $P-\delta$  and  $P-\Delta$  effects. 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 in practice, 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.

Consequently, it is highly desirable to provide practical analysis and design methods on multi-storey modular steel scaffolds for structural engineers to ensure their safe and effective use in building construction.

## Scope of work

This paper presents an extensive numerical investigation on the structural behaviour of three-storey modular steel scaffolds. The test results of 7 three-storey modular steel scaffolds conducted by Weesner & Jones (2001) are first presented. Three analysis and design approaches which incorporate initial geometrical imperfections of modular steel scaffolds in different ways are adopted to back analyze the structural behaviour of the test specimens as follows:

- *Notional Load Approach (NLA)*
- *Eigenmode Imperfection Approach (EIA)*
- *Critical Load Approach (CLA)*

Moreover, their suitability and accuracy in predicting the structural instability of typical multi-storey modular steel scaffolds are presented and discussed in details. It should be noted that during the back analysis of the three-storey modular steel scaffolds, the assumed boundary condition (Yu et al., 2004) are adopted in order to correlate the numerical results directly with the test results. Other important parameters to the structural stability of modular steel scaffolds include restraints provided to the top and the bottom of scaffolds, cross-bracings within modular height, and splices between column members (Yu, 2004a).

### **Experimental Investigation of Modular Steel Scaffolds from Literature**

A total of 7 three-storey modular steel scaffolds were tested to failure by Weesner & Jones (2001), and the test results were presented in this paper in order to provide data for comparison with numerical results. Four different types of modular steel scaffolds, namely, test series A, B, C and D, were tested; refer to Figure 1 for detailed dimensions of the scaffolding frames.

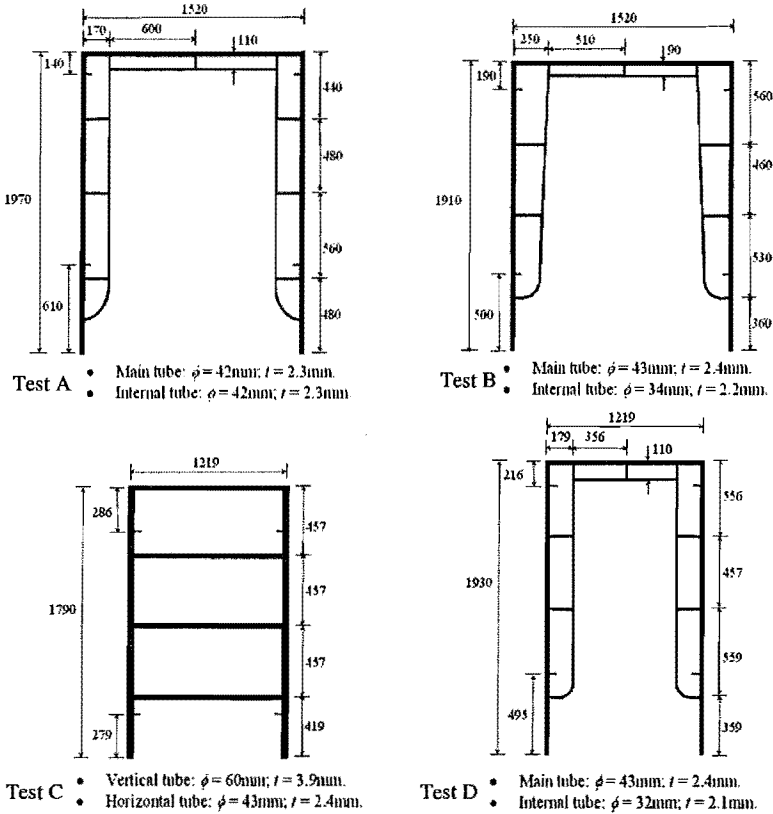


Fig. 1 Geometry scaffolding frames

The nominal values of both external diameters and thicknesses are also presented in Table 1 while the nominal yield strength and the Young's modulus are taken to be  $350 \text{ N/mm}^2$  and  $205 \text{ kN/mm}^2$  respectively. The overall height of all the test specimens is approximately equal to  $5900 \text{ mm}$ , including a jack extension of  $152 \text{ mm}$  at the base.

Table 1 Failure loads of modular steel scaffolds obtained from tests

Test	Diameter $D$ (mm)	Thickness $t$ (mm)	Area $A$ (mm <sup>2</sup> )	Maximum failure load per leg (kN)	Averaged failure load per leg $P_{test}$ (kN)	Strength utilization ratio $\chi_{test}$
A1	42	2.3	290	48.8	49.7	<b>0.43</b>
A2	42	2.3	290	50.5		
B1	43	2.2	306	46.1	46.8	<b>0.38</b>
B2	43	2.2	306	47.5		
C1	60	3.9	687	125.6	128.5	<b>0.46</b>
C2	60	3.9	687	131.3		
D	43	2.4	306	45.2	45.2	<b>0.37</b>

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 nominal yield strength and equal to 350 N/mm<sup>2</sup>, and the actual yield strength is taken to be equal  $1.15 \times 350 \text{ N/mm}^2 = 402.5 \text{ N/mm}^2$  according to Weesner & Jones (2001).

The averaged failure loads per leg in test series A, B, C and D are also presented in Table 1 together with the corresponding strength utilization ratios, i.e. the buckling failure load divided by the cross-section capacity of the column member. It should be noted that lateral displacements with significant out-of-plane member buckling (in the plane of the bracing members) were observed in test series A, B and D while torsional buckling was observed in test series C.

### Analysis and Design Incorporating Initial Geometrical Imperfections

In order to perform non-linear analysis on modular steel scaffolds, the concept of "non-linear integrated design and analysis" is introduced (Chan & Zhou, 1998). Finite element models using the advanced structural analysis software *NIDA* are established to evaluate the load carrying capacities of these three-storey modular steel scaffolds, and the predicted failure loads are the applied loads at which the stresses of extreme fibers of the cross-section reach the design strength under combined compression and bending. It should be noted that according to Weesner, the actual yield strength is taken to be 1.15 times the

nominal value, i.e.  $1.15 \times 350 = 402 \text{ N/mm}^2$ , which is adopted in all finite element analyses.

In order to predict realistic load carrying capacities and buckling modes of these modular steel scaffolds under test conditions, a reference boundary condition with the extensional stiffness of  $100 \text{ kN/m}$  at the top and the rotational stiffness of  $10 \text{ kNm/rad}$  at the bottom of the scaffolds (Yu et al., 2004) is employed in all models. The finite element models of the three-storey modular steel scaffolds are presented in Figure 2. In general, the effects of initial geometrical imperfections are considered to be important to the structural stability of multi-storey modular steel scaffolds, and initial geometrical imperfections may be incorporated as follows:

### Notional Load Approach

The effect of initial geometrical imperfection on the structural behaviour of framed structures is conventionally allowed for through the use of notional loads. However, only storey out-of-plumbness is accounted for indirectly in this approach through the deformations caused by the notional loads during structural analysis. As a separate procedure, the effect of member out-of-straightness is incorporated directly in member design through column buckling curves of imperfect columns.

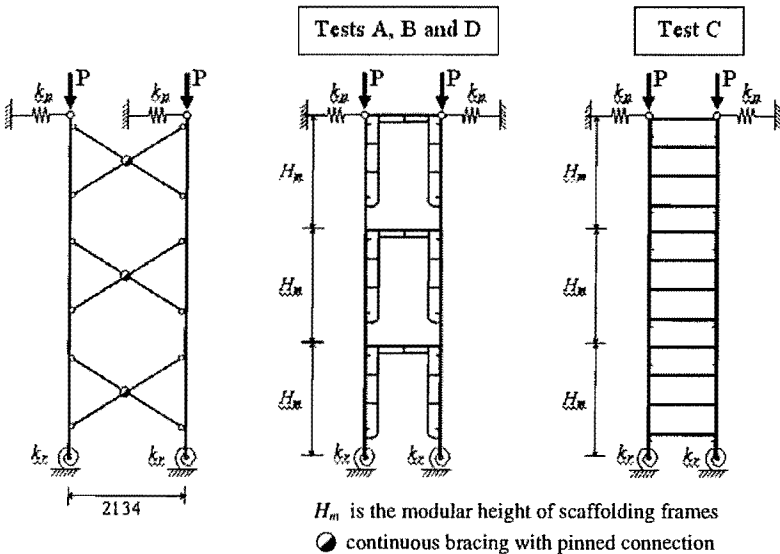


Fig. 2 Finite element models of three-storey modular steel scaffolds

The three-storey modular steel scaffolds with different member configurations are analyzed; all the elements are straight, i.e. without member out-of-straightness. Although Peng et al. (1996) recommends that notional loads equal to 0.1% of vertical loads may be applied to the mid-height of two-storey scaffolds, it is found to be necessary that notional loads at all levels equal to 0.5% of total vertical loads according to BS5950: Part 1 (BSI, 2000) should be used for the analyses of the modular steel scaffolds, as shown in Figure 3.

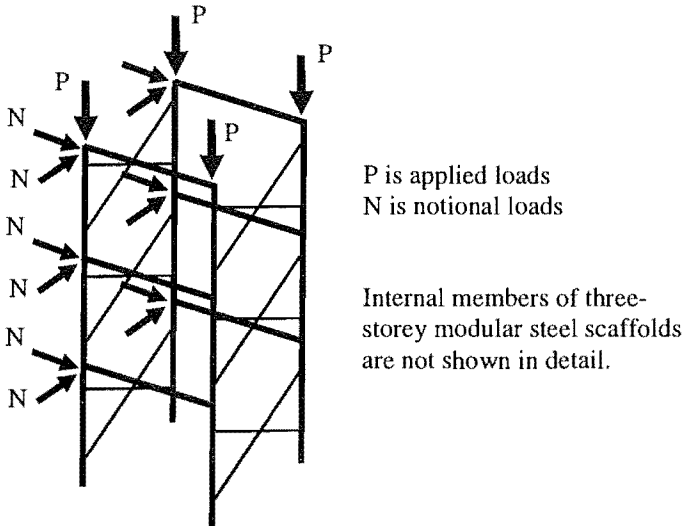


Fig. 3 Notional loads for storey out-of-plumbness

- Linear analysis

The conventional linear analysis ignores any geometrical non-linearities and associated instability problems. However, the linear analysis with notional load is considered to be a simplified method for the analyses of modular steel scaffolds with initial geometrical imperfections. The predicted strength utilization ratios of various modular steel scaffolds are summarized in Table 2.

- Non-linear analysis

Due to the complicated member configurations and high slenderness of the modular steel scaffolds, non-linear analysis is always required. The undeformed shapes of these scaffolds and their predicted failure modes are shown in Figure 4, and the typical failure mode of modular steel scaffolds is



shown to be torsional buckling. The predicted strength utilization ratios of various modular steel scaffolds are presented in Table 2, and they are found to be fairly close to the test results. Although the notional load ratio at 0.5% is suitable for most cases, the load carrying capacities of modular steel scaffolds are significantly affected by the boundary conditions of these scaffolds and the locations of the notional loads (Chu et al., 2002).

Table 2 Summary of strength utilization ratios from back analyses

Test		Notional Load Approach (NLA)		Eigenmode Imperfection Approach (EIA)		Critical Load Approach (CLA)	
		Linear analysis	Non-linear analysis	$\delta = H_n/1000$	$\delta = 3H_n/1000$	Elastic	Interaction
	$\chi_{test}$	$\chi_{NLA-L}$	$\chi_{NLA-NL}$	$\chi_{EIA-h}$	$\chi_{EIA-3h}$	$\chi_{CLA-E}$	$\chi_{CLA-I}$
A	<b>0.43</b>	0.54	0.43	0.42	0.37	0.48	0.42
B	<b>0.38</b>	0.63	0.39	0.38	0.35	0.41	0.36
C	<b>0.46</b>	0.75	0.45	0.49	0.46	0.49	0.43
D	<b>0.37</b>	0.61	0.37	0.36	0.32	0.38	0.34

Notes: a) The yield strength of steel tubes is assumed to be 402.5 N/mm<sup>2</sup>.

b) The modular steel scaffolds under the reference boundary condition are adopted.

c) NLA - The notional loads are applied at all levels and equal to 0.005  $\sum P$  each.

d) EIA - The lowest eigenmodes of the structures under modified boundary condition are adopted as the initial geometrical imperfection.

e) CLA - The lowest eigenvalues of the structures under modified boundary condition are adopted as the elastic critical buckling load. In order to allow for material yielding and initial geometrical imperfection, a Perry-Robertson interaction curve is adopted where the Robertson constant is taken as 2.0.

### Eigenmode Imperfection Approach

In order to simulate both the member out-of-straightness and the storey out-of-plumbness of multi-storey modular steel scaffolds, standard eigenvalue analyses on the models under the assumed boundary condition are performed (Yu, 2004b). The eigenmodes with the lowest eigenvalues are then extracted and superimposed onto the geometry of the perfect models. It should be noted that the eigenmodes define the relative deformations of all the members explicitly, coupling up the member out-of-straightness and the storey out-of-plumbness. Thus, the magnitude of the initial geometrical imperfection of imperfect models,  $\delta$ , may be readily assigned according to fabrication and erection tolerances in

practice. Two practical limiting values of  $\delta$  are assigned in the current study for comparison:

$$\begin{aligned}\delta &= 1 / 1000 \text{ of the modular height of scaffolding frames as the lower} \\ &\quad \text{bound value} \\ &= H_m / 1000\end{aligned}$$

or

$$\begin{aligned}&= 1 / 1000 \text{ of the overall height of three-storey modular steel} \\ &\quad \text{scaffolds as the upper bound value} \\ &= 3 H_m / 1000\end{aligned}$$

where

$H_m$  is the modular height of scaffolding frames.

Table 2 also presents the strength utilization ratios of test series A, B, C and D together with the test results while the corresponding modes of failure are illustrated in Figure 4 for direct comparison. It is shown that the predicted failure loads are close to the test results with similar buckling mode shapes for test series A, B and D. However, the predicted buckling mode shape for test series C is shown to be bi-axial buckling, i.e. buckling with significant in-plane and out-of-plane deformations. Moreover, it is shown in Table 2 that the strength utilization ratios of modular steel scaffolds decrease by 10% when the magnitude of the initial geometrical imperfection  $\delta$  is increased by a factor of 3.

### Critical Load Approach

In order to provide a simple method to assess the failure loads of multi-storey modular steel scaffolds, it is proposed that the scaffolds may be designed in a similar method for real 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 initial imperfections. Thus, it is proposed that the failure loads of three-storey modular steel scaffolds may be determined as a function of the elastic critical loads and the cross-section capacities of the scaffolds through a Perry-Robertson interaction formula. The Robertson constant is equal to 2.0 for cold-formed steel tubes according to BS5950: Part 1 (BSI, 2000), while a value of 2.0 is taken for the modular steel scaffolds in this approach after comparison with test data.

The failure loads of the modular steel scaffolds are evaluated through the use of column buckling curves for both elastic and real columns, and the corresponding strength utilization ratios are also presented in Table 2 while the predicted failure modes are presented in Figure 4. It is shown that the strength utilization ratios based on elastic columns are significantly larger than the test results as the effects due to initial geometrical imperfection and material yielding are neglected. For those strength utilization ratios based on real columns, they are shown to compare favorably with the test results.

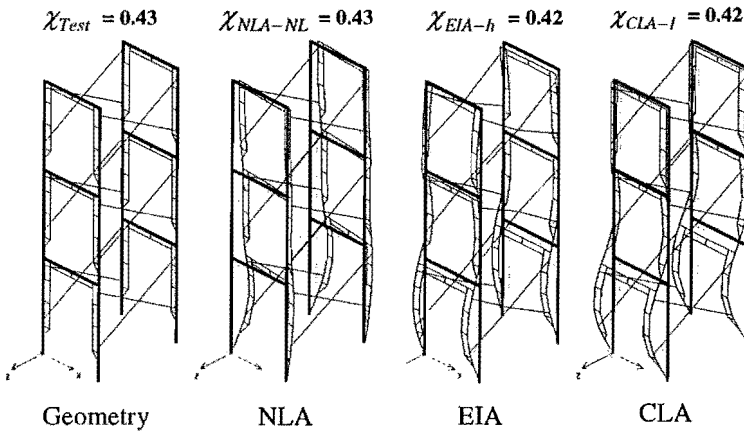


Fig. 4a Predicted failure modes of Test A

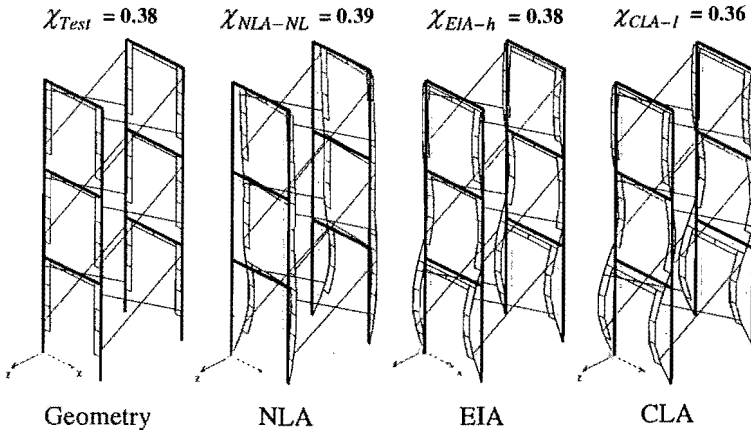


Fig. 4b Predicted failure modes of Test B

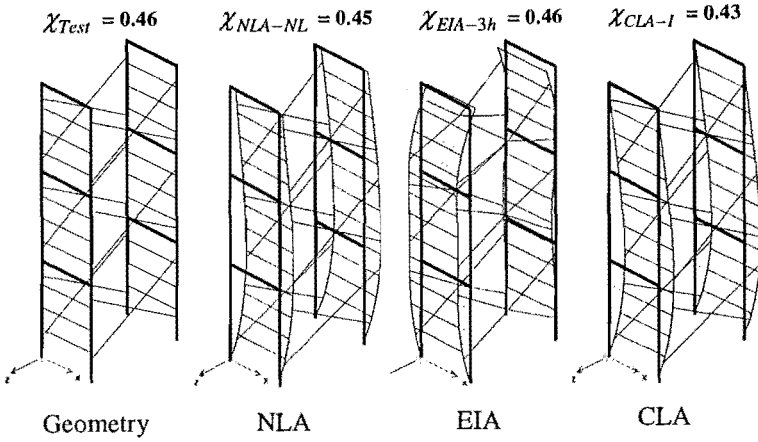


Fig. 4c Predicted failure modes of Test C

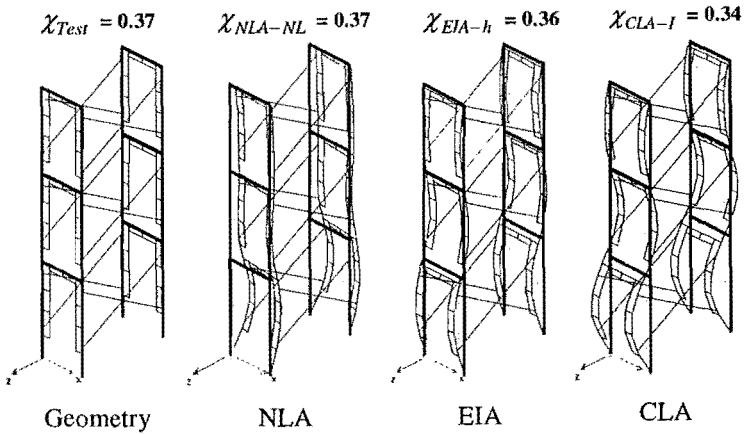


Fig. 4d Predicted failure modes of Test D

## Conclusions

This paper presents an extensive numerical investigation on initial geometrical imperfection through three different approaches, namely, a) *Notional Load Approach*, b) *Eigenmode Imperfection Approach*, and c) *Critical Load Approach*. A reference boundary condition is adopted in the back analyses of modular steel scaffolds in order to predict the structural performances of these scaffolds under test conditions. It is found that among all, the Eigenmode Imperfection Approach tends to give conservative and yet economical results. However, it should be noted that the load carrying capacities of modular steel scaffolds are very sensitive to both the types and the magnitudes of restraints provided by attachments at the top and the bottom of these scaffolds. Structural engineers are thus encouraged to analyze and design modular steel scaffolds with extreme care to ensure safety on sites.

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