因公出國報告書

96 年 9 月 22 日

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及職稱</td><td>國立雲林科技大學 營建工程系
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<tr><td>時間
會議地點</td><td>96 年 7 月 26 至 8 月 3 日
1.英國(牛津 Oxford)
2.英國(曼徹斯特 Manchester)</td><td>本會核定
補助文號</td><td>計畫編號： 95-1186、95-215
94-251、94-124</td></tr>
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(英文) 6th International Conference on Steel and Aluminium Structures
2. (中文) 第三屆鋼結構及鋼骨鋼筋混凝土結構國際研討會(地點:曼徹斯特)
(英文) Third International Conference on Steel and Composite Structures</td></tr>
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報告內容應包括下列各項：

一、 參加會議經過

為廣泛的與歐美日等先進國家進行鋼結構與鋁結構研究及應用的成果交流,『第六屆鋼結構及鋁結構國際研討會』(6th International Conference on Steel and Aluminium Structures）特於 2007 年 7 月 24 日～27 日，於英國牛津之 Oxford Brookes University 舉辦此次國際性研討會。參與本次研討會成員，除了各國大學從事鋼結構及鋁結構的研究學者外，亦包含鋼結構及鋁結構領域之設計工程師及製造者、建築師、相關研究機構人員、博士後研究人員等。本次研討會討論主題，主要包含下列部分：

- 鋼結構與鋁結構之分析與設計
- 銲接及螺栓等結合行為
- 鋼骨鋼筋混凝土結構
- 地震及動態分析
- 薄殼結構
- 冷軋型鋼結構
- 鋼構材與鋁構材建築使用及抗火
- 樑柱行為
- 鷹架與細長結構
- 不銹鋼結構
- 營造技術

- 專家系統

此次研討會主辦單位在人員接待服務及論文發表等都甚有經驗，場地均有電腦及單槍投影機等器材設備，也有專人作技術服務，以方便論文發表者能有最佳之成果展示。本研討會全部使用筆記型電腦搭配單槍投影設備發表論文，已完全沒有使用幻燈機、投影機等落伍器材。本研討會集結了國際研究成果並發表多篇文章，均刊載於研討會論文集中。（本研討會個人有發表論文。）

另外一場與鋼結構領域相關的國際性研討會：『第三屆鋼結構及鋼骨鋼筋混凝土結構國際研討會』（Third International Conference on Steel and Composite Structures），今年在英國曼徹斯特舉辦，舉辦時間為於 2007 年 7 月 30 日～8 月 1 日，恰於『第六屆鋼結構及鋁結構國際研討會』7 月 27 日結束後兩天舉行。本次研討會研討主要討論下列主題：

- 建築物、橋樑、樑柱、結合、空間結構、版殼、整體建築、混合結構、非金屬結構、複合材料應用
- 挫屈及穩定、防火功能、疲勞、破壞力學、震動及控制、防震功能、防風工程
- 分析、試驗、數值計算
- 法規、案例研究、鋼結構施工、製造及組搭
- 教育訓練、維護及安全監測、最佳化設計

同樣的本次研討會，場地也都使有電腦及單槍投影設備等器材，方便論文成果發表且能以最佳方式展示。本研討會集結了國際研究成果並發表多篇論文，均刊載於研討會論文集中。（本研討會個人雖沒有發表論文，但有與會並參加交流及討論，並自費購買研討會論文集。）

二、與會心得

(I). 『第六屆鋼結構及鋁結構國際研討會』發表論文

本次『第六屆鋼結構及鋁結構國際研討會』為國際性鋼結構技術研討會，邀請歐美日鋼結構領域學有專精的學者、研究員及工程師參加，參與研討會的人員，除各國大學從事鋼結構研究的學者及學術單位的研究員外，多半為歐洲地區鋼結構工程界之工程師及工程實務界人員。本研討舉辦主要是提供國際鋼結構研究成果的交流，為方便並有效將學術成果落實在工程界，研討會主要採用英文方式發表，以達充分溝通交流的目的。

本次研討會個人與香港理工大學土木與結構系陳紹禮教授共同發表論文，發表的論文題目為『鷹架系統以「虛擬力」及「非完美挫屈模式」之進階穩定設計』（Advanced stability design of scaffolding systems by “Notional force” and by “Buckling mode as imperfection mode” methods），本篇論文為 keynote paper，本次研討會共於兩百多篇論文中選取 6 篇論文為 keynote paper。陳紹禮教授專精於數值分析，而個人則專精於鋼管鷹架，因此此論文分成兩部分發表，數值部分由陳教授為主，鋼管鷹架範例分析及試驗則由筆者負責，二人共同發表此篇論文，並作與會提問的回應。

筆者於本次研討會所提出之論文，內容主要在探討數值分析中，以「虛擬力」及「非完美挫屈模式」為基礎進而用於實際工程設計之「進階穩定設計」，此種設計選用的研究對象為營造階段之假設構造物「鋼管鷹架」。由於這種先進之分析方式屬於非線性分析，工程實務界

對線性分析較為熟悉，而對非線性分析較為排斥，故有必要證明此種分析的可靠性以說服工程界，因此考慮採用營造施工中的臨時結構「鋼管鷹架」來進行研究。這是因為鋼管鷹架較為便宜，故可以大量的全尺寸試驗來驗核這些理論的正確性，筆者為國內、外從事鋼管鷹架方面研究的少數專家，對鋼管鷹架力學行為較為熟悉，故於此次研討會發表此篇論文。

本篇論文『鷹架系統以「虛擬力」及「非完美挫屈模式」之進階穩定設計』主要介紹兩種數值分析模式：「虛擬力」及「非完美挫屈模式」用於非線性分析，依其概念所發展的軟體為 NIDA，此軟體已可進行商業上應用，由於應用的對象多為建築鋼結構體，這些龐大的結構體無法驗核正確與否，故採用鋼管鷹架作為驗核對象。

本篇論文內容概略分成三大部分：一為介紹「虛擬力」及「非完美挫屈模式」應用在穩定分析的理論架構，二為介紹其應用在室內鋼管鷹架的試驗結果，三為應用在全尺寸室外載重試驗的驗核。本論文理論架構上可同時考慮 P-Δ及 P-δ effects，但是不用考慮傳統鋼結構設計之有效長度係數 K，這將更接近實際結構體的破壞，由鋼管鷹架室內試驗及室外試驗結果的驗核，可確定本研究的正確性，此外本論文提出的設計概念可取代舊有的設計方法，由於本研究成果頗具前瞻性也顯示其特有價值，因此本論文被選為此次研討會六篇 keynote paper 的其中一篇。

檢附本篇論文於本報告後，詳見附件一。

(II). 參加『第三屆鋼結構及鋼骨鋼筋混凝土結構國際研討會』

由於『第三屆鋼結構及鋼骨鋼筋混凝土結構國際研討會』距『第六屆鋼結構及鋁結構國際研討會』僅差兩日，且研討會內容與個人研究領域相關，能參與學習實屬機會難得，故於第一場研討會結束後，即赴 Manchester 參加此第二場研討會。此研討會個人沒有繳交報名費，僅與會參加並和其他參會者作學術交流，主要是因為英國研討會報名費頗貴，報名費約需要新台幣三萬多元，沒有這筆預算。

(III). 與會心得

綜合上述兩場研討會的參與，筆者發現此二研討會中，亞洲地區日本及大陸有非常多的學者參加，但是來自台灣的僅有筆者一人。此外，來自美國的學者也不多，許多美國有名的鋼結構大師級學者未見一人，似乎美國與歐洲鋼結構研究的交流不是很熱絡。

筆者驚訝的發現，在英鎊未與歐元整合的情況下，英國鋼結構設計規範卻已與歐盟規範整合了。不過本次研討會參與人員，歐洲除了法國外幾乎各國都有學者參加，此可看出歐盟規範建立主要以英國與德國著力最深。非線性分析及設計在歐盟規範下，工程界已普遍使用，顯示鋼結構設計規範在歐洲已逐漸整合成功，目前歐盟推廣重點，是如何在各大學中落實非線性分析及設計的教育工作。

筆者與國內其他鋼結構領域研究者多為留學美國，美國高科技產業如國防、電腦、生物科技等領先全球，但化工、機械、土木等傳統產業卻是相對落後歐洲，如公認最好的汽車為 Benz、BMW，最好的手錶由瑞士出產。美國在傳統營造土木之鋼結構設計領域已明顯落後歐洲一大段距離，歐洲已將非線性分析及設計應用在工程中，而我們鋼結構設計跟隨美國，還在教授學生基礎之線性及構件為主的設計。

筆者專長在鋼結構領域，因緣際會由於需要解決鋼管鷹架倒塌問題，必須使用非線性分析，因而走入此非線性研究領域。筆者在國內想要推動此種較為合理之設計方式，多年來充滿著無力感，不過此次參加這兩場在歐洲舉辦之鋼結構研討會，卻發現有這麼多歐洲研究者與筆者理念相通，未來可與這些歐洲研究者交流及合作，必要時也可將歐盟設計規範在國內工程界推廣及介紹，實在是此行最大的收穫。

三、總結與建議

『第三屆鋼結構及鋼骨鋼筋混凝土結構國際研討會』及『第六屆鋼結構及鋁結構國際研討會』主題涵蓋了鋼結構分析、設計、製作、安裝及材料等方面的先進經驗與最新研究成果，此外，對於鋼結構的新理論、新技術及新材料、工程中應用等問題也都作探討。藉由參加這兩次研討會除了可以將研究成果與大家共同分享外，更可以瞭解目前國際鋼結構領域各方面研發、設計、工程應用等發展現況及遇到的瓶頸，可作為未來專案計畫執行之參考。此行不但參與研討會發表論文，並與同領域之同儕相互交換研究心得與經驗，相當有收穫。

四、攜回資料名稱及內容

攜回此次『第三屆鋼結構及鋼骨鋼筋混凝土結構國際研討會』及『第六屆鋼結構及鋁結構國際研討會』之論文集與大會提供的相關資料。

(附件 一)

<u>6th International Conference on Steel and Aluminium Structures</u>

Advanced stability design of scaffolding systems by "Notional force" and by "Buckling mode as imperfection mode" methods

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ABSTRACT

This paper introduces a new computer-based design method for finding the design load of scaffolding systems. Both the methods of notional force and of using buckling mode as imperfection mode are used for the present studies. The method is based on the rigorous second order analysis, accounting for P-Δ and P-δ effects and their imperfections without assuming any effective length nor application of notional force. The proposed method is employed to predict the collapse loads of seven tested 3-storey steel modular metal scaffolding units. It was found that the <u>n</u>otional <u>f</u>orce <u>m</u>ethod (NFM) is more economical or less conservative than the <u>b</u>uckling mode as <u>i</u>mperfection mode <u>m</u>ethod (BIM).

Keywords: scaffolding, P-Δ and P-δ effects, effective length, imperfection, notional force, connection stiffness

INTRODUCTION

Metal scaffolding is a common type of temporary engineering structures used worldwide with a relatively high number of collapses reported every year. Design code, EN 12811-1[1] gives rules for structural design of this special structural form, specifying that factors such as imperfection, connection stiffness and stiffness of jack base plate etc, which have significant influence on the structural behavior of a metal scaffolding system, should be considered in the structural analysis. However, the methodology of analyzing this sort of slender structures has not been specified and this paper is aimed to demonstrate the application of the code by the use of a curved element with varying curvature against axial force to simulate initial imperfection and buckling in members.

Many researchers assert that the design of metal scaffolding is based on experiments with judgment. It is complicated and unreliable to accurately determine the effective length factor (L_e/L) as it depends on the number of bays, storey height, connection stiffness, stiffness of the jack base, bracing details and applied loading. The accuracy of the assumed effective length is also in doubt. Beale and Godley[2] used the elastic buckling load to determine the effective length of the most critical member in their design method in order to improve the accuracy of the effective length.

It must emphasized that the elastic critical load factor (λ_{cr}) in BS 5950(2000)[3] does not consider factors such as material yield stress, initial imperfection and disturbing forces and it should be merely used as a system stability indicator. Interestingly, most scaffolding systems have an elastic buckling load factor less than 4 indicating a second-order analysis should be used in place of the effective length method.

The suggested requirements for an acceptable design method are that the predicted buckling resistance should be smaller but close to the tested failure loads. This implies that the predicted load should be conservative and economical that the design load should be slightly below the tested load. An expectation that the theoretical load resistance is the same as the tested failure load is considered as unrealistic since each scaffolding system has different and random pattern and magnitude of imperfections that even two appearingly identical scaffolds will have different tested failure loads. The assumption of imperfection pattern and magnitude for scaffolding system then become crucial in determining the design load of a scaffolding system and this paper is aimed for a safe and yet economical method for the design. In line with the above observation that the structural system is sensitive to the imperfection, the paper investigates the difference on the use of two imperfection approaches, namely as the notional force method (NFM) and taking buckling mode as the imperfection mode method (BIM). The first approach has been studied by Chan et. al[4] with their predicted buckling loads compared with the series of tests by Jones and Weesner[5] This paper further compares the results by the use of buckling mode as imperfection mode against the tested results and the results by Chan et. al[4] who used 1% vertical load as notional force, compared with the present more economical method of taking 0.5% vertical load as the notional force. In design codes[4,6], 1% is normally recommended for temporary structures while 0.5% is recommended for permanent structures.

NOTIONAL FORCE METHOD (NFM)

Chan et.al[4] proposed a notional force method (NFM) for design of scaffolding systems. The validity of the proposed method is confirmed by comparison with test results of seven 3-storey steel scaffolding units and for the design of a 30m x 20m x 1.3 m 3-dimensional scaffolding system. In the proposed method, it is not necessary to assume or compute the effective length of which the effect is automatically included in the analysis. The external force and moments acting on any section along the member length and allowing for P-δ and P-Δ effects, is then calculated and compared directly with the sectional strength of the member in the system. Unlike the elastic critical load analysis (or the eigen-buckling analysis) providing an upper bound solution, the proposed method traces the deflection in the load-deflection path and study the complete response of the structure under an increasing load. In addition, the effect of imperfection on the modular metal scaffolding is simulated using a refined notional force method which is useful if software does not allow input of elastic critical mode as imperfection mode for the complete frame.

The deformations of the scaffold tower frames increase the lateral deflections or the P-Δ effect of the structure. The P-δ and P-Δ effects are significant in a high storey and slender structure when subjected to large axial loads. Such destabilizing effects will inevitably reduce the load capacity of a metal scaffold. For scaffolding systems, the use of 1% notional force was noted to produce consistent but slightly conservative results against the tested failure loads by Jones and Weesner[5] and therefore it was recommended by Chan et. al[4] as a reliable design parameter.

When the concept is applied to the design of a frame with restraints at top and at base, the notional force is applied at the mid-height in order to create a system imperfection with maximum magnitude at mid-height level. When using a single number of scaffolding unit such that no junction of scaffolding units exists near the mid-height of the scaffold, the notional force was suggested by Chan et. al[4] as split between the junctions immediate above and below the mid-height of the system. This approach simplifies significantly the analysis and design procedure with obtained result close to a well-controlled laboratory test.

THE BUCKLING MODE AS IMPERFECTION MODE METHOD (BIM)

Stability problems can be categorized as elastic critical load and second order analyses. The first method assumes the structure remains undeformed until buckling occurs and the second method traces the equilibrium path of a structure. Many researchers use the first approach for stability analysis of a scaffolding system whilst others adopt the second order method which should account for nodal coordinate change as well as change in member curvature. However, a proper second-order analysis for practical structures should also allow for imperfections in the frame and in the member of which the latter one adopts the curved element as imperfect member. For global frame imperfection, the elastic buckling mode from the first method mentioned above can be used as the imperfection mode with an amplitude taken as $\frac{Height}{500}$ recommended in code[6]. This elastic critical load mode can be obtained as the eigen-vector of the following non-trivial equation of the tangent stiffness.

$$| K_L + \lambda_{cr} K_G | = 0 \tag{1}$$

in which K_L and K_G are respectively the linear and geometric stiffness matrix and λ_{cr} is the elastic critical (bifurcation) load factor.

As a more refined method of analysis, the second-order analysis traces the load vs deflection path of a structure by applying incrementally the load onto a structure with large deflection P-Δ and P-δ effects considered. The corresponding increments of displacement are calculated from the tangent stiffness and the incremental displacements are accumulated to obtain the total final displacements. The amplitude of the imperfect initial geometry is taken as one-500th of the height of the scaffold. The load causing the formation of the first plastic hinge is taken as the design load using the same approach as the previous notional force method. The plastic hinge and plastic element methods are also available in software NIDA[7] used in the present analysis but the difference between these methods and the first plastic hinge design method is minimal because of lack of strength reserve and therefore they are not used here.

SECTION CAPACITY CHECK FOR DESIGN LOAD

In second-order analysis, the section capacity check allowing for the P-Δ and the P-δ effects is carried out so that individual member sectional and buckling capacity can be assessed throughout the analysis procedure. As the second-order effects related to member slenderness have been simulated, it is not necessary to assume the effective length for a member in the buckling capacity check. The equation for this section capacity check can be carried out as follows.

$$\frac{F_c}{A_g P_y} + \frac{M_R}{M_{cx}} = \chi \leq 1 \tag{2}$$

in which MR is the resultant moment from bending moments about two perpendicular axes as Fc(Δx+δx) and Fc(Δy+δy) which must include the second-order moments. Mcx is the moment capacity of the section and it is taken as the product of design yield strength and plastic modulus.

The complete theory described in this paper has been coded in the computer software NIDA (2001)[7]. The checking of member strength is carried out automatically in the software for all members in the frame using Equation (2).

In this paper, the second-order elastic analysis (nonlinear geometry and linear material) is performed on the modular metal scaffolding specimens. The proposed analysis method is based on the one previously adopted by Peng and Chan[8]. The formulation of the element for the metal scaffolding system is based on the solution the differential equilibrium equation for the relationship between force, nodal moments and rotations with due consideration of the P-Δ and P-δ effects. The presently used method employs an explicit expression for a beam-column element under the Timoshenko's beam-column theory. Apart from the use of an accurate beam-column element, the numerical method used in a second-order analysis is essential in order to prevent divergence in iteration. This paper adopts the minimum residual displacement method[9] with a variable load increment size for tracing of equilibrium path of a skeletal structure in the large displacement range.

NUMERICAL ANALYSIS PROCEDURES

The load versus deflection and the load versus stress path for the structure analysed can be plotted by the derived element using a numerical scheme suggested. The section capacity check is then used to assess the buckling strength of individual members via equation 2. In the computer program, this is indicated by various colors for different stages of consumed sectional capacity, χ in Equation 2. When χ for any of the vertical posts exceeds 1.0, the complete system is considered failed. However, plastic hinges are allowed and thus the plastic modulus is used for beams of compact section that is applicable to circular sections of yield stress 350 MPa and with diameter-to-thickness ratio less than 45. This load will be of great interest to the engineer and it can be seen that, depending on the yield stress of material used, the load calculated by the present non-linear theory is more accurate than the predicted failure load by the conventional method.

Example

Weesner and Jones[5] tested a series of 3-storey steel scaffolding units. The dimensions of the four types of units can be found in Reference 5. The material yield stress is assumed nominally as 350 MPa and the Young's modulus of elasticity is 200 kN/mm^2. The tested systems are assumed vertical with four legs pinned to a rigid floor. In the tests, the specimen was mounted onto a loading frame and an I-beam installed on the top of the steel scaffold. The load is applied to the channel at top of the scaffolding unit and the loaded mode is assumed restrained against all horizontal directions, but free to move vertically in order to allow the direct transfer of vertical load to the scaffolding. The I-beam and the loading frame are not assumed to take any moment during loading. Thus the top connections to the loading I-beam are assumed as hinge-joined. It is recognized that the base-plate at the bottom of the scaffold provides partial stiffness against rotation during loading. However, for conservative practical design and lack of measured connection stiffness, the supports are assumed pinned. The sleeve connections between the modular units were modeled as rigid joints since the over-lapping sleeved length is long of length greater than 100 mm, thus providing a moment resisting connection.

If there is no top lateral restraint, the notional force should obviously be applied to the load location and in a horizontal direction. However, as the loaded node is horizontally restrained, this application of notional force becomes meaningless as all forces will then go to the support. However, considering the expected imperfect curvature of the complete system, these notional forces are applied at the junction between each scaffolding unit. Physically this simulation can be illustrated as replacing the imperfect system by a truly vertical system with notional forces, namely as the NFM here. An alternative and more direct method is to use buckling mode as imperfection mode designated as the BIM method here.

The computed design capacities of the four types of 3-storey scaffolding systems are tabulated in Table 1, together with the tested results. It can be seen that the design load capacity ranges from 75% to 101 % of the averaged tested failure load. Contrast to the eigenvalue buckling loads, the present loads is always below the tested failure load, indicating the suitability of the proposed methods as a reliable tool giving a lower bound solution for practical design. The predicted failure loads by the notional force and by the buckling mode as imperfection mode are tabulated below against the tested failure loads.

Frame Type Number	Experimental Results Load (kN)	Computed Design Load (kN) (Theory/Test)		
		I	II	III
A1	195.1	193 (0.97)	198 (1.00)	165 (0.83)
A2	201.9			
B1	184.4	171 (0.91)	188 (1.00)	141 (0.75)
B2	190.0			
C1	502.4	422 (0.82)	512 (1.00)	445 (0.87)
C2	525.3			
D	180.9	158 (0.87)	182 (1.01)	135 (0.75)

NOTE : Types A1 to D are types of scaffolding systems tested in Reference 5.
Averaged test load is used for comparison with the theory.

I Theory, using 1% notional force

II Theory, using 0.5% notional force

III Theory, using buckling mode as initial imperfection mode of maximum imperfection equal to Height/500

Table 1 – Tested results from experiments and computer analysis

From the tabulated results, it can be seen that the theoretical failure loads by both the methods of NFM and BIM are close to the tested results. Both the NFM using 1% notional force and BIM method gives loads lower than the tested loads in all cases, but the NFM with use of 0.5% notional force is nearly the same as the averaged tested loads with some individual predicted loads non-conservatively above the tested loads. However, it must be emphasized that the tested loads here are expected to be higher than the actual collapse loads when used on site, because of the better fabricated condition in laboratory than on site and therefore it may be unfair to concluded that NIF with 0.5% notional force is better than the BIM method which always predicts conservative failure loads.

The computed model for the collapse scaffold type “C” in Reference 4 is shown in Figure 1 and the typical load vs. deflection plot is shown in Figure 2 which shows the eastic-plastic collapse plot the scaffold type “B”.

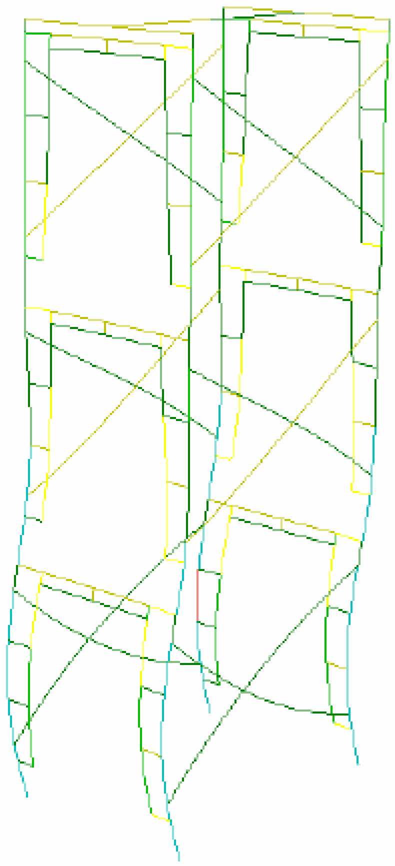

Figure 1   The simulated elastic-plastic buckling of a scaffold

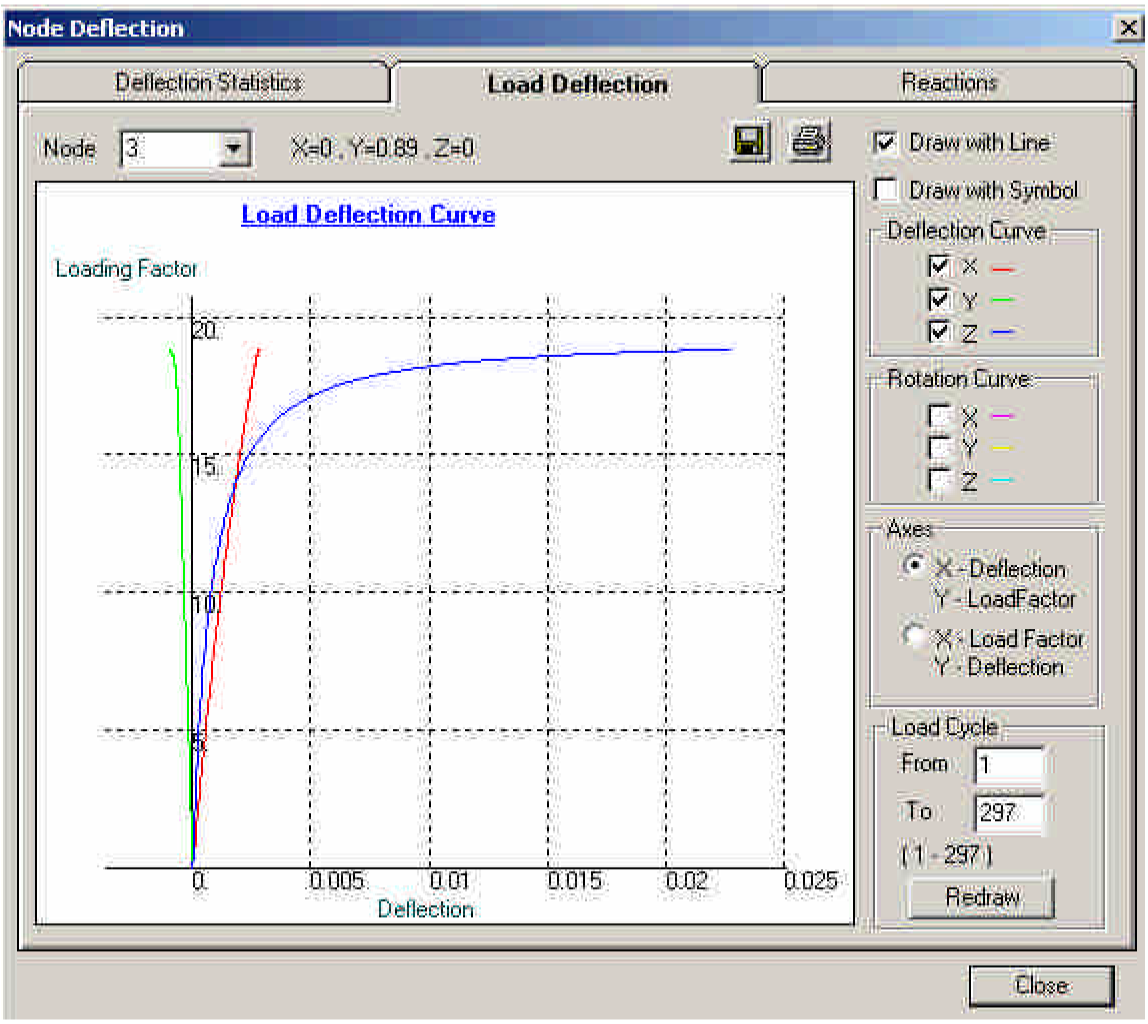

Figure 2 The elastic-plastic load deflection curve of a node in the scaffold

CONCLUSION

Two computer methods for the second-order analysis for scaffolding systems which are based on the 0.5% and 1% notional force (NFM) and using the buckling mode as the initial imperfection mode (BIM) have been coded in software NIDA[7] and proposed in this paper. The accuracy of these two methods has been compared with seven tested specimens.

From the results of analysis using code recommended imperfection parameters of Height/500, the BIM gives convenient and conservative design loads for scaffolds which may be more suitable for use in construction site condition where quality of scaffolds is not as good as the ones in laboratory. The NFM gives very close results to the tested loads when using coded values of 0.5% and 1% vertical load as notional force. Both the methods do not require assumption of effective length and they are believed to be more scientific, rational, reliable and accurate than the effective length method.

ACKNOWLEDGEMENT

The authors acknowledge the financial support of the Research Grant Council of the Hong Kong Special Administrative Region Government under the projects “Advanced analysis of steel frames and trusses of non-compact sections using the deteriorating plastic hinge method (PolyU 5117/06E)” and “Second-Order and Advanced Analysis of Wall-Framed Steel Structures (PolyU 5115/05E)”.

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