Steel and Composite Structures, Vol. 2, No. 1 (2002) 21-34
DOI: http://dx.doi.org/10.12989/scs.2002.2.1.021

Modeling of local buckling in tubular steel frames by using plastic hinges with damage

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Abstract. A model of the process of local buckling in tubular steel structural elements is presented. It is assumed that this degrading phenomenon can be lumped at plastic hinges. The model is therefore based on the concept of plastic hinge combined with the methods of continuum damage mechanics. The state of this new kind of inelastic hinge is characterized by two internal variables: the plastic rotation and the damage. The model is valid if only one local buckling appears in the plastic hinge region; for instance, in the case of framed structures subjected to monotonic loadings. Based on this damage model, a new finite element that can describe the development of local buckling is proposed. The element is the assemblage of an elastic beam-column and two inelastic hinges at its ends. The stiffness matrix, that depends on the level of damage, the yielding function and the damage evolution law of the two hinges define the new finite element. In order to verify model and finite element, several small-scale frames were tested in laboratory under monotonic loading. A lateral load at the top of the frame was applied in a stroke-controlled mode until local buckling appears and develops in several locations of the frame and its ultimate capacity was reached. These tests were simulated with the new finite element and comparison between model and test is presented and discussed.

Key words: local buckling; damage mechanics; structural failure; structural steel; inelastic behavior.

1. Introduction

Local buckling is one of the main modes of collapse of slender metallic structures. This is why this is an important subject in fields such as earthquake and offshore engineering. Extensive experimental analysis on the subject has been carried out. Karamanos & Tassoulas (1996), for instance, report the following references on the subject: (Reddy 1979), (“Collapse” 1985), (“Effects 1988”), (“Hydrostatic” 1989) and (Kyriakides & Ju 1992). These works describe the behavior of steel or aluminum tubular members of circular cross section. Additionally, Chan *et al*. (1991) mentioned a report by Key & Hancock (1985) that studies beam-columns of square hollow sections. More recently, the experimental research carried out for the preparation of the Eurocode 9 (1998) on aluminum structures and the tests carried out at the Salerno University (Faella *et al*. 2000) can be mentioned.

In order to predict the behavior of tubular elements with local buckling, three different approaches can be found in the literature: semi-empirical methods based experimental analysis (see for instance Mazzolani and Piluso 1992, Faella *et al*. 2000), finite elements analysis using nonlinear 3D shell elements

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(For instance Ju & Kyriakides 1992, Chan *et al.* 1991 and Karamanos &Tassoulas 1996) and "strength of material approaches", or more generally speaking, 2D analysis (Sohal & Chen 1987 and Karamanos & Tassoulas 1996). A special mention must be made to this last approach that represents a simple and effective procedure for the analysis of a complex phenomenon. All these approaches may be considered as complementary and correspond to different aspects of an engineering application.

However, none of the aforementioned references considered the possibility of lumping local buckling in plastic hinges. This paper and others by researchers of the University of Los Andes (Inglessis *et al.* 1999, 2000) propose a model of the behavior of steel frame members with local buckling that combines concepts of continuum damage mechanics and the notion of plastic hinge. This approach has also been used for modeling the behavior of reinforced concrete structures where the main damage mechanism is due to the cracking of the concrete (see for instance Perdomo *et al.* 1999).

The range of applications of the model proposed in this paper corresponds to the case of large and complex structures where the appearance of local buckling in one location may have a significant influence on the behavior of the remaining members of the structure and on the appearance and development of local buckling in other locations. It is clear that more detailed analysis, like those based on shell theories are inadequate for this goal. It is also important to underline that this influence may be not negligible, since local buckling changes significantly the stiffness and strength of the members affected, and therefore it may force important stress redistributions in the entire structure.

This paper is organized as follows: in section 2 some experimental results that support the model are presented. The model itself is described in sections 3, 4 and 5. Section 3 presents the stiffness matrix of a frame member with local buckling lumped in inelastic hinges. Section 4 introduces a local buckling evolution law as a function of the plastic rotation of the hinge. Section 5 describes how the yield function of the hinge is modified by the presence of local buckling. Section 6 introduces a finite element based on this model that can be included in the library of standard structural analysis programs. Finally, section 7 presents the structural analysis of a frame with local buckling, and compares these data with experimental results.

2. Experimental results obtained in steel tubular members subjected to bending

Some specimens representing steel frame members were subjected to bending. The elements were supported by an enlarged end block simulating a rigid column and loaded at the tip as cantilever beams (see Fig. 1).

The specimens were subjected to series of loadings, in displacement-controlled mode, and unloadings, in force-controlled mode, as shown in Fig 1. Fig 2 shows the results obtained in a test carried out with a

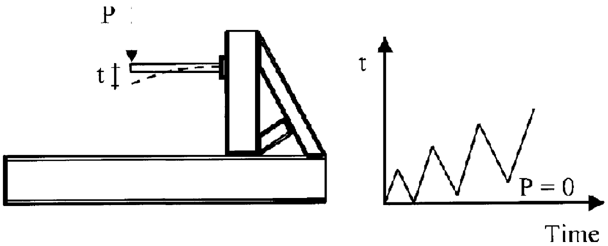

Fig. 1 Test on steel members: specimen and loading

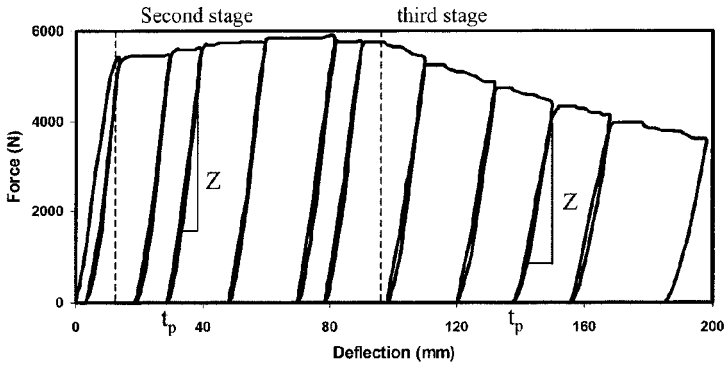

Fig. 2 Displacement at the tip vs. force in tubular steel member

tube of circular cross section (external diameter 60.3 mm, 2.6 mm of thickness, length 489 mm)

It can be noticed that the behavior of the element can be divided in three stages. First a zone of quasilinear response, followed by a phase of plastic hardening that seems to stabilize and finally a softening stage. The latter presents a behavior that could be represented by a straight line of negative slope. The last assumption corresponds to an idealization of the real behavior that is in fact much more complex. In this sense, the model that will be proposed in the following sections is the equivalent of the perfect plasticity model or the linear kinematic hardening model., i.e., a simplification that allows for a qualitative representation of the real behavior although not always quantitatively accurate. Other mathematical approximations could be used. For instance, in (Febres 2002) it has been explored the advantages of the use of an exponential function instead of a linear function. However, it is important to note that the influence of the specific softening representation is less critical as the frame is more complex.

The softening observed in the graph of Fig. 2 is due to the appearance of local buckling in the plastic hinge region (see Fig. 3). Local buckling develops decreasing the strength of the tube up to its

Fig. 3 Local buckling in the frame member

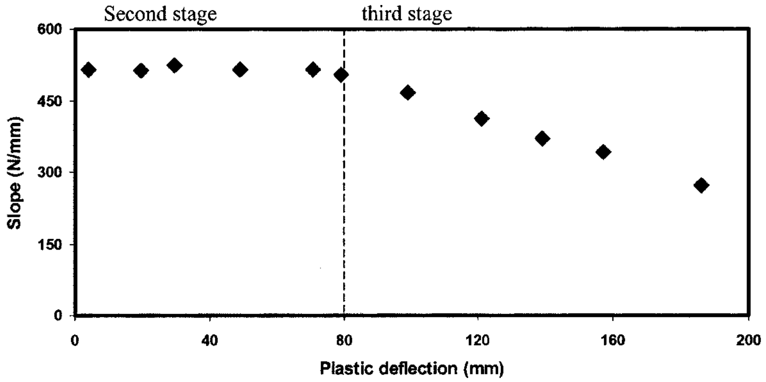

Fig. 4 Stiffness vs. permanent deflection

total collapse.

Another effect associated to the apparition of local buckling is a progressive loss of stiffness, as shown in Fig. 4, that can be observed during the softening phase. In Fig. 4, the slope Z of the elastic unloading is plotted against the plastic or permanent deflection t^p that corresponds to that unloading. The meaning of the values of Z and t^p are indicated in the graph of Fig. 2.

The first part of the behavior can be modeled using the theory of elasticity; the second stage may be represented using the conventional concept of plastic hinge. This is obviously not the case of the third phase. The main goal of this paper is to propose an extension of the concept of plastic hinge that would allow the description of the three stages and the main phenomena observed during the tests, i.e., plastic deformations and the loss of stiffness and strength related to local buckling.

3. Modeling of local buckling in steel frame members with inelastic hinges

A member of the structure is isolated as indicated in Fig. 5. Matrices $\boldsymbol{\Phi}^t = (\phi_i, \phi_j, \delta)$ and $\boldsymbol{M}^t = (m_i, m_j, n)$ define generalized deformations and stresses of the member. The superscript t means “transposed” and the interpretation of the elements of the matrices is indicated in Fig. 5.

Matrix $\boldsymbol{\Phi}$ is the equivalent of the strain tensor in continuum mechanics in the sense that it represents changes of shape of the member. The matrix $\boldsymbol{M}$ is then the equivalent of the Cauchy stress tensor. The relationship between the history of generalized deformations and the generalized stress matrix is called in this paper “generalized constitutive model” or constitutive model for a frame member. For instance, in an elastic element, the constitutive law is given by (1):

$$\boldsymbol{M} = \boldsymbol{S}^0\,\boldsymbol{\Phi} \quad \text{or} \quad \boldsymbol{\Phi} = \boldsymbol{F}^0\,\boldsymbol{M} \tag{1}$$

where $\boldsymbol{S}^0$ is the elastic stiffness matrix and $\boldsymbol{F}^0$ the elastic flexibility of the frame member.

In order to include local buckling and plastic effects, the member is assumed to be the assemblage of an elastic beam-column and two inelastic hinges as indicated in Fig. 6.

The state law of the frame member is obtained by assuming an additive decomposition of the generalized deformations in beam-column deformations $\boldsymbol{\Phi}^b$ and hinge deformations $\boldsymbol{\Phi}^h$:

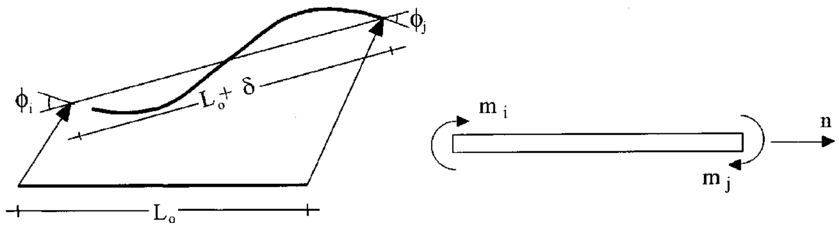

Fig. 5 Generalized stresses and deformations

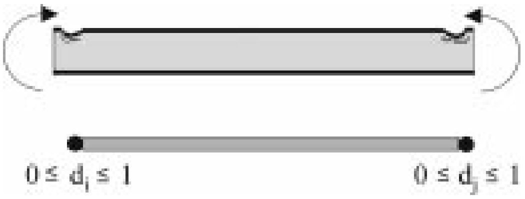

Fig. 6 Lumped inelasticity model

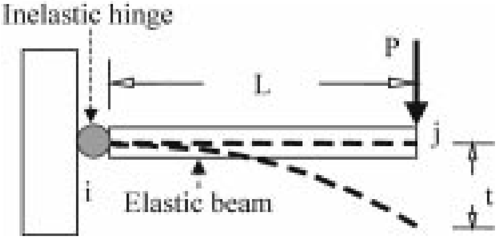

Fig. 7 Lumped inelasticity model of the test

$$\boldsymbol{\Phi} = \boldsymbol{\Phi}^{b} + \boldsymbol{\Phi}^{h} = \boldsymbol{F}^{0}\boldsymbol{M} + \boldsymbol{\Phi}^{h} \tag{2}$$

In these inelastic hinges, plasticity as well as local buckling is lumped. Therefore, it is assumed that hinge deformations result from plastic rotations $\boldsymbol{\Phi}_p^t = (\phi_i^p, \phi_j^p, 0)$, as defined in conventional plastic theories for frames, and an additional term specifically related to local buckling $\boldsymbol{\Phi}^d$:

$$\boldsymbol{\Phi} = \boldsymbol{F}^{0}\boldsymbol{M} + \boldsymbol{\Phi}^{p} + \boldsymbol{\Phi}^{d} \tag{3}$$

It can be noticed that permanent elongations of the chord are neglected. This is a simplifying assumption and not a requirement of the model. The local buckling rotations depend on the “degree of local buckling”. In order to characterize the state of local buckling, a new set of internal variables is introduced: $D^t = (d_i, d_j)$, where parameters d_i and d_j represent the level of damage of hinges i and j. These damage parameters can take values between zero and one. Zero represents a conventional plastic hinge without local buckling. In order to represent the loss of stiffness observed in Fig. 4 due to local buckling, the following expression of the local buckling deformations $\boldsymbol{\Phi}^d$ is introduced:

$$\boldsymbol{\Phi}^{d} = \boldsymbol{C}(\boldsymbol{D})\boldsymbol{M} \tag{4}$$

where $\boldsymbol{C}(\boldsymbol{D})$ is a diagonal matrix whose non-nil terms are: $C_{11} = d_i F_{11}^0/(1-d_i)$ and $C_{22} = d_j F_{22}^0/(1-d_j)$. These expressions can be justified on the basis of concepts of continuum damage mechanics (Flórez-López 1998). The flexibility matrix of a frame member with local buckling can be obtained by substitution of Eqs. (4) in (3):

$$\boldsymbol{\Phi} - \boldsymbol{\Phi}^{p} = \boldsymbol{F}(\boldsymbol{D})\boldsymbol{M} \text{ or } \boldsymbol{M} = \boldsymbol{S}(\boldsymbol{D})(\boldsymbol{\Phi} - \boldsymbol{\Phi}^{p}) \tag{5}$$

where

$$\boldsymbol{F}(\boldsymbol{D}) = \boldsymbol{F}^0 + \boldsymbol{C}(\boldsymbol{D}) \text{ and } \boldsymbol{S}(\boldsymbol{D}) = \boldsymbol{F}(\boldsymbol{D})^{-1} \tag{6}$$

4. Local buckling evolution law

The test discussed in section 2 of the paper is again considered. This test can be modeled using plastic hinges with damage as represented in Fig. 7.

The following boundary conditions applies to the problem represented in Fig. 7:

$$m_i = P.L; \quad m_j = 0; \quad \phi_i = \frac{t}{L}; \quad \phi_i^p = \frac{t^p}{L}; \quad d_i = d; \quad d_j = 0 \tag{7}$$

Where t^p is again the permanent deflection measured at the end of each elastic unloading. The state law (5) and the precedent boundary conditions determine the relationship between force and deflection during the test:

$$P = Z(d)(t - t^p) \quad \text{where } Z(d) = (1 - d)Z^0, \qquad Z^0 = \frac{3EI}{L^3} \tag{8}$$

The term Z is again the slope of the elastic unloading during the test. Z^0 is the initial slope before local buckling. The second of Eqs. (8) suggest an experimental procedure for the determination of the local buckling state in the hinge from the graph of Fig. 4 (Inglessis *et al.* 1999):

$$d = 1 - \frac{Z(d)}{Z^0} \tag{9}$$

It is now possible to obtain the plot of damage in the hinge against plastic rotation as shown in Fig. 8.

In order to describe the behavior observed in Fig. 8, the following "local buckling function" is introduced for each inelastic hinge of the structure:

$$g_i = \left|\phi_i^p\right| - R(d_i) \le 0 \tag{10}$$

The local buckling evolution law can now be written as:

$$\begin{cases} \dot{d}_i = 0 \text{ if } & g_i < 0 \quad \dot{g}_i < 0 \\ \dot{d}_i > 0 \text{ if } & g_i = 0 \quad \dot{g}_i = 0 \end{cases} \tag{11}$$

In other words, local buckling evolution is only possible if the plastic rotation of the hinge reaches some critical value or "local buckling resistance" R. This notation is suggested by an analogy with Fracture Mechanics. In the monotonic case, the "local buckling driving variable" is the plastic rotation of the hinge.

It is assumed that the local buckling resistance R is a function of the damage, i.e., the local buckling state, of the hinge. Function R can be identified from the graph of Fig. 8. For instance, a straight line of slope $\boldsymbol{b}$ and intersection with the plastic rotation axe p_{cr} can represent the tendency observed in the figure. In this case, the corresponding local buckling resistance has the following expression:

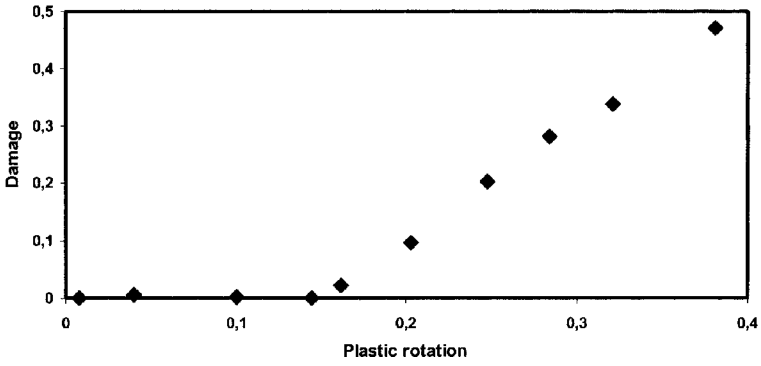

Fig. 8 Damage against plastic rotation in the hinge

$$R(d_i) = p_{cr} + \frac{d_i}{b} \tag{12}$$

The evolution law defined by Eqs. (11)-(12) is only valid in the case of monotonic loadings. In a more general case, more than one local buckling may appear in the plastic hinge region and the use of only one damage parameter and the plastic rotation as local buckling driving variable as proposed in this paper would not be sufficient.

In the examples presented in the following sections, the parameters p_{cr} and b are obtained from experimental results. This procedure is not the most convenient for real engineering applications. A systematic and rational procedure for the determination of local buckling parameters must be established if the model is to be used in the engineering practice. This is a problem that remains open, that requires additional experimental and theoretical work, and that will not be addressed in this paper. However some general ideas are discussed in the conclusions of the paper.

5. Plastic rotation evolution law

The yield function of a plastic hinge without local buckling can be written in the following way:

$$f_i = |m_i - x_i| - m_e \tag{13}$$

where x_i is a kinematic hardening term and m_e is the last elastic moment of the cross section member. As aforementioned, plastic hardening in the member can be developed to an important degree, close to saturation, before local buckling appears. Therefore, some kind of non-linear kinematic law is needed to describe the behavior of the hinge before local buckling appears. For instance, in the numerical simulations presented in the nest section, the one proposed by Chaboche (1978) was used:

$$\dot{x}_i = \beta(m_y - m_e)\dot{\phi}_j^p - \beta x_i\left|\dot{\phi}_i^p\right| \qquad x_i = 0 \text{ for } \phi_i^p = 0 \tag{14}$$

Where m_y is the ultimate moment of the cross section and β is a member-dependent parameter. The meaning of the terms m_e and m_y is illustrated with the help of the conventional concepts of the strength of materials in Fig. 9.

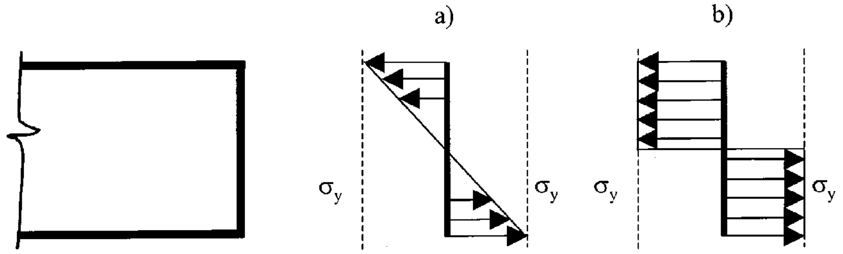

Fig. 9 (a) Stress distribution in the cross section when the moment reaches the value m_e (σ_y yielding stress). (b) Stress distribution for the ultimate moment m_y

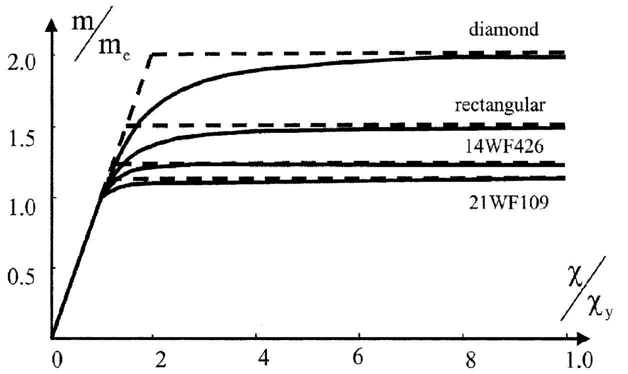

Fig. 10 Exact moment-curvature relationship for different cross sections after (Chen and Sohal 1995)

The velocity of the transition from the last elastic moment m_e to the ultimate moment m_y depends on the shape of the member cross section as shown in Fig. 10.

With the law (14), the evolution from m_e to m_y in the plastic hinge is represented by an exponential law and the velocity of the transition is given by the parameter β as shown in Fig. 11. Thus, the constant β must be computed to fit the specific hardening velocity for each particular cross section.

Plastic rotation evolution law can now be described by an equation similar to (11):

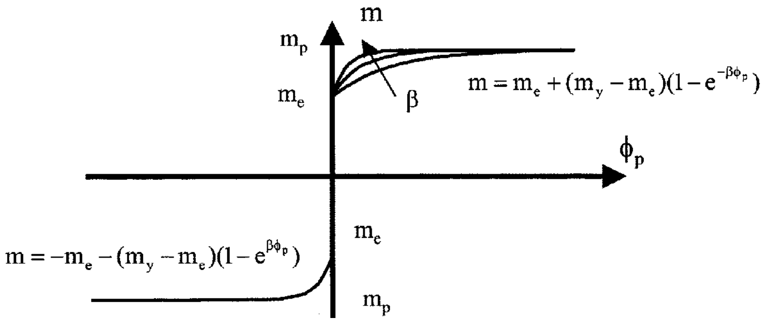

Fig. 11 Moment rotation relationship

$$\begin{cases} \dot{\phi}_i^p = 0 \ \text{ if } \ f_i < 0 \quad \dot{f}_i < 0 \\ \dot{\phi}_i^p \neq 0 \ \text{ if } \ f_i = 0 \quad \dot{f}_i = 0 \end{cases} \tag{15}$$

As aforementioned, when local buckling appears in the plastic hinge region, a sudden loss of strength is observed in the member. Therefore, the maximum moment of the cross section is no longer m_y but a lower value that depends on the local buckling state. In order to model this phenomenon, the concept of "equivalent moment" is introduced. This concept is similar to that of equivalent stress used in poro-elasticity and damage mechanics. The equivalent moment $\overline{m}_i$ on a plastic hinge with damage is defined as:

$$\overline{m}_i = \frac{m_i}{1 - d_i} \tag{16}$$

Then, the yield function of a plastic hinge with local buckling can be obtained by introducing the effective moment instead of the conventional moment in expression (13).

$$f_i = \left| \frac{m_i}{1 - d_i} - x_i \right| - m_e \tag{17}$$

In continuum damage mechanics, this procedure is called "hypothesis of equivalence in deformations". The plastic rotation evolution law is now defined by this yield function with the unmodified expressions (14-15).

In some cases, local buckling appears in the plastic hinge region before the plastic hardening reaches saturation. The model can reproduce this situation by an adequate choice of the parameters β in Eq. (14) and p_{cr} in expression (12).

6. Formulation of a finite element with local buckling

Fig. 12 shows the degrees of freedom $\boldsymbol{q}$ and the nodal forces $\boldsymbol{Q}$ in a frame element. The relationship between generalized deformations $\boldsymbol{\Phi}$ and the elements degrees of freedom can be obtained by simple

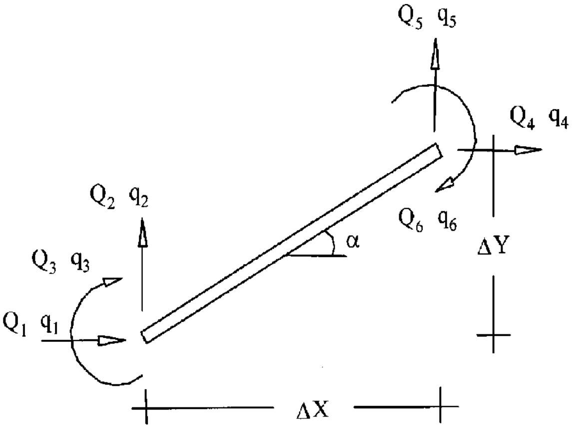

Fig. 12 Degrees of freedom and nodal forces in a frame element

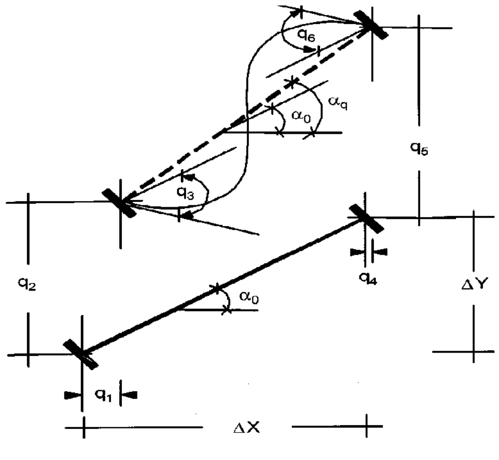

Fig. 13 Physical configuration of the member

geometrical considerations. In the general case, including geometrically nonlinear effects (see Fig. 13), this relationship is:

$$\phi_i = q_3 - (\alpha_0 - \alpha(\boldsymbol{q})); \ \ \phi_j = q_6 - (\alpha_0 - \alpha(\boldsymbol{q})) \quad \delta = L(\boldsymbol{q}) - L_0 \tag{18}$$

Where

$$\alpha = \tan^{-1}((\Delta Y_0 + q_5 - q_2)/(\Delta X_0 + q_4 - q_1));$$
$$L = \sqrt{(\Delta Y_0 + q_5 - q_2)^2 + (\Delta X_0 + q_4 - q_1)^2} \tag{19}$$

In Eqs. (18)-(19), the terms with the index 0 represent quantities in the reference configuration.

The relationship between generalized stresses $\boldsymbol{M}$ and nodal forces $\boldsymbol{Q}$ can be obtained by considering the equilibrium of the member in the deformed configuration:

$$Q_1 = (m_i + m_j)(\sin\alpha/L) - n\cos\alpha; \ Q_2 = -(m_i + m_j)(\cos\alpha/L) - n\sin\alpha; \ \ Q_3 = m_i$$
$$Q_4 = -(m_i + m_j)(\sin\alpha/L) + n\cos\alpha; \ Q_5 = (m_i + m_j)(\cos\alpha/L) + n\sin\alpha; \ \ Q_6 = m_j \tag{20}$$

Eqs. (18)-(20) and the constitutive model defined by expressions (5,6, 10-12, 14-17) constitute a set of equations that define a conventional finite element. This element has been implemented in a commercial F.E. program that allows nonlinear analysis (Inglessis 2000).

7. Verification of the model

In order to verify the model, another series of tests was carried out in laboratory. (Medina 1998) This time, the specimens consisted of a steel frame of two levels and two spans (see Fig. 14). The elements had rectangular hollow cross section and were welded at the joints. Nominal characteristics and dimensions of the frame and its members are shown in Tables 1 and Fig. 14 respectively. The frame was

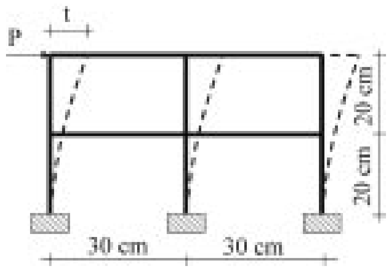

Fig. 14 Tested frame

Table 1 Nominal characteristics of the tested frame

Frames	*H* (mm)	*B* (mm)	*e* (mm)	sect. (mm^2)	*lx* (mm^4)	*ly* (mm^4)
1	41.0	24.0	2.5	300	63970	26655

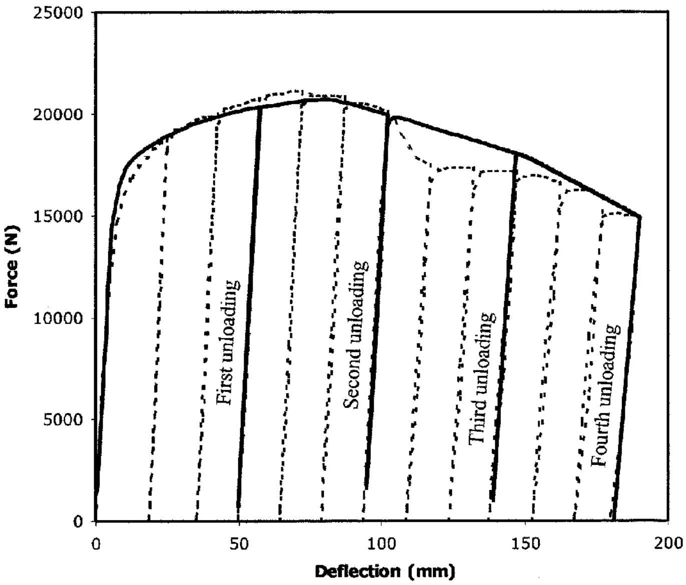

Fig. 15 Test and simulation on a planar frame

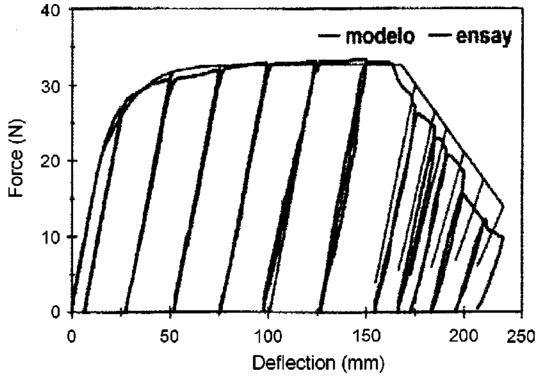

Fig. 16 Identification test and numerical simulation

subjected to the same class of loading that was described in section 2 of the paper and that is represented in Fig. 1. The experimental results of one of the monotonic tests are shown in Fig. 15.

Single elements of the frame were tested in order to identify the parameters of the model. Fig. 16 shows the results of the identification in one of the member frame tests. The parameters are presented in Table 2.

Fig. 15 shows the comparison between model and experimental results in the case of a frame. For the sake of clarity, only four of the elastic unloadings are represented in the simulation. Fig. 17 indicates the state of damage at the end of the four unloadings shown in Fig. 15. The numbers beside the hinges represent the damage values. The first distribution presents six plastic hinges with no damage, i.e. without local buckling. It can be noticed that this state corresponds to the plastic hardening phase of the test.

The maximum resistance of the frame is reached between the first and second elastic unloading of the

Table 2 Parameters of the model

Test	m_e (N-mm)	b	p_{cr}	m_y (N-mm)	β
beam 1	622722,28	1,30	0,210	760015,38	18,00
beam 2	715885,45	1,60	0,170	941438,45	21,00
column 1	583495,68	1,30	0,200	706078,80	18,50
column 2	764918,70	1,60	0,190	961051,70	20,00
beam	669303,86	1,45	0,190	850726,89	19,50
columns	674207,19	1,45	0,195	833565,25	19,25

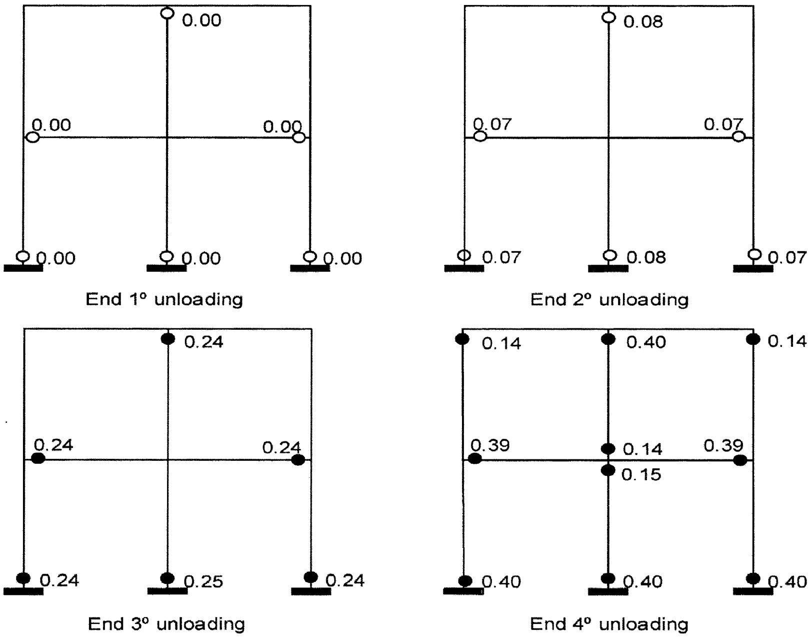

Fig. 17 Damage distribution in the frame

simulation where a sudden change of the tangent slope can be appreciated. In the simulation, this modification of the tangent stiffness is due to the appearance of local buckling in the same six plastic hinges. Four new plastic hinges appear in the frame without local buckling while damage continues to evolve in the first six hinges. After the third unloading, local buckling appears also in the four remaining hinges. In the simulation, a slight additional modification in the tangent stiffness can be appreciated when that happens. The test was stopped after the fourth unloading and the computed final state of damage is shown in the last of Fig. 17.

8. Conclusions

The model presented in this paper constitutes an alternative approach for the analysis of structures with local buckling. The authors believe that this alternative may be valuable in many engineering problems. It must be underlined that the structure analyzed in the precedent section corresponds to a very complex and expensive problem if, for instance, shell theory is to be used. On the other hand, only ten finite elements were needed with the use of inelastic hinges that lumps local buckling as well as plasticity. The difference of costs between both kinds of analysis may be huge: expensive commercial finite element programs vs. cheap direct stiffness programs; tens of thousands of elements or even more vs. dozens or hundreds of elements at the most; parallel computers vs. PCs and so on.

When shell theory is used, only one joint of the structure or even only one element of the structure is, usually, analyzed. Of course, this analysis is important and meaningful but one important phenomenon has to be ignored: the coupling between local buckling in different locations of the structure. This phenomenon cannot be neglected since the experimental results show important variations on the stiffness and the strength of the individual member. The stress redistribution that results from the stiffness modification changes the rate of plastic flow and local buckling in the entire structure. Probably, in strategic structures such as those of the offshore engineering both analysis, shell and lumped damage, are needed.

The use of the lumped inelasticity model for local buckling description implies that the user has some knowledge of the location where this phenomena can happen in the structure. The discretization of the frame in elements must be carried out taking into account this fact. This is also the case of the conventional lumped plasticity model without local buckling that has been extensively used in the practice. In most cases this discretization does not present difficulties for the user and refinement of the model is not customary. In this sense, the use of inelastic frame theories is more related with the direct stiffness method than with the finite element method.

One important subject is that of the parameter determination. In the example presented in this paper, these parameters were obtained via experimental identification of single elements of the frame. The authors believe that this is also possible in the case of real engineering applications, since metallic elements comes usually in a limited number of predetermined sizes. However this is probably not necessary. For instance, the strength of materials approach or even shell theory might be used for an estimation of the parameter values. In fact, this is the procedure employed for the determination of the constants in the case of conventional plastic hinges: the values of the last elastic moment m_e and the ultimate moment m_y are computed via strength of material analysis of individual elastic-plastic beams.

The model discussed in this paper do not takes into account the influence of the axial force on the development of local buckling. This influence can be determinant. A simplifying approach to this problem could be the determination of the model parameters as a function of the axial force. An even simpler approach can be the determination of these constants for a given average value of the axial force. This was the case of the frame analyzed in the last section of the paper where it was assumed a zero value of the axial force average of all the frame members. A more rigorous approach would need of the inclusion of permanent axial elongations and the use of some kind of plastic flow rule. In any case, this remains also an open problem.

It might be argued that the strength of material approach could also be used to analyze frames with local buckling and this is, of course, true. However, this is also the case of the plastic analysis of frames without local buckling and, nevertheless, practitioners usually prefer the use of plastic hinges. The reason is that even when compared with strength of material approaches, the use of plastic or inelastic hinges is considerably simpler and cheaper and provides for very good results. It is the hope of the authors that this may be also the case for local buckling.

Acknowledgements

The authors are grateful to CONICIT and CDCHT of the University of Los Andes for providing the financial support for the project.

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