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BUCKLING OF STEEL PORTAL FRAMES CONSIDERING MATERIAL NON-LINEARITY

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ABSTRACT:-In this research, an experimental study was carried out to investigate the failure modes of eight frames with IPE cross-sections. The study was conducted by subjecting these frames to two equal concentrated loads (applied directly on columns); however, the resulted data was obtained using load cells to record load increments.

Comparison between experimental and theoretical results was tabulated by analyzing the eight portal frames using the following methods:

1- Quasilinear analysis.
2- Non-linear material analysis.
3- Codes AISC ASD, and AISC LRFD.

For non-linear material analysis, direct tensile and stub column tests were performed to obtain the secant modulus as a function of plastic energy density. A mathematical formula was designed for this purpose using a special computer program. Moreover, a comparison of "AISC LRFD & AISC ASD" with the experimental results were implemented. Furthermore, the non-linear material model was also applied to these Codes. Linear analysis gives a reasonable approximation to deflections before yield occurs. Calculation of buckling loads for different frames using Euler formula, ASD, and finite element quasilinear analysis are significantly overestimate the experimental results. Whereas, incorporating the non-linear material model into the above mentioned methods of analysis brings the values very close to the experimental results.

Keywords: Buckling Load, Non-linear, Quasilinear, Strain Energy Density, Plastic Strain Energy Density, Initial Modulus, Secant Modulus, Propped Cantilever.

1. INTRODUCTION

This research was carried out to investigate the buckling modes of steel frames and compares the experimental with the theoretical methods using linear and nonlinear methods of analysis.

To achieve this goal an engineering analysis was carried out to assess the buckling load of eight steel portal frames using linear and non-linear methods based on non-linear material model.

The nonlinear material model adopted in this study was secant modulus model. Experimental tests for these steel portal frames was performed by subjecting them to two equal concentrated loads applied directly on columns.

Many researchers have investigated non-linear behavior of structures (material). A material non-linearity was adopted by Ziehkiewicz (1971), and by Marcal (1971). Particularly, for plasticity, the tangential stiffness structural matrix (relating increments of load to increments of displacement) incorporated the tangential modular matrix, relates the increments of stress to increments of strain [Pope (1956), Zienkiewicz et al (1965), Marcal and King (1967), Yamada et al (1968), and Zienkiewicz (1971)]. A special form using initial elastic stiffness matrix was referred to as the initial stress method. Abu-Farsakh (1989) developed a new model for non-linear material.

Two new non-linear material models were developed in this paper. For an orthotropic material, the non-linear secant mechanical properties are expressed as a function of plastic energy density of an equivalent linear elastic system. The original model represents a direct application to the mechanical property equation; whereas, the modified model and the iterative model are considered as an extrapolation of the original model. The new models can treat multiple mechanical property non-linearity and predict strain very reasonably at high stress levels.

Salmon and Johnson (1990) followed LRFD in tackling of non-linearity of steel structures. Crisfield (1991) used finite element technique and Newton-Raphson’s iteration to solve problems of non-linearity in trusses, beams, frames, and space structures. Toma and Chen (1994) compared the test results of full sized steel portal frame with the second order inelastic analysis. The results are intended to be used to verify future developments of practical second-order inelastic analysis methods. Chen and Shoal (1995) discussed in detail the second-order analysis. In this reference the second order-analysis formulation is derived for both material and geometric non-linearity. Moreover, many examples on steel portal frames are presented and compared with LRFD. Moy (1996) explained the plastic methods for steel and concrete structures; for this reference, examples of steel portal frames using non-linear analysis are discussed in details. Ziemian et al (1997) discussed the second-order inelastic analysis. It is concluded that the second-order inelastic analysis cannot be used to full advantage if designs are required to satisfy all of the current AISC-LRFD member strength equations. In any regard, the effectiveness of an inelastic design method can be realized only when: (1) used in conjunction with performance type strength criteria such as the prevention of system and member instability under factored loads, and (2) coupled with necessary serviceability requirements. Archer (2001) presented algorithm based on the requirements of the non-linear static procedure specified by the guidelines for seismic rehabilitations of buildings; however, it is formulated for two and three-dimensional analysis. This algorithm implements a displacement control technique, which used a single displacement component as an independent variable. This procedure involves the use of a single fictitious spring attached to control node. Abdel Jawad et al (2002) developed a computer program (FAIL FOR) to facilitate a comparison of four criteria of material non-linearity models.

STRESS-STRAIN MODEL

For an isotropic material with elasto-plastic behavior like steel, the secant modulus at the stress-strain curve is expressed as a function of plastic strain energy density at an equivalent linear elastic system as follows (Abu-Farsakh, 1989):

$$E_s = E_o\left[1 - B\left(\frac{U_{pi}}{U_{po}}\right)^C + D\left(\frac{U_{pi}}{U_{po}}\right)\right] \qquad (1)$$

Where:

E_s: Secant modulus.

E_o: Initial modulus.

B, C, and D: Constants determined by specifying sampling points by least square method.

U_{pi}: Plastic strain energy density.

U_{Po}: Quantity to non-dimensionlize the plastic strain energy density.

The plastic strain energy density is expressed as:

$$U_p = U_s - U_e \qquad (2)$$

Where:

U_s: The total strain energy density of the equivalent elastic system.

U_e: The elastic strain energy density due to unloading.

Therefore, for isotropic material:

$$U_s = \frac{1}{2}\sigma\varepsilon$$
$$U_e = \frac{1}{2}\sigma\varepsilon_e \qquad (3)$$

Where:

σ: Stress.

ε: Total strain.

ε_e: elastic strain due to unloading.

The idealized Stress-Strain curve is shown in Fig. 1.

2. EXPERIMENTAL WORK

Experimental tests were carried out inside a rig. This rig consists of two loading jacks (400 kN capacity each) with built-in load cell. These loading assemblies are connected by long bolts (20mm in diameter) and steel plates (25mm thick) to steel loading frame. The floor at which the sample is to be tested is a steel strong floor with threaded holes (of 30mm in diameter and 500mm distance c/c in both directions along the horizontal floor plane).

Several preparations were conducted before testing. The load assemblies were adjusted to the required positions, and the supporting system were fixed to the steel strong floor. To support any movement, twelve plates (25mm thick) were bolted with the supports. To control lateral displacement, two lateral supports were erected. The data cables of the load cells were connected to data acquisition system. To overcome stress concentrations, pair (25mm thick) plates were laid under each load cell. The whole assembly is shown in Fig. 2.

After preparation of the sample under the test rig, a vertical load of 2 kN is applied and then released prior to testing (to insure that no load loss was dissipated due to any movement of the supports at the column ends).

Two 400 kN loading assembly are loaded for each specimen continuously. The rate of loading was 0.6 to 0.8 kN/sec to about 80% of the expected failure load, and then the mode of loading was changed to displacement control with displacement rate of 0.01mm/sec in order to have reliable results to study the true behavior of the tested frames in all stages. This history of loading was required to estimate the failure mode of the frames, and thus to achieve a good understanding of the behavior of these frames.

Experimental tests were carried out on eight portal frames with IPE cross-sections as shown in Table 1. In this table, the girder cross section is constant for all frames (IPE160);

whereas, the columns have different cross sections. As shown in Fig. 3, the failure mode is buckling in minor axis bending; therefore, the analysis was propped cantilever.

DERIVATION OF NON-LINEAR MATERIAL MODEL

The material property constants B, C, and D shown in Equation (1) are determined by specifying three sampling data points on the stress-strain curve derived from the experimental data. The best fitting curve is obtained using the method of least square. The last data point on the uniaxial stress-strain curve was considered as one of the three sampling points. By substitution the values of the plastic strain energy density extracted from the experimental curve in Equation (1); therefore, three equations in B, C, and D are formed. The solution of the equations resulted the determination of the secant modulus property constants.

In this study, many trials were attempted to derive the secant modulus constants (B, C, and D) using the tensile and stub column tests (as mentioned before). For this purpose, a special program was prepared using Visual Basic programming technique. However, the calculated constants are listed in Table 2.

PROPPED CANTILEVER ANALYSIS

In general, the structural idealization (propped cantilever) for the test frame was accepted due to the occurred failure was buckling in minor axis bending. However, the analysis is carried out based on finite element second order analysis i.e., taking into account geometric nonlinearity for linear and non-linear material properties. Furthermore, the analysis was carried out using AISC ASD, and AISC LRFD for both linear and non-linear material properties. The results are shown in Fig's 4, 5, and 6. However, these figures indicate that the experimental results are close to AISC ASD after application of non-linear material model. In addition, the analysis using finite element second order based on non-linear material model results are very close to the experimental results. In general, Table 3 shows a comparison between the experimental results and the calculated values using the models mentioned above (i.e. second order, AISC ASD, AISC LRFD, and second-order non-linear material model).

3. RESULTS & DISCUSSION

Considering the experimental results, observations, and the comparison with the theoretical analysis of the steel portal frames; the following conclusions are drawn:

1- For material non-linearity, the mechanical properties are not constant because they are a function of stress level; however, it requires an iteration technique. In this research a mathematical was developed based on Newton-Raphson numerical method. The results are close to AISC ASD after application of non-linear material model.

2- Non-linear analysis shows a significant deflection before collapse starts. This is due the significant increase of the strain in the plastic zone. Codes limit the deflection of the structural members to insure function and safety.

3- Non-linear analysis ignores the spread of yield around the section, where plastic hinge forms.

4- Linear analysis gives a reasonable approximation to deflections before yield occurs. This due to the material before yielding obeys Hooks law.

5- Calculation of buckling loads for different frames using Euler formula, ASD, and finite element quasilinear analysis are significantly overestimate the experimental results. Whereas, incorporating the non-linear material model into the above mentioned methods of analysis brings the values very close to the experimental results. This reasonable, since the nonlinear material analysis reflects the real behavior of the structural parameters for different material state i.e., elastic and plastic.

6- Calculation of buckling load for different frames using LRFD overestimate the experimental results for small slenderness ratios. On the other hand, the theoretical buckling load underestimate the corresponding experimental buckling loads for higher slenderness ratios. However, applying non-linear material model to LRFD brings those values closer to the experimental results.

7- Quasilinear analysis with non-linear material model shows good estimate results and very close to the experimental results.

8- Yielding and collapse loads of hinged frames are smaller than the corresponding fixed frames having similar geometry and loading conditions.

9- Applying non-linear material model to warping formula (LRFD) gives reasonable results (when comparing the experimental load-deflection curve with the corresponding theoretical analyses).

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NOTATIONS

E_s: Secant modulus
E_o: Initial modulus
B, C, and D: Constants determined by specifying sampling points by least square method.

U_{pi}: Plastic strain energy density
U_{Po}: Quantity to non-dimensionlize the plastic strain energy density

U_s: Total strain energy density of the equivalent elastic system
U_e: Elastic strain energy density due to unloading

σ: Stress

ε *:* Total strain
ε_e: Elastic strain due to unloading
EXP: Experimental buckling load

P_q: Buckling load by quasilinear analysis

P_{qm}: Buckling load by quasilinear analysis with material nonlinearity

ASD_m: Buckling load by ASD with material nonlinearity

$LAFD_m$: Buckling load by LRFD with material nonlinearity

Table 1: Dimensions for the Frame Specimens

Frame ID	Beam Span (m)	Column Height (m)	Beam Section	Column Section	Support Type
Fr1	2.7	1.5	IPE160	IPE160	Hinged
Fr2	2.7	1.5	IPE160	IPE160	Fixed
Fr3	2	1.5	IPE160	IPE160	Hinged
Fr4	2	1.5	IPE160	IPE160	Fixed
Fr5	2	1.5	IPE160	IPE140	Fixed
Fr6	2	1.5	IPE160	IPE120	Fixed
Fr7	2	1.5	IPE160	IPE100	Fixed
Fr8	2	1.5	IPE160	IPE80	Fixed

Table 2: Coefficients for Secant Modulus Equation

E_o	B	C	D	U_{po}
2.00E+08	0.450327	0.08204	0.000002	1

Table 3: Comparison of Failure Loads for Propped Cantilever

Column section	P_e (kN)	P_{em} (kN)	P(ASD) (kN)	P_m(ASD) (kN)	P(LRFD) (kN)	P_m(LRFD) (kN)	P_q (kN)	P_{qm} (kN)	P_{ex} (kN)
IPE160	916	284	590	493	451	361	855	400	317
IPE140	666	216	492	371	361	321	619	376	283.56
IPE120	410	144	396	243	264	198	385	200	230
IPE100	200	79.4	311	213	162	96.4	186	168	200
IPE80	139	57.2	249	91.6	118	67.2	129	125	100

P_e: Euler buckling load.
P_{em}: Euler buckling load with material nonlinearity.
P (ASD): ASD buckling load.
P_m (ASD): ASD buckling load material nonlinearity
P (LRFD): LRFD buckling load.
P_m (LRFD): LRFD buckling load with material nonlinearity.
P_q: Buckling load using finite element second order analysis.
P_{qm}: Buckling load using finite element second order analysis with material nonlinearity.
P_{ex}: Experimental failure load.

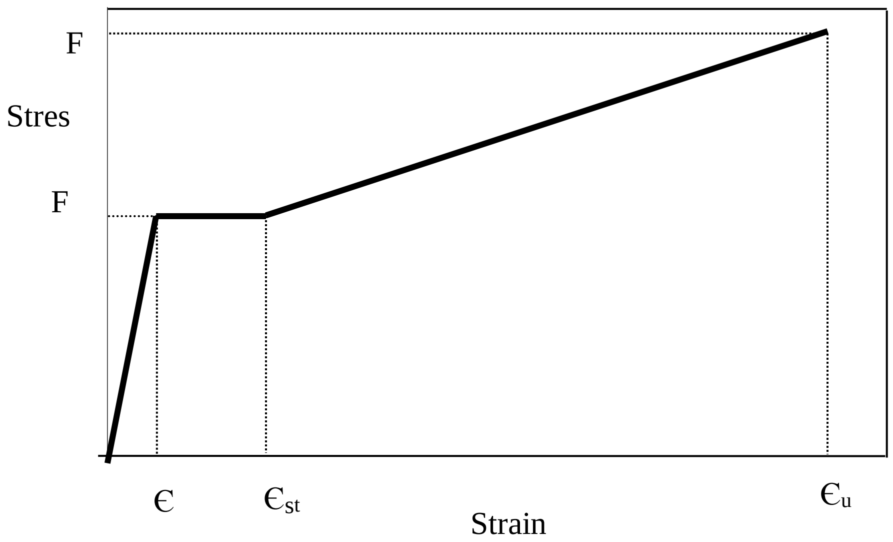

Fig. (1): Idealized Stress-Strain Curve derived from experimental tensile tests.

Fig. (2): Test Rig setup (steel portal frame IPE section subjected to two concentrated loads)

Fig. (3): Column Failure due to Buckling in Minor Axis mode Bending

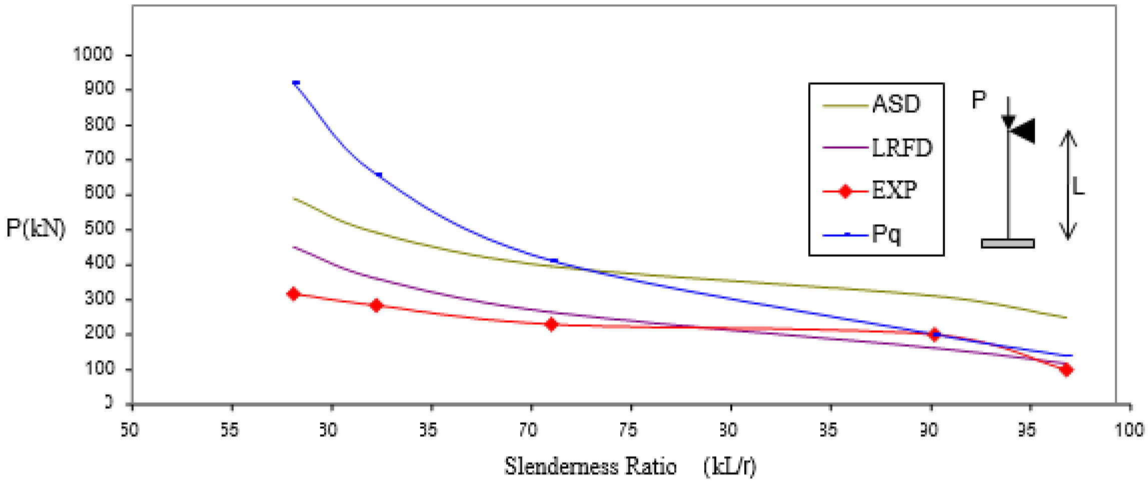

Fig. (4): Comparison between Theoretical and Experimental Loads for Propped Cantilever.

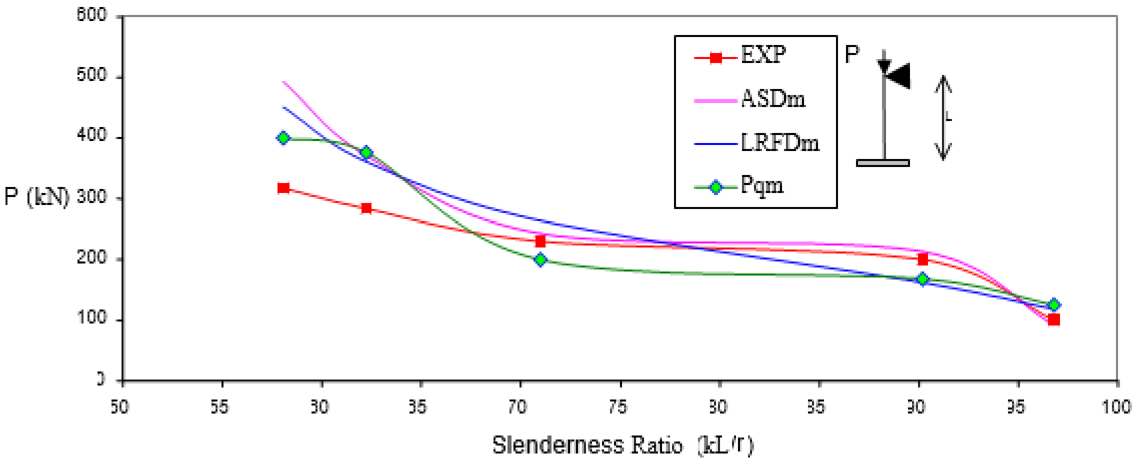

Fig. (5): Comparison between Theoretical and Experimental Loads (Material Nonlinearity)

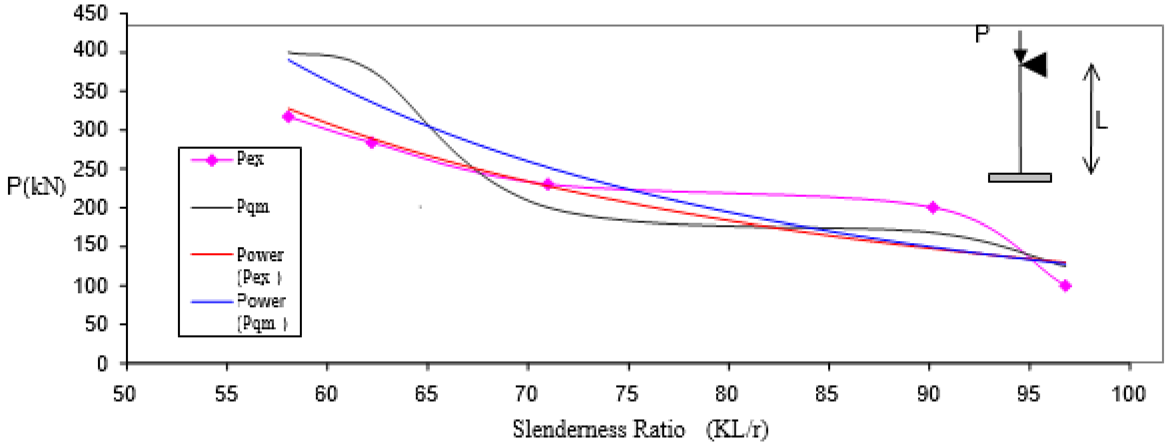

Fig. (6): Comparison between Quasilinear and Experimental Loads (Material Nonlinearity).

إنبعاج الأطر الفولاذية المقنطرة باعتبار لاخطية خواص المواد

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الخلاصة:

في هذا البحث تم اجراء تجارب على ثماني اطر فولاذية مقنطرة وذلك بتطبيق احمال مركزة فوق العناصر العامودية. تم تسجيل النتائج بواسطة مقاييس تشوه حيث تم ربطها بحاسوب.

تم مقارنة النتائج العملية بالدراسة النظرية من خلال تحليل الاطر الثمانية بالطرق التالية:

- التحليل شبه الخطي
- التحليل الخطي
- الكودات العالمية (AISC ASD, AISC LRFD)

ولتحديد الخواص الميكانيكية لعينات الفولاذ فقد تم اجراء فحوصات شد وضغط حيث تم استنتاج علاقة لاخطية بين الاجهاد والتشوه لهذه العينات. وبناء على هذه العلاقة فقد تم استنتاج علاقة معامل المرونة بالطاقة الداخلية لهذه المقاطع وذلك باستخدام طرق احصائية.

تم استخدام العلاقة الاخيرة في طرق التحليل النظريه اعلاه وتم عمل مقارنة بالنتائج العملية.

اثبتت هذه الدراسة حساب الترخيم باستخدام الطرق الخطية بأنها قريبة من النتائج العملية. اما في حساب مقاومة قوة الانبعاج، فقد اثبتت الدراسة ان استعمال طريقة التحليل اللاخطي يؤدي الى نتائج ادق وقريبة من النتائج العملية.