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INVESTIGATION OF PRACTICAL GEOMETRIC NONLINEAR ANALYSIS METHODS FOR SEMI RIGID STEEL PLANE FRAMES

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Abstract

In this study, various practical geometric nonlinear analysis methods which encompass both the member stability (P-δ) and frame stability (P-Δ) are investigated for different types of semi rigid beam-to-column connections. Odd power polynomial function is used for the moment–rotation relationships and beam-to-column connections are modeled with rotational flexible springs. In order to capture the P-delta effects of ach beam-to-column connection, a numerical study of a six storey one bay steel plane frame is used and the results for lateral storey drifts, end moments are summarized in tables and graphics.

Keywords: *Geometric Nonlinear, Semi Rigid, Steel Frame*

1. INTRODUCTION

In conventional analysis and design of steel frame structures, the actual behavior of rigid beam-to-column connections is simplified by assuming each joint accommodates full transfer of moment, and the connection is assumed to rotate as a rigid body. In a pinned joint, the joint is assumed to act as a frictionless hinge connection with no moments acting at the joint. However, the actual behavior of most bolted, welded or a combination of bolted and welded connections used in steel frame structures are indeed semi rigid and are governed by a nonlinear relationship between the connection moment, and the relative rotation between the connected members [1]. The nonlinear behavior between beam-to-column connections is called as connection nonlinearity [2].

In structural analysis, there is also another type of nonlinear behavior for steel frames. If the equilibrium and kinematic relationships are written with respect to the undeformed geometry of the structure, the analysis is referred to as a first order analysis and when the deformed geometry of the structure is used, the analysis is referred to as a second order analysis, P-delta analysis or geometric nonlinear analysis [3]. In linear elastic analysis, solutions are obtained in direct manner but in geometric nonlinear analysis, since the deformed geometry of the structure is not known during the formulation of the equilibrium and kinematic relationships, solutions generally require an iterative type of procedure. The geometric nonlinear analysis usually proceeds in an incremental manner and the deformed geometry of the structure obtained from the previous cycle of calculations is used as basis in order to formulate the equilibrium and kinematic relationships for the current cycle of calculations [4].

In this paper, practical geometric nonlinear analysis methods are investigated with respect to P-Δ and P-δ effects, and the behavior is discussed for both perfectly rigid assumption and different types of semi rigid beam-to-column connections. Semi rigid connection types are modeled with odd power polynomial function method. Double web angles, header plate, top and seat angles and t-stub types of connections are used in the study for modeling different type of beam-to-column connections in order to capture the importance of connection types on the steel frame behavior.

2. PRACTICAL GEOMETRIC NONLINEAR ANALYSIS METHODS

Geometric nonlinear behavior of steel frames is due to the presence of P-delta effects. P-delta effects can be discussed under two main topics; the member instability (P-δ) and frame instability (P-Δ) [5].

When lateral forces ΣP_x acts on a frame, the frame will deflect laterally until equilibrium position is reached. The corresponding lateral deflection may be calculated on the basis of the original geometry and is referred to here as the first order deflection and shown as Δ_1 in Figure 1. If, in addition to ΣP_x, vertical forces ΣP_y will interact with the lateral displacement Δ_1 caused by ΣP_x and drift the frame further until a new equilibrium position is reached in which the lateral deflection is denoted by Δ. The phenomenon by which the vertical forces ΣP_y interacts with the lateral displacement of the frame is called the P-Δ effect. The consequence of this effect is both an increase in lateral drift and in overturning moment. Since the additional deflection and overturning moment have detrimental effects on the stiffness and stability of the frame, they should be considered in steel plane frame analysis [6].

In order to capture the exact P-delta effects, structural engineers have to perform rigorous iterative nonlinear analysis which is also generally tedious and time-consuming. For cases in which accurate solutions are not required, it may be more advantageous to use practical analysis techniques by which second order effects are considered in an approximate and good manner.

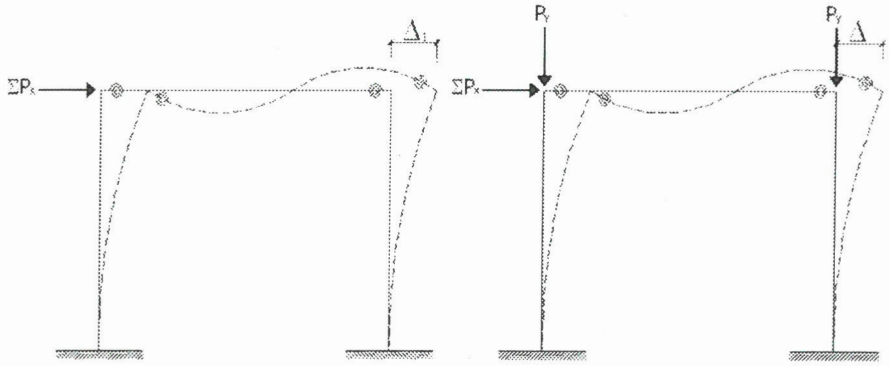


Figure 1. Lateral displacement of semi rigid frame under vertical and horizontal loadings

2.1. Stability Functions Method

The effect of loading acting on the deformed geometry of the structure creates what is referred as the second order effects [7]. Second order effects can also be considered by using approximate schemes. Stiffness matrix for frame element including stability functions can be used in practical second order analysis of steel frames and is given in equation (1) for compression members [8]:

$$[K_c]=\frac{EI}{L}\begin{bmatrix} \frac{A}{I} & 0 & 0 & -\frac{A}{I} & 0 & 0 \\ 0 & \frac{12}{L^2}\varphi_1 & \frac{6}{L}\varphi_2 & 0 & -\frac{12}{L^2}\varphi_1 & \frac{6}{L}\varphi_2 \\ 0 & \frac{6}{L}\varphi_2 & 4\varphi_3 & 0 & -\frac{6}{L}\varphi_2 & 2\varphi_4 \\ -\frac{A}{I} & 0 & 0 & \frac{A}{I} & 0 & 0 \\ 0 & -\frac{12}{L^2}\varphi_1 & -\frac{6}{L}\varphi_2 & 0 & \frac{12}{L^2}\varphi_1 & -\frac{6}{L}\varphi_2 \\ 0 & \frac{6}{L}\varphi_2 & 2\varphi_4 & 0 & -\frac{6}{L}\varphi_2 & 4\varphi_3 \end{bmatrix} \quad (1)$$

$$\varphi_1 = \frac{(\alpha L)^3 \, Sin(\alpha L)}{12\,(2 - 2\,Cos(\alpha L) - (\alpha L) Sin(\alpha L))} \tag{2}$$

$$\varphi_2 = \frac{(\alpha L)^2 \,(1 - Cos(\alpha L))}{6\,(2 - 2\,Cos(\alpha L) - (\alpha L) Sin(\alpha L))} \tag{3}$$

$$\varphi_3 = \frac{(\alpha L)(Sin(\alpha L) - (\alpha L) Cos(\alpha L))}{4(2 - 2\,Cos(\alpha L) - (\alpha L) Sin(\alpha L))} \tag{4}$$

$$\varphi_4 = \frac{(\alpha L)((\alpha L) - Sin(\alpha L))}{2\,(2 - 2\,Cos(\alpha L) - (\alpha L) Sin(\alpha L))} \tag{5}$$

$$\alpha = \sqrt{\frac{N}{E\,I}} \tag{6}$$

In equations (2-6); L is the length of the member, E is the modulus of Elasticity, I is the moment of inertia and N is the axial force of the member.

2.2. Geometrical Stiffness Matrix

The stability functions account for the change in bending stiffness of the member due to the presence of an axial force. Indeed, for a small axial force, the stability stiffness functions may become numerically unstable and to circumvent this situation and also to avoid the use of different expressions for compressive and tensile forces, Taylor series expansions can be applied [9]. Since in equation (1), the stiffness matrix of a beam-column element incorporating both P-delta effects is given in terms of the stability functions φ_i, first two terms can be in written in Taylor series expansion of these functions in equations (2-5) and the stiffness matrix can also be expressed as the sum of two matrices [10].

$$[K_0] = \frac{EI}{L}\begin{bmatrix} \frac{A}{I} & 0 & 0 & -\frac{A}{I} & 0 & 0 \\ 0 & \frac{12}{L^2} & \frac{6}{L} & 0 & -\frac{12}{L^2} & \frac{6}{L} \\ 0 & \frac{6}{L} & 4 & 0 & -\frac{6}{L} & 2 \\ -\frac{A}{I} & 0 & 0 & \frac{A}{I} & 0 & 0 \\ 0 & -\frac{12}{L^2} & -\frac{6}{L} & 0 & \frac{12}{L^2} & -\frac{6}{L} \\ 0 & \frac{6}{L} & 2 & 0 & -\frac{6}{L} & 4 \end{bmatrix} + \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6N}{5L} & \frac{N}{10} & 0 & -\frac{6N}{5L} & \frac{N}{10} \\ 0 & \frac{N}{10} & \frac{2NL}{15} & 0 & -\frac{N}{10} & -\frac{NL}{30} \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{6N}{5L} & -\frac{N}{10} & 0 & \frac{6N}{5L} & -\frac{N}{10} \\ 0 & \frac{N}{10} & -\frac{NL}{30} & 0 & -\frac{N}{10} & \frac{2NL}{15} \end{bmatrix} \tag{7}$$

First part of equation (7) is the familiar first order frame element stiffness matrix and the remaining part is the called as geometrical stiffness matrix [11]. The second part of the equation (7) can further be written as sum of two matrices as seen in the equation (8). The first part of the matrix in the equation (8) accounts for the P-δ effect, where as the second part of the matrix in the equation (8) accounts for the P-Δ effect [11].

$$[K_g] = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{N}{5L} & \frac{N}{10} & 0 & -\frac{N}{5L} & \frac{N}{10} \\ 0 & \frac{N}{10} & \frac{2NL}{15} & 0 & -\frac{N}{10} & -\frac{NL}{30} \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{N}{5L} & -\frac{N}{10} & 0 & \frac{N}{5L} & -\frac{N}{10} \\ 0 & \frac{N}{10} & -\frac{NL}{30} & 0 & -\frac{N}{10} & \frac{2NL}{15} \end{bmatrix} + \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{N}{L} & 0 & 0 & -\frac{N}{L} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -\frac{N}{L} & 0 & 0 & \frac{N}{L} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix} \tag{8}$$

2.3. Incremental Equivalent Lateral Load Method

Incremental equivalent lateral load method is also referred as the fictitious lateral load or the iterative P-Δ method [12]. This method investigates only with the P-Δ (frame instability) effect and the P-δ (member instability) effect is ignored.

Incremental equivalent lateral load method uses a set of lateral loading to simulate the P-Δ effects. The sum of the end moments of the member ($M = M_A + M_B$) in Figure 2. including the P-Δ effect can be written as in equation (9):

$$M = V \times h + P \times \Delta \qquad (9)$$

In equation (9), V is the member end shear, h is the member length, P is the member axial force, and Δ is the member relative end displacement. Also, equation (9) can be written as equation (10) where $\overline{V}$ represents the fictitious or equivalent shear.

$$M = V \times h + \left(\frac{P \times \Delta}{h}\right) \times h = \left(V + \overline{V}\right) \times h \qquad (10)$$

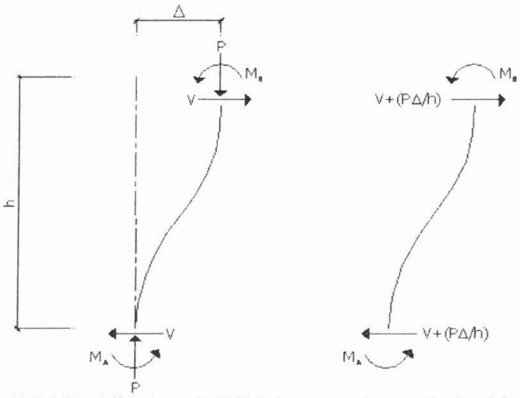


Figure 2. Fictitious shear force representing P-Δ effect

In order to calculate the sum of member end moments; the real shears V in conjunction with the fictitious shears $\overline{V}$ should be calculated.

The incremental equivalent lateral load method can readily to be extended to a multistory multi bay building. For such buildings, the fictitious shear at floor i. is computed from the equation (11):

$$\overline{V} = \frac{\Sigma P_i}{h_i} \times \left(\Delta_{i+1} - \Delta_i\right) \qquad (11)$$

In equation (11); ΣP_i is the sum of all column axial loads in storey i., h_i; is the height of storey i., Δ_{i+1} is the lateral displacement of storey i+1., and Δ_i is the lateral displacement of storey i. The fictitious lateral load which is applied at storey i. to simulate the P-Δ effect is obtained as the difference between the fictitious shears at adjacent stores:

$$\overline{H}_i = \overline{V}_{i-1} - \overline{V}_i \qquad (12)$$

In order to obtain the correct second order P-Δ moments for design, the incremental equivalent lateral load method must be applied in an iterative way. This is because the storey deflections Δ in equations are not known in advance. The procedure begins with a first order frame analysis. Storey deflections obtained from this analysis are used to calculate fictitious shears and fictitious lateral loads according to equation (11) and equation (12), respectively. The fictitious lateral loads are then applied with the real lateral loads and the frame is reanalyzed using first order theory. The process is repeated until the moments obtained in two consecutive analyses do not vary significantly.

3. SEMI RIGID MODELING

In conventional steel structure analysis and design, beam-to-column connection behavior is generally neglected, and fully rigid or ideally pinned behavior is assumed in order to facilitate the analyze procedures. The development in computer technology and high desire in representing the realistic steel frame behavior in structural modeling, semi rigid behavior of steel beam-to-column connections have gained interest and started to be investigated frequently [13].

Researchers have performed extensive experimental and analytical works on steel beam-to-column connections in last 30 years. Large amount of moment – rotation data collected from these studies and several connection models such as; linear, polynomial, B-spline, power, and exponential are developed [14, 15, 16].

3.1. Odd Power Polynomial Function Model

In order to provide a smoother moment-rotation curve, odd power polynomial function model of Frye and Moris is used in this study [17]. The odd power polynomial function model which is based on a procedure by Sommer has the form as in equation (13) [18]:

$$\varphi_c = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \tag{13}$$

In equation (13); K is a standardization parameter which is a function of the significant geometrical parameters such as adjoining member size, plate thickness, etc. and C_1, C_2, C_3 are curve-fitting constants [18]. The standardized moment rotation functions for each connection type based on the previous test data are listed in Table 1 [19]. The slope of the curve, which is the tangent connection stiffness S_c, is derived in equation (14) and the beam-to-column connections which are used in this study is given in Figure 3.

Table 1. Standardized Constants of the Odd Power Polynomial Function Model [19]

Connection Type	Curve-fitting coefficients	Standardized Constant	Tests
Double Web Angles	$C_1= 3.66\text{x}10^{-4}$ $C_2= 1.15\text{x}10^{-6}$ $C_3= 4.57\text{x}10^{-8}$	$K= d^{-2.4}t^{-1.81}g^{0.15}$	Munse et al. (1959) Sommer (1969)
Header Plate	$C_1= 5.10\text{x}10^{-5}$ $C_2= 6.20\text{x}10^{-10}$ $C_3= 2.40\text{x}10^{-13}$	$K= d^{-2.3}t^{-1.6}g^{1.6}w^{0.5}$	Sommer (1969)
Top and Seat	$C_1= 8.46\text{x}10^{-4}$	$K=d^{-1.5}t^{-0.5}f^{-1.1}l^{-0.7}$	Ratbun (1936)

Angles	$C_2 = 1.01x10^{-4}$ $C_3 = 1.24x10^{-8}$		Hechtman (1947) Brandes (1944)
T-Stub	$C_1 = 2.10x10^{-4}$ $C_2 = 6.20x10^{-6}$ $C_3 = -7.60x10^{-9}$	$K = d^{-1.5} t^{-0.5} f^{-1.1} l^{-0.7}$	Ratbun (1936) Douty (1964)

$$S_c = \frac{dM}{d\varphi_c} = \frac{1}{C_1K + 3C_2K(KM)^2 + 5C_3K(KM)^4} \quad (14)$$

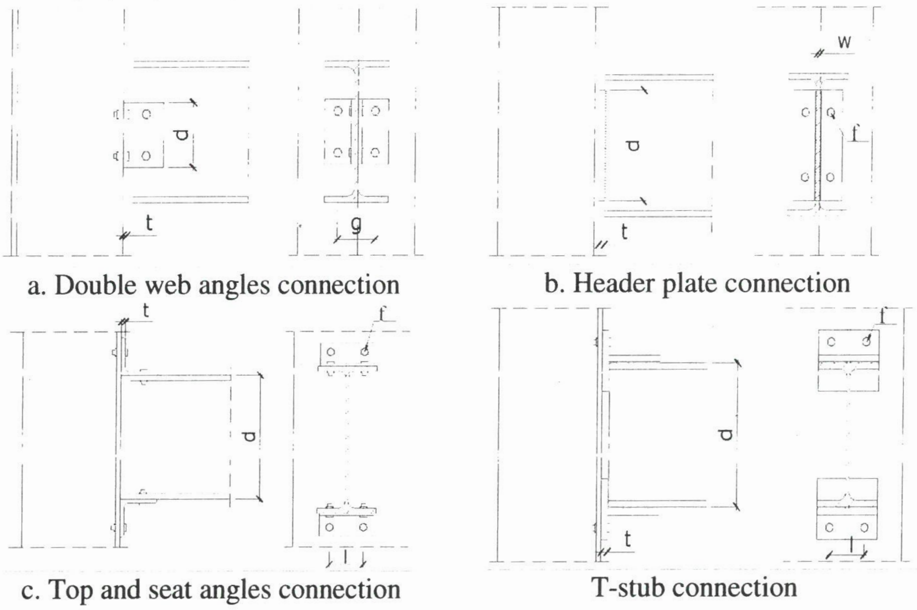


Figure 3. Polynomial function models and parameters for different types of connections

3.2. Semi Rigid Modeling using Rotational Springs

Advanced methods of steel structure analysis require the modeling of the beam-to-column connection in order to closely present the structural behavior. The modeling of the beam-to-column connection can be significantly simplified with a negligible loss of accuracy by using rotational spring elements as in Figure 4. The influence of semi rigid connections can easily be accounted by modifying the slope deflection equation for the beam and their presence will introduce relative rotations of $\theta_{r,A}$ and $\theta_{r,B}$ at the ends of the beam [20]. Since the stiffness of connections at the ends of beam $S^0_{c,A}$ and $S^0_{c,B}$, respectively, the relative rotation between the joint and the beam end can be given by equation (15) and (16).

$$\theta_{r,A} = \frac{M_A}{S^0_{c,A}} \quad (15)$$

$$\theta_{r,B} = \frac{M_B}{S^0_{c,B}} \quad (16)$$

The slope-deflection equations for the beam modified for the presence of semi rigid connections can be written as equations (17) and (18).

$$M_A = \frac{EI}{L}\left[4\left(\theta_j - \frac{M_A}{S^0_{c,A}}\right) + 2\left(\theta_k - \frac{M_B}{S^0_{c,B}}\right)\right] \tag{17}$$

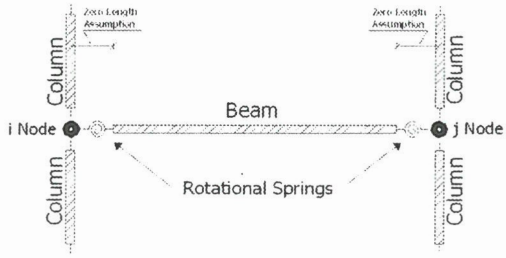


Figure 4. Semi rigid beam and column element modeling with rotational springs

$$M_B = \frac{EI}{L}\left[2\left(\theta_j - \frac{M_A}{S^0_{c,A}}\right) + 4\left(\theta_k - \frac{M_B}{S^0_{c,B}}\right)\right] \tag{18}$$

Also, the slope-deflection equations can be written and transformed into beam matrix form as in equation (19) [21].

$$[K_b] = \begin{bmatrix} \frac{AE}{L} & 0 & 0 & -\frac{AE}{L} & 0 & 0 \\ 0 & \frac{EI}{L^3}\Psi_1 & \frac{EI}{L^2}\Psi_2 & 0 & -\frac{EI}{L^3}\Psi_1 & \frac{EI}{L^2}\Psi_3 \\ 0 & \frac{EI}{L^2}\Psi_2 & \frac{EI}{L}\Psi_4 & 0 & -\frac{EI}{L^2}\Psi_2 & \frac{EI}{L}\Psi_5 \\ -\frac{AE}{L} & 0 & 0 & \frac{AE}{L} & 0 & 0 \\ 0 & -\frac{EI}{L^3}\Psi_1 & -\frac{EI}{L^2}\Psi_2 & 0 & \frac{EI}{L^3}\Psi_1 & -\frac{EI}{L^2}\Psi_3 \\ 0 & \frac{EI}{L^2}\Psi_3 & \frac{EI}{L}\Psi_5 & 0 & -\frac{EI}{L^2}\Psi_3 & \frac{EI}{L}\Psi_5 \end{bmatrix} \tag{19}$$

$$\Psi_1 = \frac{\left(4+(12\times EI)/\left(L\times S^0_{c,B}\right)\right)+(4)+\left(4+(12\times EI)/\left(L\times S^0_{c,A}\right)\right)}{\left(1+(4\times EI)/\left(L\times S^0_{c,A}\right)\right)\times\left(1+(4\times EI)/L\times S^0_{c,B}\right)-(EI/L)^2\times 4/\left(S^0_{c,A}\times S^0_{c,B}\right)} \tag{20}$$

$$\Psi_2 = \frac{\left(4+(12\times EI)/\left(L\times S^0_{c,B}\right)\right)+(2)}{\left(1+(4\times EI)/\left(L\times S^0_{c,A}\right)\right)\times\left(1+(4\times EI)/L\times S^0_{c,B}\right)-(EI/L)^2\times 4/\left(S^0_{c,A}\times S^0_{c,B}\right)} \tag{21}$$

$$\Psi_3 = \frac{\left(4+(12\times EI)/\left(L\times S^0_{c,A}\right)\right)+(2)}{\left(1+(4\times EI)/\left(L\times S^0_{c,A}\right)\right)\times\left(1+(4\times EI)/L\times S^0_{c,B}\right)-(EI/L)^2\times 4/\left(S^0_{c,A}\times S^0_{c,B}\right)} \tag{22}$$

$$\Psi_4 = \frac{\left(4+(12\times EI)/\left(L\times S^0_{c,B}\right)\right)}{\left(1+(4\times EI)/\left(L\times S^0_{c,A}\right)\right)\times\left(1+(4\times EI)/L\times S^0_{c,B}\right)-(EI/L)^2\times 4/\left(S^0_{c,A}\times S^0_{c,B}\right)} \tag{23}$$

$$\Psi_5 = \frac{\left(4+(12\times EI)/\left(L\times S^0_{c,A}\right)\right)}{\left(1+(4\times EI)/\left(L\times S^0_{c,A}\right)\right)\times\left(1+(4\times EI)/L\times S^0_{c,B}\right)-(EI/L)^2\times 4/\left(S^0_{c,A}\times S^0_{c,B}\right)} \tag{24}$$

4. NUMERICAL STUDY

A numerical example of one bay six storey steel frame is analyzed by using various practical second order analysis methods for both rigid assumption and four different types of semi rigid connections. The modulus of elasticity is assumed as $2.1x10^6$ kg/cm^2 and steel frame geometry is given in Figure 5. Double web angles, header plate, top and seat angles and t-stub types of connections are used in the study for modeling different type of beam-to-column connections. The first order and second order behavioral difference between semi rigid connections and rigid assumption is widely discussed in terms of lateral displacement and end moments.

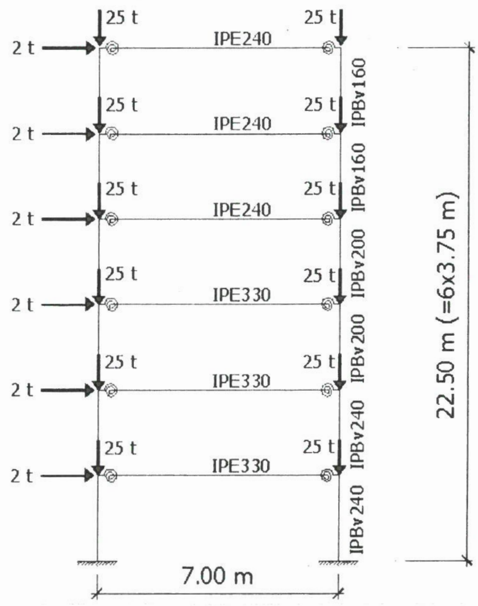
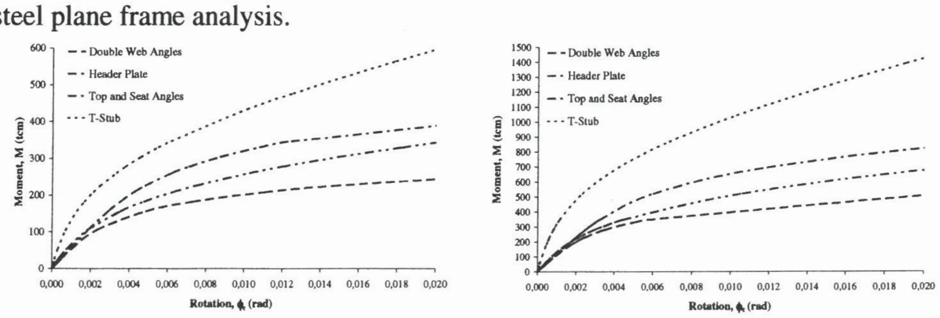


Figure 5. Six storey and one bay steel plane frame

The moment–rotation (M–θ_r) behaviors of four different semi rigid connections are derived by using odd power polynomial function model. In all types of connections L 150.150.15 angle elements are used and moment–rotation curves for beam elements are shown in Figure 6. The moment – rotation curves of all four types of connections are nonlinear. The double web angle represents flexible joint and the T-stub connection presents a rather rigid joint behavior. Hence the analyses of connection behavior used in steel frame analysis are approximate with drastic simplifications, the initial connection stiffness values are used in the practical second order steel plane frame analysis.



a. IPE 240 Beam

b. IPE 330 Beam

Figure 6. Moment – rotation curves for beam elements

4.1. Numerical Study Results

A numerical example of one bay six storey steel frame is analyzed for each connection type of double web angles, header plate, top and seat angles, t-stub and theoretical perfect rigid assumption. Also, all the semi rigid connections and rigid assumption are investigated for first order and second order analysis methods separately. Stability functions method, geometric stiffness method, P-δ stiffness method, P-Δ stiffness method and incremental equivalent lateral load method are used in practical geometric nonlinear analysis. All results are also compared with a well known structural engineering program Sap2000 [22].

The first order and various practical geometric nonlinear analysis results of top storey lateral displacements for four types of semi rigid connections and perfectly rigid assumption are given in Figure 7. The lateral displacement result of first order analysis of double web angles type connection is much higher than second order analysis of perfectly rigid assumption. This shows the importance of accounting semi rigid connection behavior in the practical geometric nonlinear analysis.

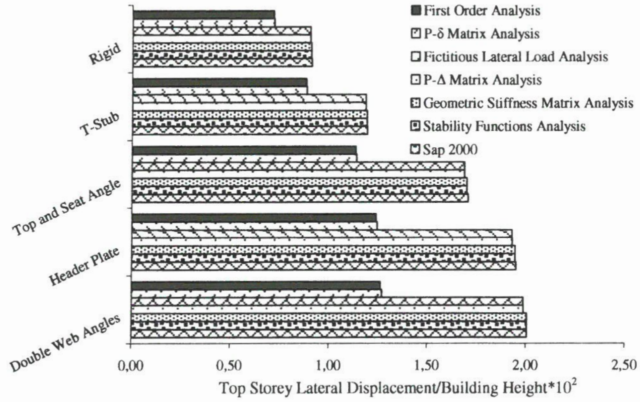
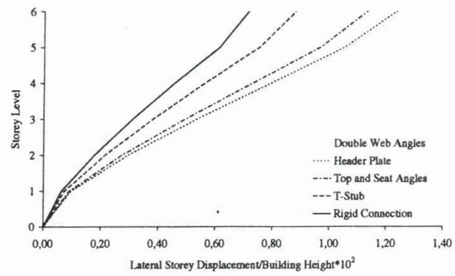
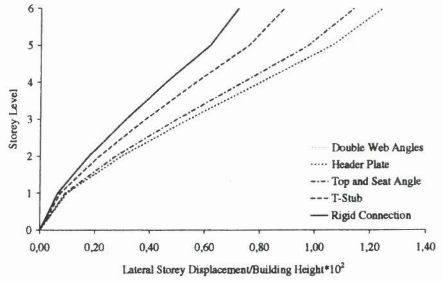


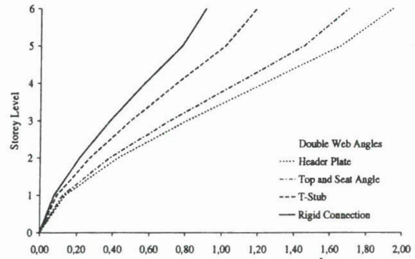
Figure 7. Top storey lateral displacements results for practical geometric nonlinear analysis of four different types of semi rigid connections and rigid assumption



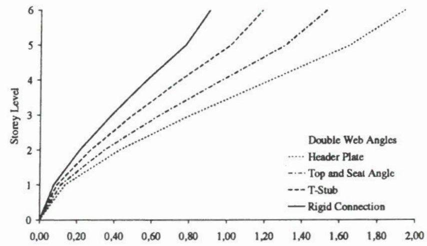
a. First order analysis



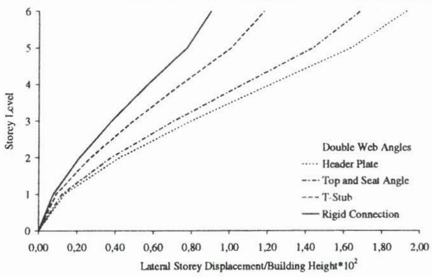
b. P-δ stiffness analysis



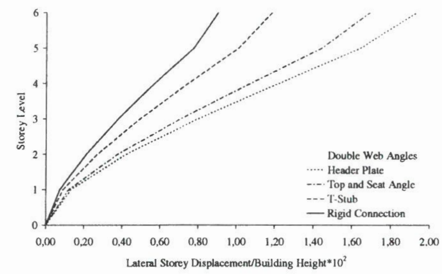
c. Stability functions analysis



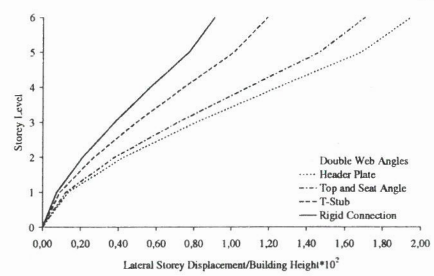
d. Geometric stiffness analysis



e. P-Δ stiffness analysis



f. Incremental equivalent lateral load method



g. SAP2000 analysis results

Figure 8. Lateral storey displacements results for practical geometric nonlinear analysis of four different types of semi rigid connections and rigid assumption

The storey drifts are obtained for each storey level and are shown in Figure 8 and the member moment values of some selected joints are given in Table 2.

Table 2. End Moment Values of Selected Joints for Practical Geometric Nonlinear Analysis of Four Different Types of Semi Rigid Connections and Rigid Assumption

Analysis Type	Connection Type	2. Storey Beam End Moments (tcm)		4. Storey Beam End Moments (tcm)		Right Base Column End Moments (tcm)	
		i. End	j. End	i. End	j. End	i. End	j. End
First Order Analysis	Double web angles	1173	1173	613	613	2549	2539
	Header plate	1181	1181	618	618	2527	2518
	Top and seat angles	1225	1225	627	627	2418	2408
	T-stub	1352	1352	628	628	2104	2095
	Rigid	1425	1425	644	644	1901	1892
Stability Functions Analysis	Double web angles	1794	1795	1023	1023	3392	3367
	Header plate	1790	1790	1018	1018	3341	3316
	Top and seat angles	1781	1781	983	983	3097	3073
		1775	1776	881	881	2502	2481
		1764	1764	843	843	2171	2153

	T-stub Rigid						
Geometric Stiffness Matrix Analysis	Double web angles	1794	1794	1023	1023	3392	3367
	Header plate	1790	1790	1018	1018	3341	3316
	Top and seat angles	1781	1781	983	983	3097	3073
	T-stub	1775	1775	881	881	2502	2481
	Rigid	1764	1764	843	843	2171	2153
P-Δ Stiffness Analysis	Double web angles	1775	1775	1013	1013	3414	3404
	Header plate	1771	1771	1008	1008	3363	3353
	Top and seat angles	1766	1766	974	974	3118	3108
	T-stub	1764	1764	874	874	2520	2510
	Rigid	1755	1756	837	837	2187	2177
P-δ Stiffness Analysis	Double web angles	1182	1182	617	617	2527	2507
	Header plate	1190	1190	622	622	2506	2486
	Top and seat angles	1234	1234	631	631	2397	2378
	T-stub	1359	1359	632	632	2085	2070
	Rigid	1431	1431	647	647	1887	1870
Increment al Equivalent Lateral Load Method	Double web angles	1775	1774	1012	1012	3407	3409
	Header plate	1771	1770	1008	1007	3356	3358
	Top and seat angles	1766	1766	974	973	3112	3113
	T-stub	1764	1764	874	874	2516	2513
	Rigid	1755	1755	836	836	2184	2179
Sap2000	Double web angles	1806	1807	1023	1023	3439	3428
	Header plate	1800	1800	1018	1018	3385	3374
	Top and seat angles	1781	1782	980	980	3129	3119
	T-stub	1765	1766	873	873	2520	2510
	Rigid	1753	1753	835	834	2185	2175

5. CONCULUSIONS

In this study, various practical geometric nonlinear analysis techniques applicable to steel plane frames are investigated by using different types of beam-to-column connections and perfectly rigid assumption. The conclusions of this study dealing with the investigation of various practical geometric nonlinear analysis methods for semi rigid beam-to-column can also be summarized as follows:

1. Practical geometric nonlinear analysis methods can capture P-delta effects reasonable when compared with Sap2000 result. Also, P-Δ and P-δ effects and their ability to capture second order effects are discussed for rigid and semi rigid frames.

2. Semi rigid connections have a great influence on the behavior of steel plane frames as seen from storey drifts and end moment values of the numerical study. In some of the beam-to-column connection types, first

order analysis with the semi rigid modeling gives more lateral drift than second order analysis with perfectly rigid assumption. This dramatically shows that, when connection rigidity is significantly overestimated by assuming it to be perfectly rigid, the lateral drift and member internal forces will be under estimated.

3. The connection deformations of semi rigid beam-to-column connections have a destabilizing effect on the steel frame stability and additional drift occurs as a result of the decrease in effective stiffness of the members to which the semi rigid connections are attached. An increased frame drift intensifies the P-delta effects, and hence the overall behavior of the steel frame is affected significantly.

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PRATİK GEOMETRİK NONLİNEER ANALİZ YÖNTEMLERİ İLE YARI RİJİT DÜZLEM ÇELİK ÇERÇEVELERİN İNCELENMESİ

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Özet. Bu çalışmada, eleman stabilitesi (P-δ) ve çerçeve stabilitesi (P-Δ) kavramlarını kapsayan birçok pratik geometrik nonlineer analiz yöntemi farklı yarı rijit kiriş-kolon birleşim tipleri için incelenmiştir. Tek kuvvetli polinom fonksiyonları moment-eğrilik ilişkisi için kullanılmış ve kiriş-kolon birleşimi esnek dönme yaylar ile modellenmiştir. Her kiriş-kolon birleşim tipi için P-delta etkilerini elde etmek amacıyla altı katlı ve tek açıklıklı çelik düzlem çerçeve sayısal çalışma için kullanılmış ve elde edilen yatay kat ötelemeleri, eleman uç momentleri sonuçları tablolar ve grafikler halinde özetlenmiştir.

Anahtar Kelimeler: *Geometrik Nonlineer, Yarı Rijit, Çelik Çerçeve*