PAMUKKALE ÜNİVERSİTESİ MÜHENDİSLİK FAKÜLTESİ
PAMUKKALE UNIVERSITY ENGINEERING COLLEGE
MÜHENDİSLİK BİLİMLERİ DERGİSİ
JOURNAL OF ENGINEERING SCIENCES
YIL : 1998
CİLT : 4
SAYI : 1-2
SAYFA : 537-539

FRAMES WITH SEMIRIGID STRUCTURAL CONNECTIONS

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ABSTRACT

During recent years, researchers have focused their attention on the actual behaviour of beam-to-column joints. In general, structural analysis of frames are performed either for rigid joints or pin-ended connections. Nevertheless in practice, no joints are either fully rigid or actually pinned. A presumed rigid joint always allows for a relative rotation or presumed pin-ended connections are never proper hinges. Therefore all joints should be treated as semi-rigid. To meet semi-rigidly connected behaviour an empirical interaction equation, given in Turkish Standard TS 4561, is adopted into analysis by using C_m values which modifies the moment relating with effective buckling length of member, where C_m values are different for rigidly and semi rigidly connected frame elements.

Key Words : Semirigid, Joints, Frames, Buckling length

YARI RİJİT BAĞLANTILI ÇERÇEVELER

ÖZET

Son yıllarda araştırmacılar kiriş-kolon bağlantılarının gerçek davranışları üzerine çalışmalarını yoğunlaştırmışlardır. Genelde çerçevelerin yapısal analizi rijit veya mafsallı bağlantı olarak ortaya konulmaktadır. Buna rağmen pratikte hiçbir düğüm noktası tam rijit veya gerçek mafsal değildir. Rijit düşünülen bir bağlantı daima relatif dönmelere izin verir veya mafsal olarak düşünülen bağlantılar hiç bir zaman tam bir mafsal özelliği taşımazlar. Bu sebeple, bütün düğüm noktaları yarı rijit olarak ele alınmalıdır. Yarı rijit bağlantıların davranışını tanımak için eleman etkili burkulma uzunluğuna bağlı moment değişim katsayısı olan C_m değerinin kullanılmasıyla çözümü kabul eden Türk Standardı TS 4561'de ampirik bir formül verilmiştir. Buradaki C_m değeri rijit ve yarı rijit bağlı çerçeve elemanları için farklıdır.

Anahtar Kelimeler : Yarırijit, Düğüm noktası, Çerçeveler, Burkulma boyu

1. INTRODUCTION

For conventional analysis and design of steel framed structures, the actual behaviour of beam to column connections is simplified to the two idealised extremes of either rigid joint behaviour or pinned joint behaviour, because such idealized joint behaviour greatly simplifies the analysis and design process. However, most connections used in steel frameworks actually exhibit semirigid deformation behaviour that can contribute substantially to overall structure displacements. In reality, experimental investigations of actual joint behaviour have clearly demonstrated that a pinned joint connection process a certain amount of rotational stiffness, while a rigid joint connection rotational possesses some degree of flexibility. Often, also, flexible connection behaviour significantly affects the internal force distribution in the members of a frame. Therefore the neglect of real connection behaviour may lead to unrealistic predictions of the response and strength of steel structures, and so to unreasonable designs in practice. Thus, in actuality, steel frame connections should be treated as semirigid connections for the purposes of analysis and design.

In past thirty five years, reasonable research has been done to determine the actual behaviour of steel frames accounting for the effect of connection flexibility, for example, Monforton and Wu (1963), Lui and Chen (1986), and Cunningham (1990). Nevertheless, much of this research has been considered mainly analysis problem.

This study presents an integrated method for the optimum design of steel frames that accounts for the behaviour of semirigid connections in the aspect of frame behaviour. The optimum design sought by method has the minimum weight of members and ensures that stresses and drifts are within acceptable limits. An iterative algorithm is applied for design of two braced steel frame.

2. THE AUTOMATED ANALYSIS AND DESIGN PROGRAM OF FRAMES

The program converges to a final design through a sequence of analysis and design iterations (Gönen, 1989). The automated design procedure starts with the computation of static and dynamic displacements and the corresponding member forces which are determined from factored design loads using a linear elastic analysis. If the strong column-weak girder design option is selected, the column and moments are modified to reflect this design philosophy which seeks to force all plastic deformation under extreme load into the girder rather than the column. The strength design of members is then carried out followed by a grouping of the specified elements. Grouping is done to reflect certain practical considerations such as splicing columns every two or three floor and holding beam sizes constant on a particular floor level.

Next, results of current design iteration are compared with the results of previous iteration. If the convergence is achieved, then the story drifts are compared with the specified allowable story drift index. If the drift criteria are not satisfied, the properties of certain members are changed to stiffen the structure. This is followed by a computation of structure displacements to ensure the satisfaction of drift criteria. Satisfaction of the drift criteria marks the end of the automated design procedure.

3. ADOPTION OF SEMIRIGID CONNECTION BEHAVIOR

The column design algorithm is based on the interaction formula of Turkish Standard TS 4561 (Anon., 1985).

$$\frac{N}{N_e}+\frac{Cm*M_i}{\left(1+\frac{N}{N_e}\right)*M_p}\leq 1 \qquad (1)$$

Where N is the applied axial load, M_i is the applied bending moment, N_{cr} is the maximum axial load in the absence of bending moment, N_e is the Euler bucking load, M_p is the plastic moment capacity, and C_m is a coefficient which accounts for the moment gradient in considering P-D effects across the member.

C_m values are given in TS 4561 for different load conditions for both rigid and semirigid end connections. Naturally, C_m values for rigid end connections are different from one for semirigid end connections for the same load conditions. For example, for the uniform span loading for semirigid end connections

$$Cm=1-\frac{0.2\left[1-\left(\frac{l_K}{l}\right)^2\right]}{0.13+\left(\frac{l_K}{l}\right)}\left(\frac{N}{N_e}\right) \qquad (2)$$

Where l_K is the effective buckling length of member and l is the span length of member.

And for rigid connections

$$Cm=1-0.4\frac{N}{N_e} \qquad (3)$$

Inserting Eqn (2) into Eqn. (1) into the design algorithm can give us an opportunity to analyse the frame elements which have semirigid end connections without specifying end connection types in terms of buckling length of frame elements. Thus, it can be possible to compare the weights of frames which have rigid and semirigid connection elements. For this purpose two braced frame will be taken as case study.

4. EXAMPLE APPLICATIONS

4. 1. Example 1

The three bay, ten story and middle span is K-braced (like V) frame is considered shown in Figure 1. This frame has been previously analysed and designed by

Anderson (1975) using conventional design methods. The width of each bay is 25 feet (7.6m) and the height of ground story is 15 feet (4.57m)

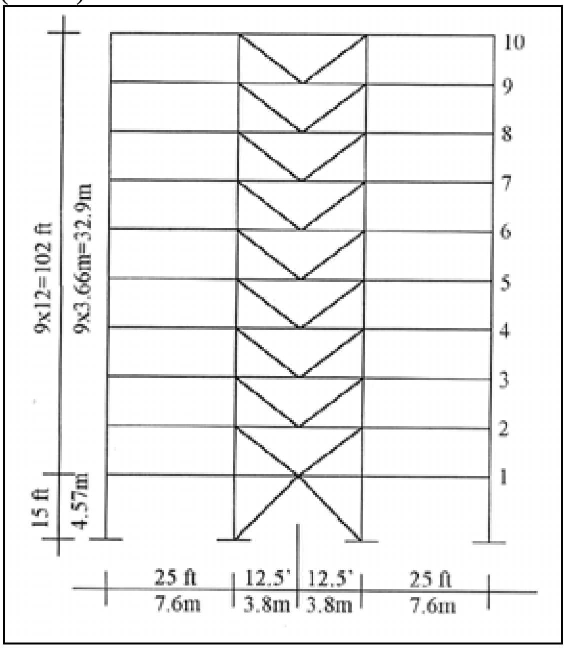

Figure 1. Frame example

and other stories are 12 feet (3.66m). The load combinations consist of dead load, live load, wind load and earthquake load shown in Table 1.

Table 1. Loads on Frame

Vertical Loads		
Dead Load	Roof	100 psf (0.489 t/m^2)
	Other Stories	120 psf (0.586 t/m^2)
Live Load	Roof	06 psf (0.078 t/m^2)
	Other Stories	70 psf (0.342 t/m^2)

Lateral Loads on Story Levels		
10	9.0 (4.08t)	30.52 (13.84 t)
9	18.0 (8.16 t)	33.10 (15.01 t)
8	16.5 (7.48 t)	29.54 (13.40 t)
7	15.0 (6.80 t)	25.96 (11.78 t)
6	15.0 (6.80 t)	22.40 (10.16 t)
5	15.0 (6.80 t)	18.82 (8.54 t)
4	13.74 (6.23 t)	15.26 (6.92 t)
3	12.0 (5.44 t)	10.30 (4.67 t)
2	9.76 (4.43 t)	8.12 (3.68 t)
1	12.0 (5.44 t)	4.36 (1.98 t)

Assuming all connections are fully rigid and the maximum allowable slenderness ratios for each bracing member are specified as 200 in compression and 300 in tension the total weight of frame designed by Anderson was 49.75 tons. Under same load combination and similar design algorithm according to TS 4561 except maximum allowable slenderness ratios for bracing members taken as 250 for both compression and tension, the total weight of frame designed by Automated Computer program of the writer was found 46.88 tons, which is 6 % lighter than Anderson’ s design. The overall weight of the same frame under the same load combination with semirigid connection conditions is found approximately 1 % heavier, because design load combinations include lateral loads, since fully rigid beam-column connections provide for greater lateral stiffness of the structure and thus allow for smaller size of column members.

4. 2. Example 2

The considered same frame with one eccentric bracing element in the middle span gave approximately 1.5 % heavier weight comparing to the fully rigid connected frame, because here also load combinations have lateral load. However, it is needed more study in this point to complete the subject.

5. CONCLUSION

The use of semirigid joints instead of rigid ones sometimes results in an increase of the amount of steel needed (if the lateral loads governs the design), but also a strong decrease of fabrication costs through a simplified detailing of joints (less stiffening for example). Frames with bolted end-plate connections and angle connections are believed to be more economical to erect and fabricate than frames with fully welded rigid connections.

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