패널존 변형을 포함한 철골모멘트골조의 탄성층간변위 근사해석

Approximate Analysis of Elastic Story Drift of Steel MRFs including Effects of Panel Zone Flexibility

이 철 호[1)]

Lee, Cheol Ho

국문초록 : 본 연구에서는 철골모멘트골조의 패널존변형을 명시적으로 고려한 탄성층간변위의 근사해석 방법을 제안하였다. 본 방법은 고전적 포탈법의 가정 및 D 치법에 기반한 해석적 접근법이다. 즉 포탈법의 가정에 따라 횡력을 받는 골조를 보-기둥 부분골조로 분해한 후 대표적 내부 부분골조의 보, 기둥 및 패널존에서 기인하는 모든 횡변위 성분을 해석적으로 계산한다. 이때에 필요한 모든 내력(가령 패널존 전단변형 산정을 위한 보의 불균형모멘트)의 결정에 D 치법을 이용한다. 구조바닥의 강막작용을 고려하면 위의 과정을 통하여 산출된 대표적 내부 부분골조의 횡변위는 전체 골조의 횡변위와 거의 동일할 것으로 기대할 수 있다. 본 방법의 타당성 여부는 반강절 접합요소를 사용한 해석적 엄밀해와 비교하여 검증하였으며 만족스런 결과를 주는 것을 확인하였다. 본 연구의 방법에 의해 컴퓨터해석에 의하지 않고도 철골모멘트골조의 탄성층간변위를 실용성있는 정확도로서 신속하게 산정할 수 있으므로 본 연구의 결과는 예비적 횡강성 평가에 유용하게 사용될 수 있다. 또한 본 방법의 적용과정에서 해석자는 철골모멘트골조의 횡변위거동에 관한 물리적 감각을 증진시킬 수 있을 것으로 사료된다.

1. INTRODUCTION

The panel zone in steel moment-resisting frames (MRFs) is the beam-column intersection which is subjected to high shear stresses under lateral earthquake loads(FIG. 1). Well-detailed panel zone can act as a major source of earthquake energy dissipation through the stable shear yielding under the severe seismic excitation. On the other hand the panel zone flexibility and finite size of the joint can significantly affect the elastic story

1) 정회원, 경남대학교 건축공학과 부교수

drift of steel MRFs and need to be considered in stiffness design. But conventional analysis for a seismic design is to base upon frame center line dimensions without explicitly considering the panel zone flexibility and finite size of the joint.

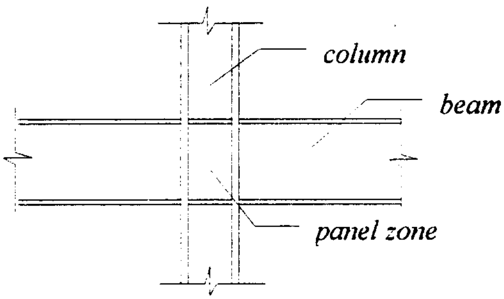


FIG. 1 Panel Zone in Beam-Column Intersection

To precisely consider the panel zone flexibility, an analytical procedure incorporating semi-rigid connection element is required (for example, Prakash et al. 1993). Such an approach needs additional computational efforts in modeling and does not seem to be common in current engineering practice. As an alternative, without increasing computational efforts, somewhat arbitrarily defined rigid end-offsets (say 50%, Habibullah 1989) were often used to compensate for the apparent underestimation of beam-column panel joint stiffness in the conventional analysis. This widely accepted approach, as was noted by Tsai and Popov (1990), may significantly underestimate the elastic lateral drifts of steel MRFs depending on the panel zone design strategies. Tsai and Popov (1990) proposed a simple correction procedure to include the effects of panel zone flexibility in the story drift calculation. Their procedure has the advantage of allowing the use of conventional computer frame-analysis program by eliminating the need for adding semi-rigid connection elements at the joint.

The objective of this study is to propose a simple and reasonably accurate analytical procedure for calculating the elastic story drift of steel MRFs including the effects of panel zone flexibility, thereby providing a practically useful tool.

2. BASIS AND ASSUMPTIONS OF THE PROCEDURE

Only lateral loading is assumed for a simplified derivation. Based on portal method assumptions, a frame can be resolved into beam-column subassemblies having inflection points at mid-spans of beams and mid-heights of columns. For the first story columns with fixed support conditions, it is well-known that the inflection points tend to move upward and usually stay at sixty to seventy percent story height from the bottom. Therefore inflection points in the first story columns with fixed supports are assumed to be at sixty-five percent of the story height from the bottom. The lateral deflection formulation of such a subassembly is already available (Krawinkler 1978). In this study Krawinkler's existing formulation is extended to account for more general member arrangement and loading conditions.

Many of practical steel frame designs usually yield the regular framing system having relatively uniform stiffness distributions of members. For such a regular frame the internal member forces such as the beam shear, column shear, and unbalanced beam moment can be determined with a sufficient accuracy using the D-value method (Muto 1974, AIJ 1988). The D-value method has often been used by practicing engineers in distributing manually the story shear force to the columns. To determine all the internal forces required for calculating the lateral deflection of a subassembly, the D-value method is used in its original form in this study. That is, the flexibility and finite size of the panel joint are neglected at this stage. Considering the rigid diaphragm behavior under the lateral loading, this study assumes that the elastic story drift of a *representative interior subassembly* is approximately equal to that of a whole steel MRF. The validity of this assumption is evaluated by comparing with the analytically precise analysis which explicitly considers the panel zone flexibility using semi-rigid connection element.

3. DETERMINATION OF COLUMN SHEAR FORCE

The aim of the D-value method is to determine approximately the share of each of the columns in a particular story in resisting the story shear force V. Subsequently, all the internal force distributions within a subassembly can be found based on statics. The shear force of the i^{th} column in a particular story, H_i, is given as follows in this approximate method :

$$D_i = ak_c \quad (1)$$

$$H_i = \frac{D_i}{\sum_{all\ columns} D_j} V \quad (2)$$

where D_i = relative "shear" stiffness of the i^{th} column, a = degree of restraint provided by the beams connected to the i^{th} column, and k_c = relative flexural stiffness of the i^{th} column. See NOTATION for the definitions of the symbols. Table 1 summarizes the calculation of the parameter a depending on the story locations and support conditions. Also refer to FIG. 2. The detailed examinations of the D-value method can also be found in the standard text book (Paulay and Priestley 1992) and are not reproduced here.

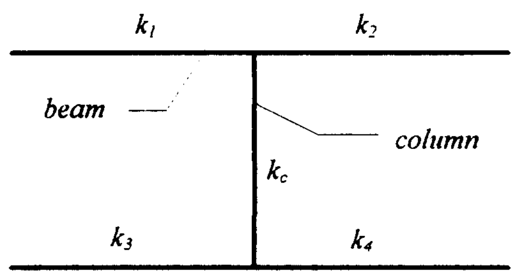


FIG. 2 Relative Flexural Stiffnesses of Members

Table 1. Calculation of Parameter a(AIJ 1988)

(1)	General Story (2)	First Story (3)	
		Fixed Support	Hinged Support
a	$\frac{\bar{k}}{\bar{k}+2}$	$\frac{\bar{k}+0.5}{\bar{k}+2}$	$\frac{0.5\bar{k}}{1+2\bar{k}}$
$\bar{k}$	$\frac{(k_1+k_2+k_3+k_4)}{2k_c}$	$\frac{(k_1+k_2)}{k_c}$	$\frac{(k_1+k_2)}{k_c}$

Note : k_c and k_1-k_4 represent the relative flexural stiffinesses of the beams and column, i.e., the second moment inertia of the member section divided by the member length. Also refer to FIG. 2.

4. ELASTIC LATERAL DRIFT COMPONENTS IN STEEL MRFS

The elastic lateral deflection in steel MRFs is the sum of three deflection components ; 1) lateral deflection caused by flexural deformations in the column, 2) lateral deflection caused by shear deformations in the panel zone, and 3) lateral deflection caused by flexural deformations in the beams. Krawinkler (1978) derived the approximate analytical expressions of a steel beam-column subassembly based on the portal method assumptions. Krawinkler's existing formulation is extended to account for more general member arrangements and loading conditions in the following.

Lateral Deflection Caused by Flexural Deformations in the Column

This lateral drift component δ_c is the cantilever bending deformations of the columns excluding the portion of the panel zone depth d_b and can be simply written as follows (refer to FIG. 3) :

$$\delta_c = \delta_{ct}+\delta_{cb} = \frac{(h_t/2-d_b/2)^3 H_t}{3EI_{ct}} + \frac{(h_b/2-d_b/2)^3 H_b}{3EI_{cb}} \quad (3)$$

By dividing δ_c in Equation (3) with the distance between the inflection points of the columns in a given subassembly, the elastic story drift ratio component θ_c due to the column flexural deformations can be expressed as

$$\theta_c = \frac{\delta_c}{(h_t/2+h_b/2)} \quad (4)$$

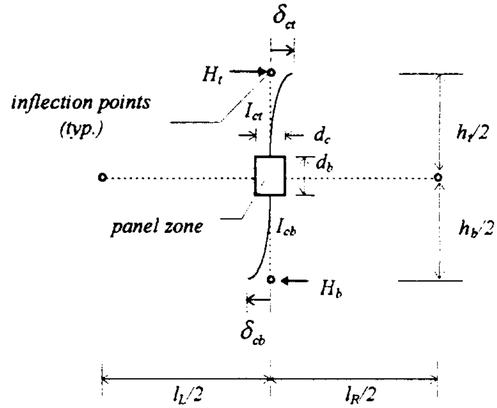


FIG. 3 Lateral Deflection Due to Column Flexural Deformations

Lateral Deflection Caused by Shear Deformations in the Panel Zone

Beam shear is determined by applying the overall moment equilibrium condition to the subassembly (see FIG. 4). Neglecting the higher order term related to the column depth, the unbalanced beam moment, ΔM, which is directly related to the shear deformation of the panel zone, can be computed as

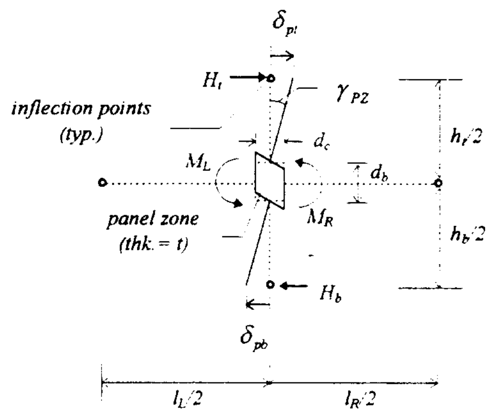


FIG. 4 Lateral Deflection Due to Panel Zone Shear Deformation

$$\Delta M = M_L + M_R = \frac{(h_t H_t + h_b H_b)}{2}\left(1 - \frac{2d_c}{l_L + l_R}\right)$$

$$\cong (h_t H_t/2 + h_b H_b/2) \tag{5}$$

Assuming that the beam moment is transferred entirely by the beam flanges, and neglecting the column shear forces outside the panel zone, the shear force acting on the panel zone V_{panel} is approximated by Equation (6).

$$V_{panel} = \frac{\Delta M}{d_b} = \frac{(h_t H_t/2 + h_b H_b/2)}{d_b} \tag{6}$$

The shear deformation angle of the panel zone γ_{PZ} (in radians) is calculated by dividing V_{panel} with the panel zone shear stiffness Gd_ct. The resulting expression for γ_{PZ} is given in Equation (7). The thickness of panel zone t includes that of a doubler plate, if any.

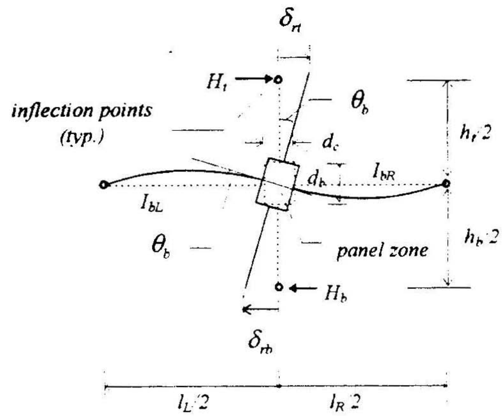
$$\gamma_{PZ} = \frac{V_{panel}}{Gd_ct} = \frac{\Delta M}{Gd_bd_ct} \cdot \frac{(h_t H_t/2 + h_b H_b/2)}{Gd_bd_ct} \tag{7}$$

The shear deformation angle of the panel zone γ_{PZ} corresponds approximately to the elastic story drift ratio due to the shear deformation of the panel zone. Therefore the lateral deflection δ_p due to the shear deformation of the panel zone at a given floor is approximated by

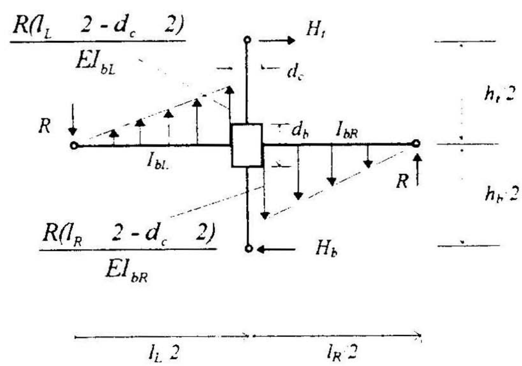
$$\delta_p = \delta_{pt} + \delta_{pb} = \gamma_{PZ}\left(\frac{h_t}{2} + \frac{h_b}{2}\right) \tag{8}$$

<u>Lateral Deflection Caused by Flexural Deformations in the Beams</u>

The beam shear determined from the overall moment equilibrium of the subassembly completely defines the bending moment distributions in the beams. So classical conjugate beam method can be applied to calculate the beam rotation at the column face (refer to FIG. 5). Note that the elastic loading is not imposed on the panel zone width due to the infinite flexural rigidity of the panel zone. By applying the conjugate beam method, it can be shown that the beam rotation at the column face θ_b is obtained as



(a) Deformed Configuration



(b) Elastic Loading

FIG. 5 Lateral Deflection Due to Beam Flexural Deformations

$$\theta_b = \frac{\Delta M}{6E(l_L + \ell_R)^2}\left(\frac{(l_L - d_c)^3}{I_{bL}} + \frac{(l_R - d_c)^3}{I_{bR}}\right) \quad (9)$$

The beam rotation in Equation (9) is also the elastic story drift ratio (in radians) of a given subassembly due to the beam flexural deformations. By multiplying θ_b with the distance between the inflection points of the columns, the corresponding lateral deflection δ_r is given as

$$\delta_r = \delta_{rt} + \delta_{rb} = \theta_b\left(\frac{h_t}{2} + \frac{h_b}{2}\right) \quad (10)$$

Using Equations (4), (7), and (9), the total elastic story drift ratio θ_t (in radians) can be written as

$$\theta_t = \theta_c + r_{PZ} + \theta_b \quad (11)$$

5. ILLUSTRATIVE APPLICATION AND COMPARISON

Two structures were selected for the application and validation of the proposed procedure: one is a 6-story office building (Tsai-Popov 1988) and the other is a 13-story office building (Lee and Uang 1995). Lateral load-resisting system of both structures is provided by MRFs on the perimeter. The elevation indicating typical member sizes as well as the story heights of the two structures are shown in FIG. 6 and FIG. 7. The 6-story frame does not have any doubler plates in the panel zones. Doubler plates of varying thickness were used in the interior joints along the height for the 13 story frame(see FIG. 7) and they were considered in determining the stiffness of these joints. All the exterior joints of the 13-story frame were assumed to be rigid because box sections were used for the corner columns.

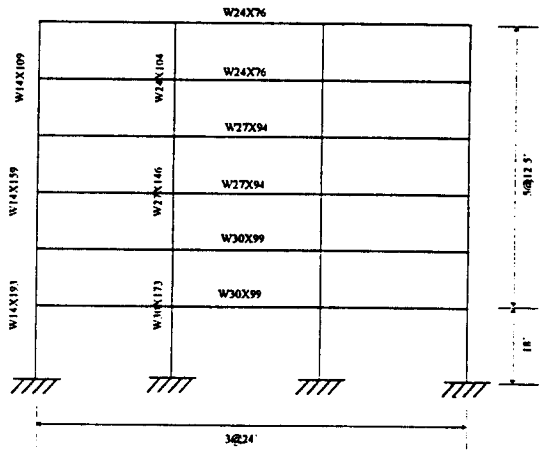


FIG. 6 Member Sizes for a 6-Story MRF (from Tsai-Popov 1988)

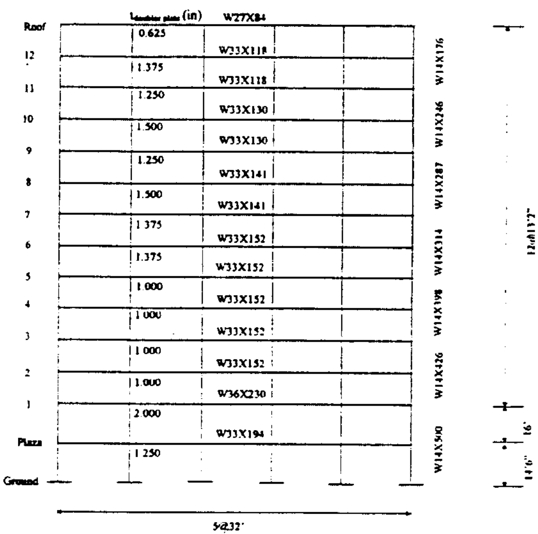


FIG. 7 Member Sizes for a 13-Story MRF (from Lee-Uang 1995)

The computer program DRAIN-2DX (Prakash et al. 1993) was used to obtain analytically precise solutions. One hundred percent rigid-end offsets were used at the end of beams and columns. All the panel zones were modeled with a semi-rigid connection element (i. e., element 4 in DRAIN-2DX). The elastic rotational stiffness of the panel zone k_e was modeled per Krawinkler's recommendation (1978), i. e.,

$$k_e = \frac{\Delta M}{\gamma_{PZ}} = G d_b d_c t \qquad (12)$$

The UBC (1994) equivalent static lateral loading was applied for the analysis. TABLE 2 and TABLE 3 summarize the results for the two structures. The predictions by Tsai-Popov's correction method (1990) were also presented in these tables for comparison purposes. The story drift ratios shown in columns (2) and (3) of these tables are based on the average of the values of two vertically adjacent subassemblies on a typical interior column line. The 5^{th} column in each table contains the errors relative to the analytically precise solution with explicit consideration of the panel zone flexibility. It is noteworthy that the existence of doubler plates in the 13-story frame reduced the contribution of the panel zone flexibility to the total lateral drift. See the 4^{th} columns of TABLE 2 and 3. The average errors from the proposed procedure were 3% underestimation and 6. 5% overestimation for the 6-story frame and 13-story frame, respectively. The errors from the Tsai-Popov's correction method were of similar magnitude. Note that the predictions by the proposed method are based on a simple calculation, using Equation (11), without resort to computer analysis. Considering approximate nature and preliminary purpose of the proposed procedure, the procedure seems to give satisfactory elastic story drift predictions for the regular steel MRFs such as those used in this study. Hence the method can be conveniently used for a quick estimation of the elastic lateral drift of steel MRFs with a reasonable accuracy.

Table 2. Summary of 6-Story MRF Results

	Story Drift Ratio(% rad)			Relative Ratio	
Story No. (1)	This Study (Eq. 11)(2)	Tsai-Popov (1990)(3)	With Semi-Rigid Connection Element(4)	(2)/(4) (5)	(3)/(4) (6)
6	.194	.184	.193 (40%)*	1.00	.95
5	.267	.253	.266 (40%)	1.00	.95
4	.287	.264	.273 (41%)	1.05	.97
3	.299	.280	.293 (40%)	1.02	.96
2	.299	.281	.287 (39%)	1.04	.98
1	.251	.232	.233 (24%)	1.08	1.00

* Percent of the contribution of panel zone shear deformations to the total lateral drift.

Table 3. Summary of 13-Story MRF Results

	Story Drift Ratio(% rad)			Relative Ratio	
Story No. (1)	This Study (Eq. 11)(2)	Tsai-Popov (1990)(3)	With Semi-Rigid Connection Element(4)	(2)/(4) (5)	(3)/(4) (6)
13	.180	.210	.200 (15%)*	.90	1.05
12	.220	.270	.260 (15%)	.85	1.04
11	.270	.300	.280 (14%)	.96	1.07
10	.310	.350	.330 (15%)	.94	1.06
9	.330	.380	.350 (14%)	.90	1.09
8	.350	.400	.390 (18%)	.92	1.03
7	.360	.410	.390 (15%)	.85	1.05
6	.370	.430	.420 (19%)	.95	1.02
5	.380	.420	.400 (18%)	.93	1.05
4	.390	.430	.420 (19%)	.98	1.02
3	.390	.430	.400 (15%)	.89	1.08
2	.310	.370	.350 (17%)	1.00	1.06
1	.250	.270	.250 (12%)	1.08	.108

* Percent of the contribution of panel zone shear deformations to the total lateral drift.

5. SUMMARY AND CONCLUSION

An approximate analytical procedure was proposed in this study to calculate the elastic lateral drift of steel MRFs including the effects of panel zone flexibility. The main feature of the procedure is to combine the classical portal method assumptions and D-value method. In spite of approximate nature of the procedure, it gives satisfactory results as compared with analytically precise solutions; The predictions following the procedure showed 3% to 6. 5% errors on the average for the two case studies undertaken in this study. Considering simple and analytical nature of the procedure, it can be conveniently used for a preliminary and quick estimation of the elastic lateral drift of steel MRFs with a reasonable accuracy. As such, the method can be a useful aid in the preliminary drift design phase. Moreover applying the procedure can convey to the analyst the physical feeling about the deformation response of steel MRFs under the lateral loading.

Acknowledgments

Funding for this research provided by Kyungnam University in 1996 is gratefully acknowledged.

REFERENCES

1. AIJ(1988). *Standard for structural calculations of reinforced concrete structures,* Architectural Institute of Japan

2. Habibullah, A. (1989). "ETABS", *User Manual,* CSI, Berkeley, Calif.

3. Krawinkler, H. (1978). "Shear in beam-column joints in seismic design of steel frames", Engineering Journal, AISC, Third Quarter, pp. 82-91.

4. Lee, C. H. and Uang, C. M. (1995). "Seismic response of haunch repaired steel SMRFs : analytical modeling and a case study, Report No. SAC-95-04, Part 2, pp. 8-1 - 8-49.

5. Muto, K (1974)., *A Seismic Design Analysis of Buildings,* Maruzen, Tokyo.

6. Paulay, T. and Priestley, M. J. N. (1992), *Seismic Design of Reinforced Concrete and Masonry Buildings,* John Wiley & Sons, INC., New York.

7. Prakash, V., Powell, G. H., and Filippou, F. C. (1993), "DRAIN-2DX", *User Guide,* Univ. of California, Berkeley, Calif.

8. Tsai, K. E. and Popov, E. P. (1988). "Steel beam-column joints in seismic steel moment resisting frames", Report No. UCB/EERC-88/19, Univ. of California, Berkeley, Calif.

9. Tsai, K. E. and Popov, E. P. (1990). "Seismic panel zone design effects on elastic story drift in steel frames", Journal of Structural Engineering, ASCE, 116(12), pp. 3285-3301

10. *Uniform Building Code (UBC),* Int. Conf. Of Bldg. Officials, Whittier, Calif., 1994.

NOTATION

The following symbols are used in this paper:

a = degree of restraint provided by the beams connected to the i^{th} column ;

d_b = beam depth ;

d_c = column depth ;

D_i = relative shear stiffness of the i^{th} column;

E = Young's modulus of steel ;

G = shear modulus of steel ;

h_b = story height below the floor under consideration ;

h_t = story height above the floor under consideration ;

H_b = column shear force below the floor under consideration ;

H_i = column shear force in the i^{th} column ;

H_t = column shear force above the floor under consideration ;

I_{bL} = second moment of inertia of the beam section on the left-hand side ;

I_{bR} = second moment of inertia of the beam section on the right-hand side ;

I_{cb} = second moment of inertia of the column section below the floor under consideration ;

I_{ct} = second moment of inertia of the column section above the floor under consideration ;

k_c = relative flexural stiffness of the i^{th} column ;

k_e = elastic rotational stiffness of the panel zone ;

l_L = beam length on the left-hand side ;

l_R = beam length on the right-hand side ;

R = beam shear ;

t = thickness of panel zone including doubler plate, if any ;

V = story shear force ;

V_{panel} = shear force in the panel zone ;

θ_b = story drift ratio due to beam flexural deformations ;

θ_c = story drift ratio due to column flexural deformations ;

θ_t = total story drift ratio ;

δ_c = lateral deflection caused by column flexural deformations ;

δ_{cb} = component of δ_c below the floor under consideration ;

δ_{ct} = component of δ_c above the floor under consideration ;

δ_p = lateral deflection caused by panel zone shear deformation ;

δ_{pb} = component of δ_p below the floor under consideration ;

δ_{pt} = component of δ_p above the floor under consideration ;

δ_r = lateral deflection caused by beam flexural deformations ;

δ_{rb} = component of δ_r below the floor under consideration ;

δ_{rt} = component of δ_r above the floor under consideration ;

γ_{PZ} = shear deformation angle of the panel zone ;

ΔM = unbalanced beam moment.