Simple Analysis for Earthquake Resistant R/C Structures of Moving Beam Plastic Hinging Zones

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Introduction

It has been well documented that a reinforced concrete building frame designed for code seismic forces will be stressed beyond the elastic limit during a major earthquake. Some of the critical regions of the building must be expected to suffer significant inelastic deformation. The critical regions are usually the beam to column connections (Figure 1) .

Because the beam inelastic activity is adjacent to the connection , some stiffness and strength deterioration is anticipated in the connection . In order to aviod or minimize such connection damage , the current recommendations require a high percentage of transverse reinforcement in the column as it passes through the connection . This may lead to steel congestion in the joint and thus , construction difficulties and higher construction costs . Even if these requirements are satisfied, damage cannot be completely avoided .

An alternative approach to solving the beam to column connection problem is to move the beam hinging zone some distance from the column face (Figure 2) (1,3).

Theoritically the joint then will be isolated from inelastic deformation and a reduction in the required joint transverse reinforcement could be anticipated . A previous experimental study was undertaken by Abdel-Fattah and wight (1) to suggest a simple reinforcement scheme for moving a beam plastic hinge away from the column face . An analytical study was undertaken by Al-Haddad and Wight (3) to expand the experimental recommendations for moving a beam hinging zone away from the column face , and to elevate the new design which was proposed by (1) from the laboratory testing level to practical application .

Now, this analytical work is expected to provide the structural engineer with a valualbe information about simple analysis and design for earthquake resistant reinforced concrete buildings of moving beam plastic hinging zones .

Objective and Scope

The main objectives of this study are as follows :

- To study the effect of relocating beam plastic hinging zone on the distribution of internal forces (moment, shear, and axial forces) of the structure .
- To formulate a simple method to obtain the internal forces resulting from gravity load and earthquake excitation after the formation of beam plastic hinging zones .
- The produce a simple analysis and design guide-lines for applying the concept of relocating beam plastic hinging zone one beam depth away from the column face .

To achieve the objectives of this study , fifteen reinforced concrete frames were chosen and analyzed by the normal or conventional method of analysis using finite elemnt technique " ASAS " computer progrman . Thereafter the frames were analyzed again but by assuming the formation of beam plastic hinges at one beam depth away from the column face and applying the analytical model to idealize the beam element whih was suggested by Al-Haddad and Wight (3) and shown in figure 3.

Modifying Design for Moving Beam Plastic Hinging Zones

An indicator for the acting moment ratio (AMR) of the anticipated maximum moment at the moved beam hinging zone , due to lateral inertia force as well as the working gravity load , to that at the column face is defined as follows :

$$AMR = \frac{M_{ed} + M_{dd}}{M_{ef} + M_{df}} \qquad \text{..............................} \qquad (1)$$

where :

M_{ed} and M_{dd} : the calculated moment at the moved beam hinging zone due to equivalent earthquake lateral loads and the working gravity dead load, respectively .

M_{ef} and M_{df} : the calculated moment acting on the beam at the column face due to equivalent earthquake lateral loads and the working gravity dead load, respectively .

Now the desired flexural strength at the moved beam hinging zone can be determined by :

Mud = AMR. Mu see Figure 4 .... (2)

where :

Mud : the required ultimate flexural strength of the moved beam hinging zone.

Mu : the calculated ultimate moment acting on the beam at the column face due to the appropriate factored code loads .

The beam section at the column face should be designed to have flexural strength at least equal to 1.25 times the maximum anticipated acting moment

Muf = 1.25 Mu see Figure 4 .... (3)

The resulting design strength ratio (DSR) between the beam flexural strength at the moved plastic hinging zone and the flexural strength at the column face is determined as follows :

DSR = AMR / 1.25 .... (4)

The column flexural strength usually needs to be checked when designing the beam -to-column connection. The ACI-ASCE Committee 352 (5) specifies that the sum of the column flexural strength at a connection be at least equal to 1.4 that of the beams. A possible extension of such a recommendation for design where the beam critical section under seismic loading is moved and its flexural strength is reduced (in accordance with Eq. 2), is as follows :

$$\sum M_{col.} \geq \frac{1.4 \sum M_{beam} \text{ (at moved hinging zone)}}{AMR} \qquad \text{............. (5)}$$

Study Frames

Fifteen moment resisting building frames were analyzed as mentioned previously twice : (1) before the formation of beam plastic hinges which is referred to as the Conventional Analysis , (2) after the formation of beam plastic hinges at one beam depth away from the column face which is referred to as the New Analysis where the plastic hinge is idealized by a rotational flexural spring .

The studied frames were chosen to follow the effect of every parameter on the analysis as number of frame spans , stories , width of the span, height of the story and cross sectional dimensions of frame elements .

The first frame , which is referred to as Frame 1 , represents an interior frame of two bay (8.0 m spans) with five stories . In this paper , the results and discussion will be concentrated on Frame 1 due to the large size of results for the fifteen frames.

Parametric Analysis and Results

It is intended to study the effect of formation of a beam plastic hinging zone one beam depth away from the column face on the distribution of the internal forces (moment , shear , and axial forces) for the frame elemnts before and after the formation of beam plastic hinges (Conventional and New Analysis) . The two types of loading (gravity load and equivalent earthquake loading) are seperated during study to enable the designer to use the coefficients of the required design criteria .

Floor Beam Results of Frame 1

According to Frame 1 , Figure 5 compares the moment diagram of 3^{rd} floor beams of frame 1 under gravity loading by the Conventional Analysis (solid line) to the New Analysis (dashed line) . This figure indicate that moving the beam hinging zone made a considerable differnce in the moments of the floor beams . The formation of the beam plastic hinges at one beam depth away from the column face allows for some rotation and decreases the ability of the section at the proposed plastic hinge to resist the moment which causes a decrease in the negative moment at both ends of the beam which causes an increase in the positive moment at the middle of the beam because the difference between negative and positive moment is constant . For a structure to be able to resist any of the two stages of forces (before and after relocating the beam plastic hinge) , it may designed to resist the larger of the two forces for every element , so that our attention may be concentrated on the increased forces due to relocating beam plastic hinge .

Figure 6 compares the moment diagram of the same previous floor beams of frame 1 under equivalent earthquake loading for the Conventional Analysis (Solid line) to the New analysis (dashed line) . This figure indicates that moving the beam hinging zone made a slight difference in the moment at the ends of the beam . The little difference in the moment at the ends of beam could be considered out of the designer attention due to its small value and its negligible effect on the positive moment at the middle of beam when superposing the factored moments during design .

Figures 7 - 8 compare the shear diagram of the same previous floor beam under gravity loading and equivalent earthquake loading, respectively , for the Conventional Analysis (solid line) to the New Analysis (dashed line) . Generally , these figures indicate that moving the beam hinging zone made a slight difference in the shear force of the floor beams which could be ignored .

Column Results of Frame 1

Figures 9 - 10 compare the moment diagram of external column of frame 1 under gravity loading and equivalent earthquake loading, respectively , for the Convetional and New Analysis . Figure 9 indicates that moving the beam hinging zone made a decrease in the moment at both ends of each column under gravity loading . Under equivalent earthquake loading , Figure 10 indicate that moving beam hinging zone made a considerable increase in the moment at the base of the lower column which is fixed with the foundation whereas there is a decrease in the moents for other columns. This increase in the base moment of the column may be considered during design.

According to axial forces , there is no considerable change in the distribution of axial forces on columns due to moving the beam hinging zone .

Comparative Study and Discussion

Studying the effect of moving beam plastic hinging zones away from the column face on the internal force is the main objective of this study .

The only two forces which have a considerable increase due to moving beam plastic hinging zones are the positive moment of beams under gravity loads and the base moment of columns under equivalent earthquake loading .

To achieve the second objective of this investigation , a simple and practical approximate formulae are derived through this investigation . Due to the complexity of analysis using the New analysis to obtain the internal froces after relocating the beam plastic hinging zones , which uses a rotational spring to represent the beam plastic hinge and due to the extra effort which may need to perform analysis , these approximate formulae were derived and listed in this investigation .

Positive Moment of Beams Under Gravity Loading

Formula 5 and others listed after were derived by a statistical method using the result of the study frames .

The following emperical formula (6) can give the percentage of increase in beam positive moment due to moving beam plastic hinging zone for external beams through a continuous floor beams in a frame with more than one bay .

$$Y_{ext.} = \frac{X - 56.5}{805.5} \times 100\,\% \qquad \text{............ (6)}$$

$$X = \frac{(L^{-})^{2.1}}{(b)^{0.25}\,(d)^{1.3}} \qquad \text{............ (7)}$$

If the value of X is less than or equal to 56.5 then the value of $Y_{ext.}$ must be taken equal to zero .

where :

$Y_{ext.}$: the percentage of increase in the positive moment for external beam in a beam-column frame of more than one bay .

L^{-} : Clear span of the beam in meters .

b and d : width and effective depth of the beam section in meters , respectively.

For internal beams of multi-bay frame , the percentage of increase in the positive moment under gravity loads is given by the following formaula (8) .

$$Y_{int} = \left(\frac{X - 56.5}{805.5} + 0.115 \right) x\ 100\ \% \qquad \text{.............} (8)$$

$$X = \frac{(L^{\cdot})^{2.1}}{(b)^{0.25}\ (d)^{1.3}} \qquad \text{..............................} (7)$$

The value of $Y_{int.}$ must be taken greater than or equal to zero always .

$Y_{int.}$: The percentage of increase in the positive moment of internal beam in a beam column frame of multi bays.

L^{-} , b and d are as before for external beam .

According to beams in a frame of one span, the increase in the positive moment of beams due to relocating beam plastic hinge is very small and it could be taken not more than 5 % .

Table 1 shows a comparison between approximate New Analysis and New Analysis . The result of comparison indicates that using the approximate New Analysis gives a high accuracy within the range of 1.5 % difference for most of floor beams except the roof beams . For external roof floor beams , Formula (6) gives a high estimate of the positive moment which could be corrected as follows :

$$Y_{roof.} = 0.6 * Y_{ext.} \qquad \text{..................} (9)$$

where :

$Y_{roof.}$: The percentage of increase in the positive moment for external roof floor beams in a beam-column frame of more than one bay .

$Y_{ext.}$: Same as in formula (6).

Column Base Moment Under Equivalent Earhtquake Loading

Due to formation of beam plastic hinging zones at one beam depth away from the column fcae , the column base moment of only the lowest column increses under equivalent earthquake loading . To supply the designer by a simple formula which may be considered as a good indication and it may give acceptable results . This formula was derived by a statistical method .

For external columns :

$$Y_{col.} = \frac{X - 2.2}{140} \times 100\ \% \qquad \text{............ (10)}$$

$$X = A \times B \times L \qquad \text{............. (11)}$$

$$A = \frac{(N_s)^{0.5}}{(N_b)^{0.65}} \qquad \text{............. (12)}$$

$$B = \sqrt{(L/I)_b / (L/I)_{col.}} \qquad \text{............. (13)}$$

where :

$Y_{col.}$: The percentage of increase in the column base moment under equivalent earthquake loading

N_s : Number of stories .

N_b : Number of bays for the frame .

L : Length of beam span beside the indicated column .

$(L/I)_b$: Length over the moment of inertia for the beam

$(L/I)_{col.}$: Length over the moment of inertia for the column .

For internal columns :

$$Y_{col.} = \frac{X - 2.2}{165} \times 100\ \% \qquad \text{.....................(14)}$$

$$X = A \times B \times L \qquad \text{.................... (11)}$$

$$A = \frac{(N_s)^{0.5}}{(N_b)^{0.65}} \qquad \text{......................(12)}$$

$$B = \sqrt{(L/I)_b / (L/I)_{col.}} \qquad \text{......................(13)}$$

where :

$Y_{col.}$: N_S , N_b are same as before for external column .

L : Average length of the left and right beam spans beside the indicated column .

$(L/I)_b$: Same as before but taking the average for both the left and right floor beams .

$(L/I)_{col.}$: Same as before for external column .

These emperical formulae give agood accuracy within the range of 2% for most of frames studied through this investigation .

Design guideline for Moving Beam Plastic Hinging Zone

It is assumed that the designer is familiar with the accepted Conventional design philosophy of strong column -weak beam approach for earthquake moment resisting frame buildings . The design guidelines given here are intended to maintain the strong column-weak beam design philosophy , but the potential beam plastic hinging zone will be relocated approximately one beam depth a way from the column face . The principal procedure of the given design guideline is to reasonably increase the beam flexural strength at column face while the beam flexural strength at the moved hinging zone is reasonably reduced (Figure 4) .

In order to minimize the design efforts , the following steps are recommended :

1. The first effort should focus on designing and detailing the beam section at the column face . During the design of the building , the designer should design a beam section with a nominal flexural strength equal to at least 1.25 times the maximum anticipated acting moment at the column face . Four intermediate depth longitudinal bars with a total area not greater than 0.35 times the total area of the section reinforcement should be used (Figure 4) . It is recommended that a large number of small diameter bars be used , rather than

a small number of large diameter bars , especially for a short span beam. This will make the coordination between the design of this section and that at the moved hinging zone much easier . Also, using a small diameter bar will tend to increase the length of the plastic hinging zone . The maximum positive moment at the middle of the beam may be increased by the percentage value obtained from formulae 6-9 to have a conservative design .

2. Following the Conventional design , column sections sould be designed simultaneously with the design of the beam sections . Satisfaction of the Conventionally required column to beam flexural stength ratio (1.4) should be checked but the expansion suggested in Eq. 5 sould be applied .

3. The intermediate depth beam bars used at the column face should be cut at one and one-half beam depths away from the column face (1) and some of the top and bottom steel should be cut approximately one beam depth away from the column face (Figure 4) . Assuming that the intermediate longitudinal bars are not effective at one beam depth away from the column face. The following criteria is used to determine how much top and bottom steel should be cut, if any :

    a) The nominal flexural strength of the beam section at one beam depth away from the column face should be approximately equal to the maximum anticipated acting moment at this location .

    b) The ratio between the beam flexural capacity at one beam depth away from column face to the increased flexural capacity at the column face should equal to the (DSR) which is mentioned in Eq. 4.

4. Flexural moment at the column bases must be increased before design by the value obtained from the formulae 10 and 14 and a special attention must be given when designing the column bases to provide a good confinement to this location .

References :

1. Abdel-Fattah , B. A., and Wight, J. K., "Experimental Study of Moving Beam Plastic Hinging Zones for Earthquake Resistant Design of R/C Buildings." Report No. UMCE 85-11, Department of Civil Engineering, The University of Michigan, Dec. 1985 , p.p. 291 .

2. ACI Comittee 318, " Building Code Requirements for Reinforced Concrete ACI. 318-83" , American Concrete Insitute, Detroit , 1983, P.P. 155.

3. Al-Haddad, M. S., and Wight, J. K. "Feasibility and Consequences of Mocing Beam Plastic Hinging Zones for Earthquake Resistant Design of R/C Buildings." Department of Civil Engineering Report No. UMCE 86-1 University of Michgan, July, 1986 .

4. ASAS " User Manual, Version HO9" January 1987.

5. " Recomendations for the design of Beam-Column joints in Monolithic Reinforced Concrete Structures " report from ASCE-ACI Comittee 352, Journal of the American Concrete Institute, V. 82, No. 3 , May -June 1985 .

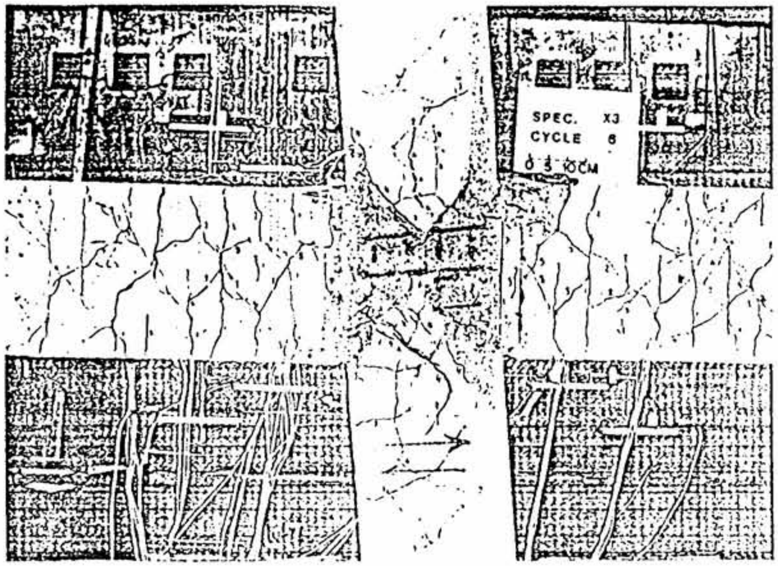

Figure 1 : Formation of plastic hinge in the connection (1)

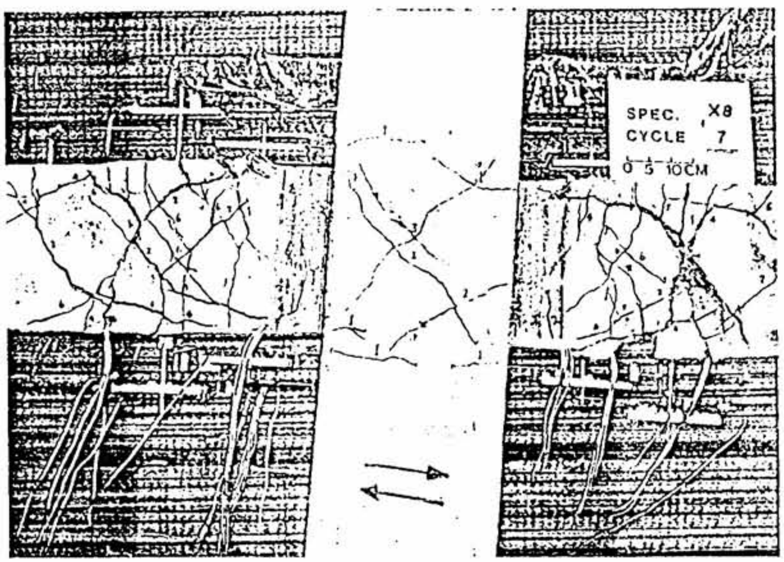

Figure 2 : Formation of plastic hinge away from the connection (1).

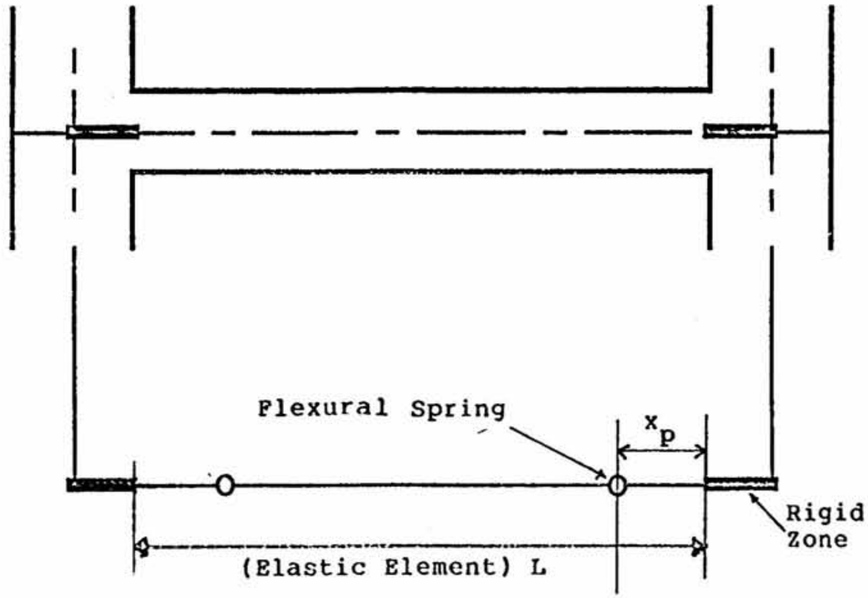

Figure 3 : Idealization of beam element for New Analysis (3).

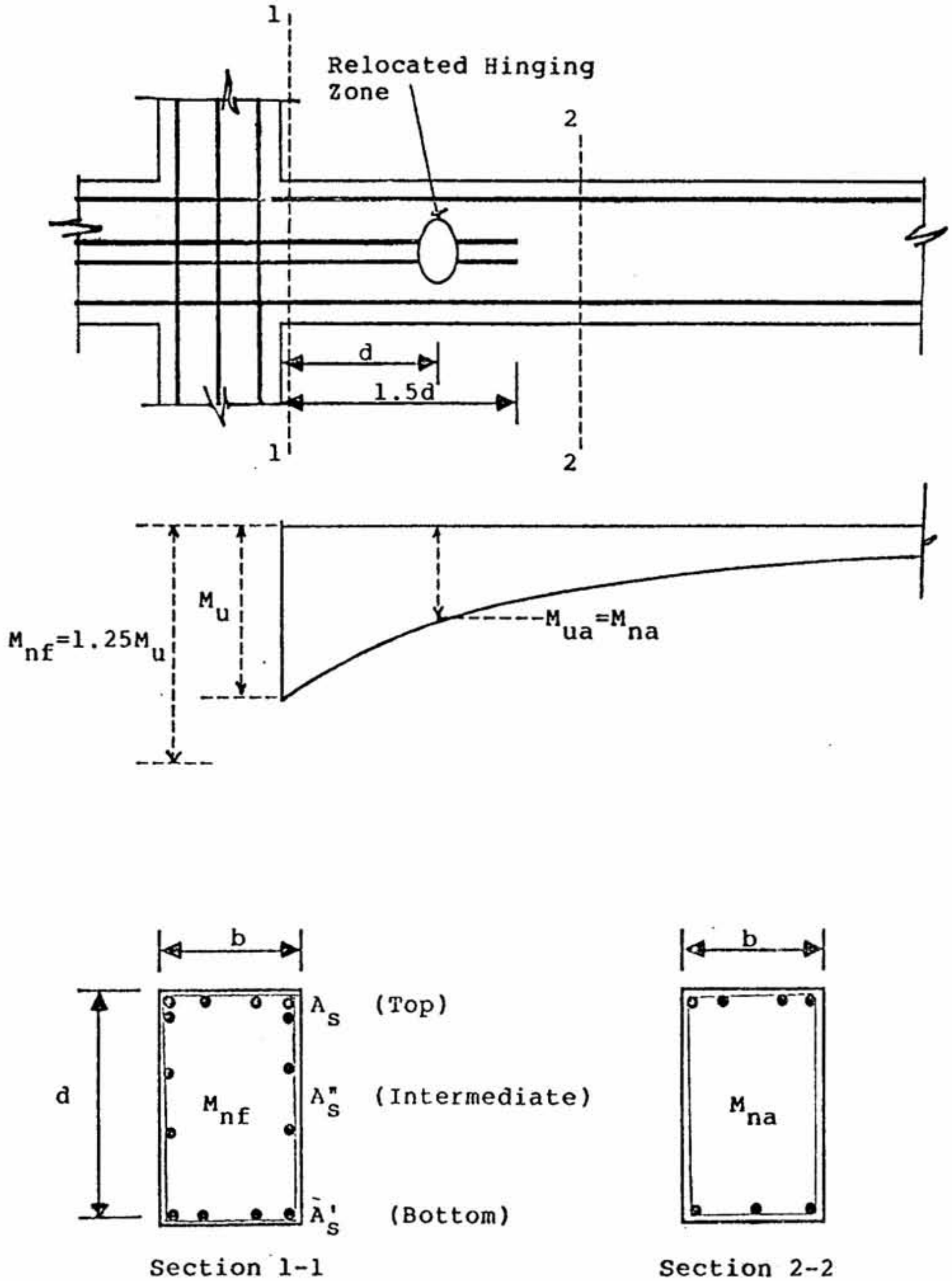

Figure 4 : Typical modified design beam (3).

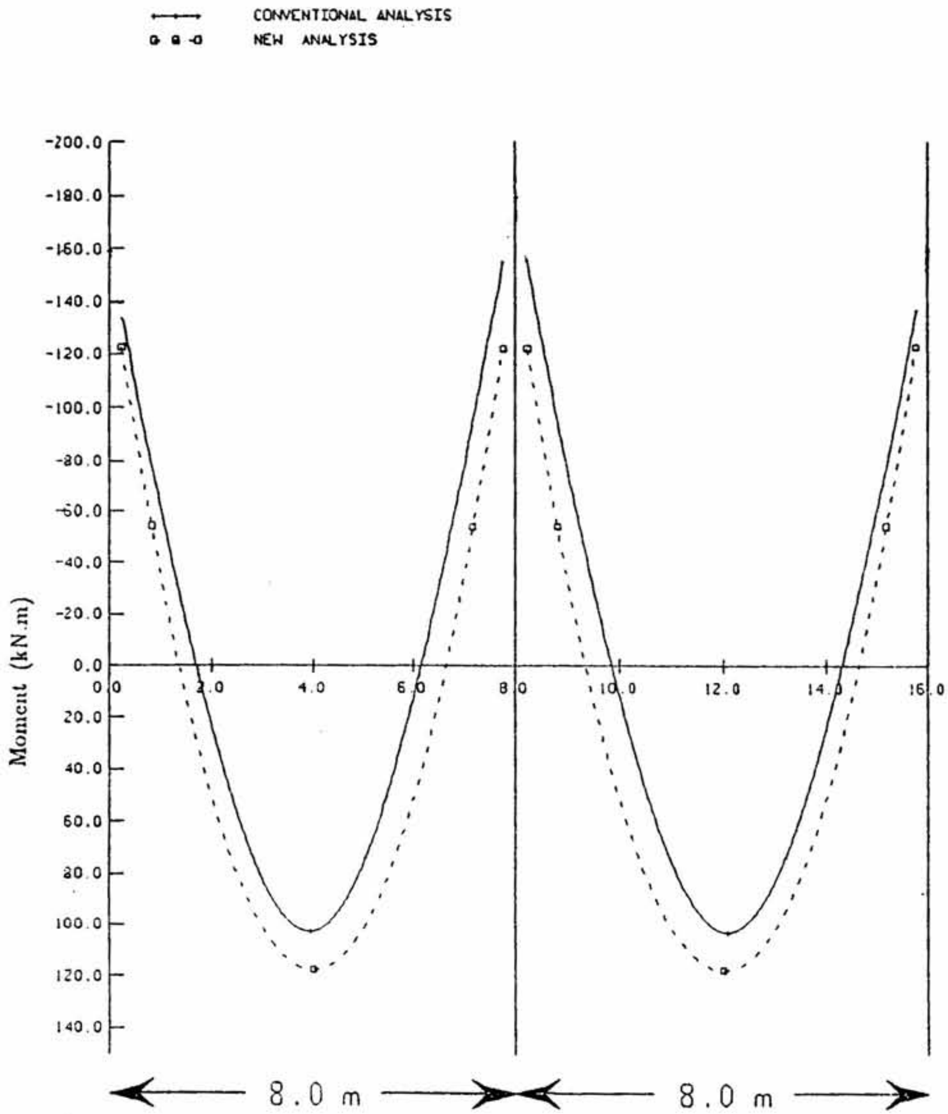

Figure 5 : Moment diagram for 3rd floor beams of Frame 1 under gravity dead load.

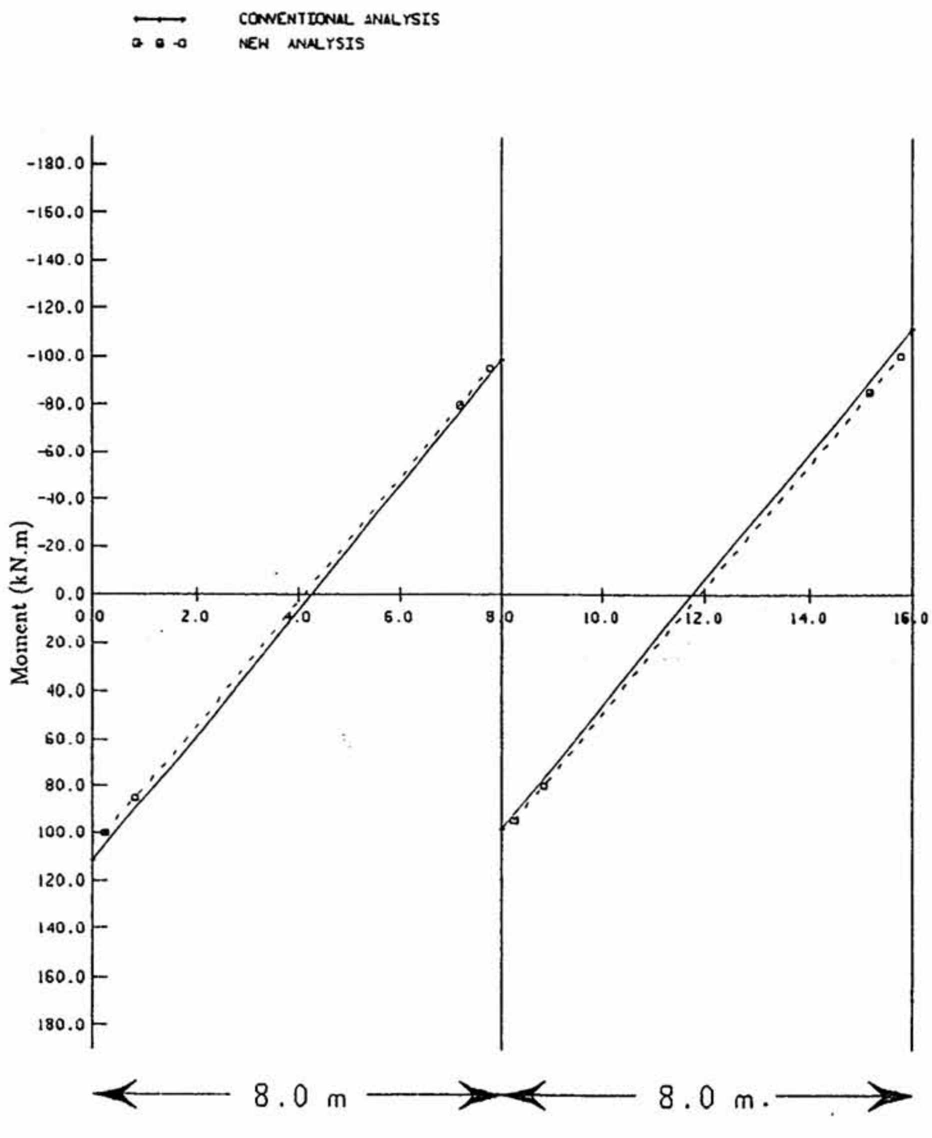

Figure 6 : Moment diagram for 3rd floor beams of Frame 1 under equivalent earthquake loading.

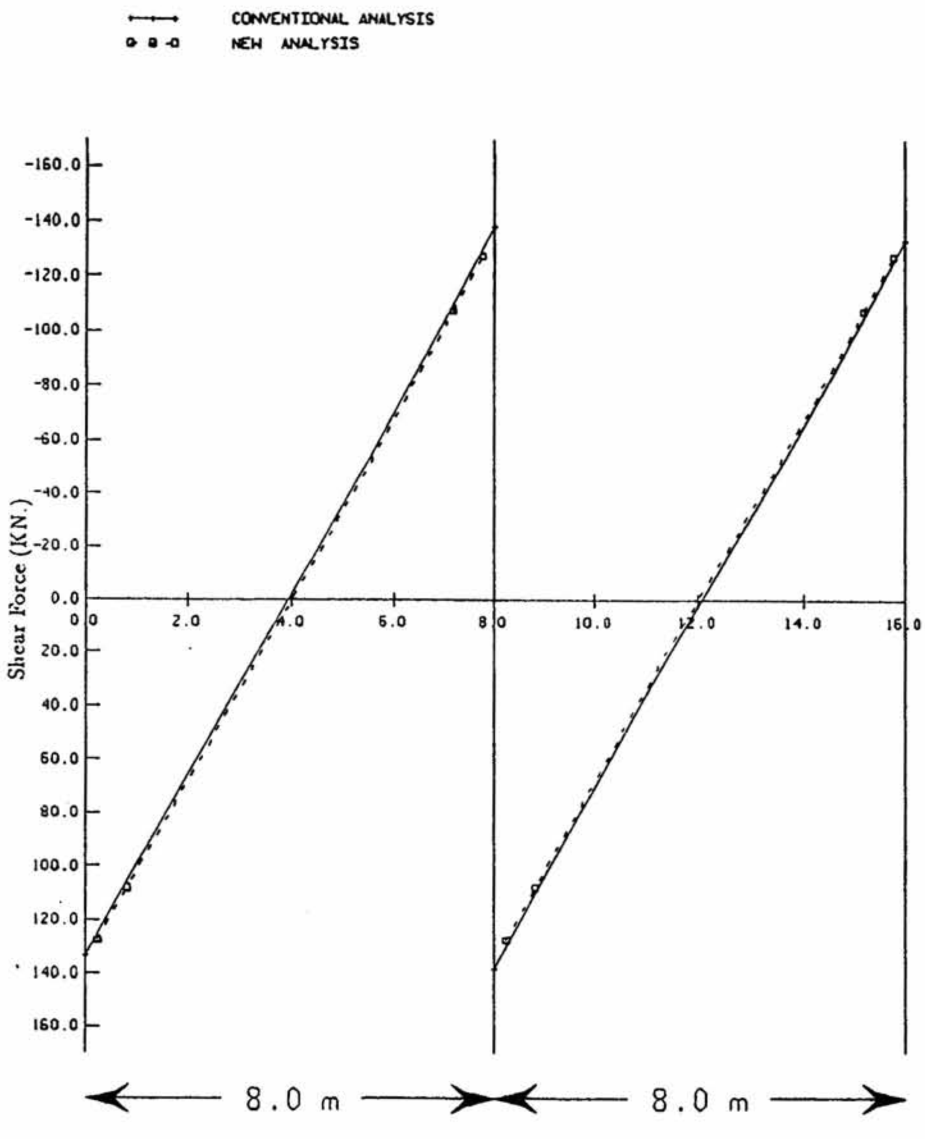

Figure 7 : Shear diagram for 3rd floor beams of Frame 1 under gravity dead load.

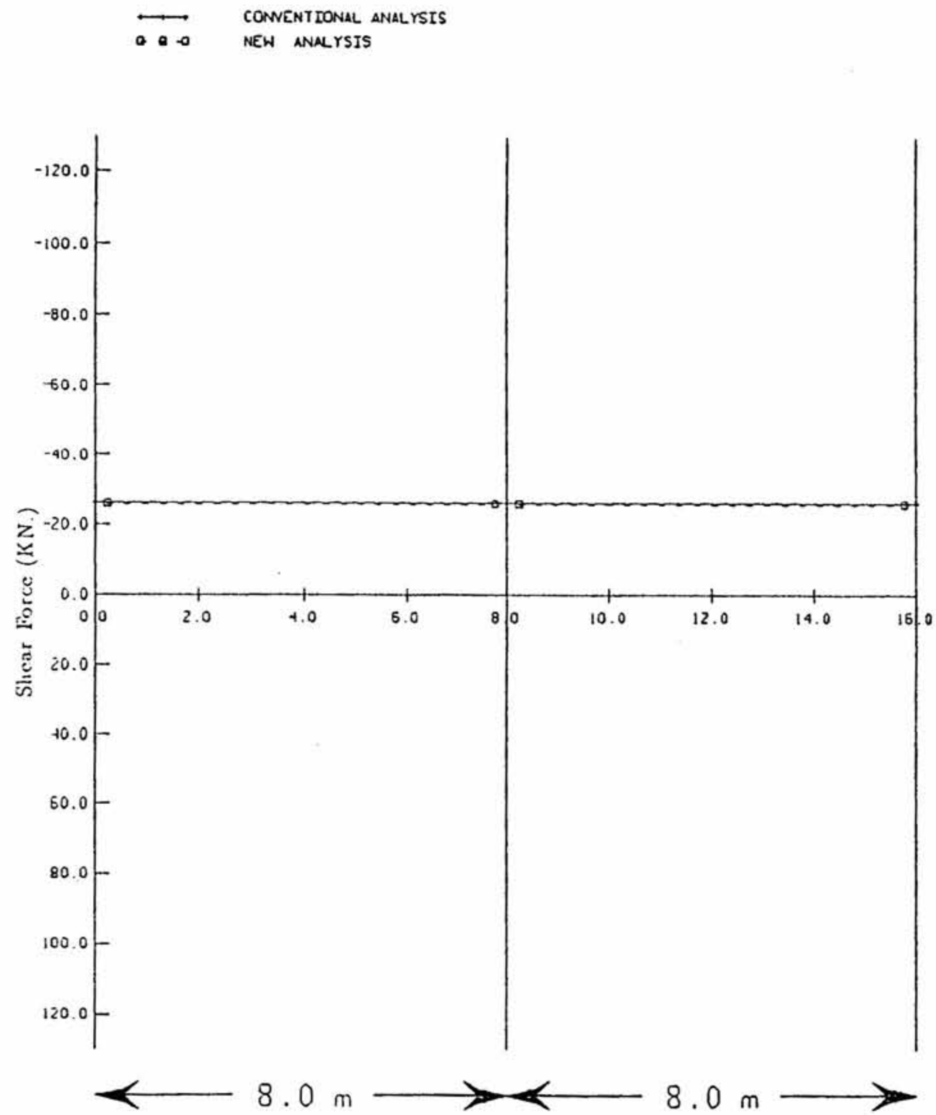

Figure 8 : Shear diagram for 3rd floor beams of Frame 1 under equivalent earthquake loading.

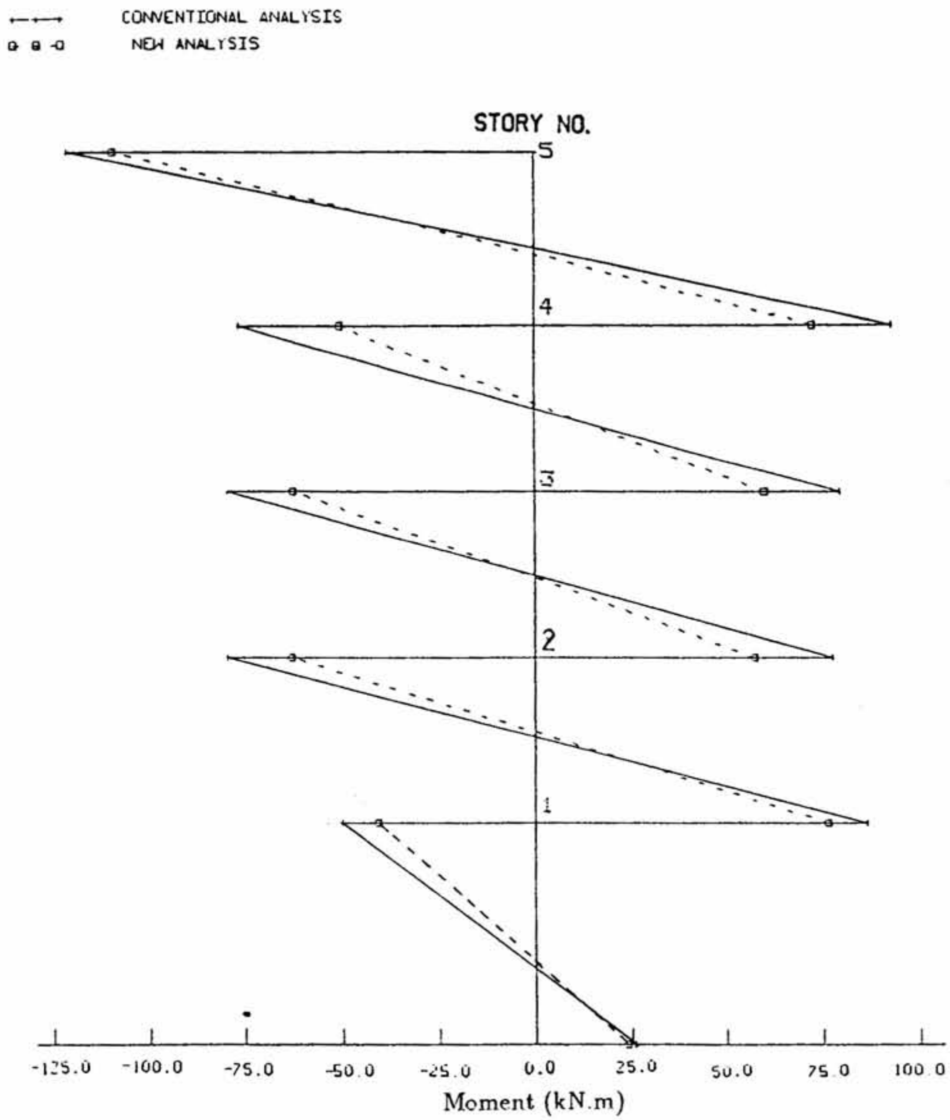

Figure 9 : Moment diagram for external columns of Frame 1 under gravity dead load.

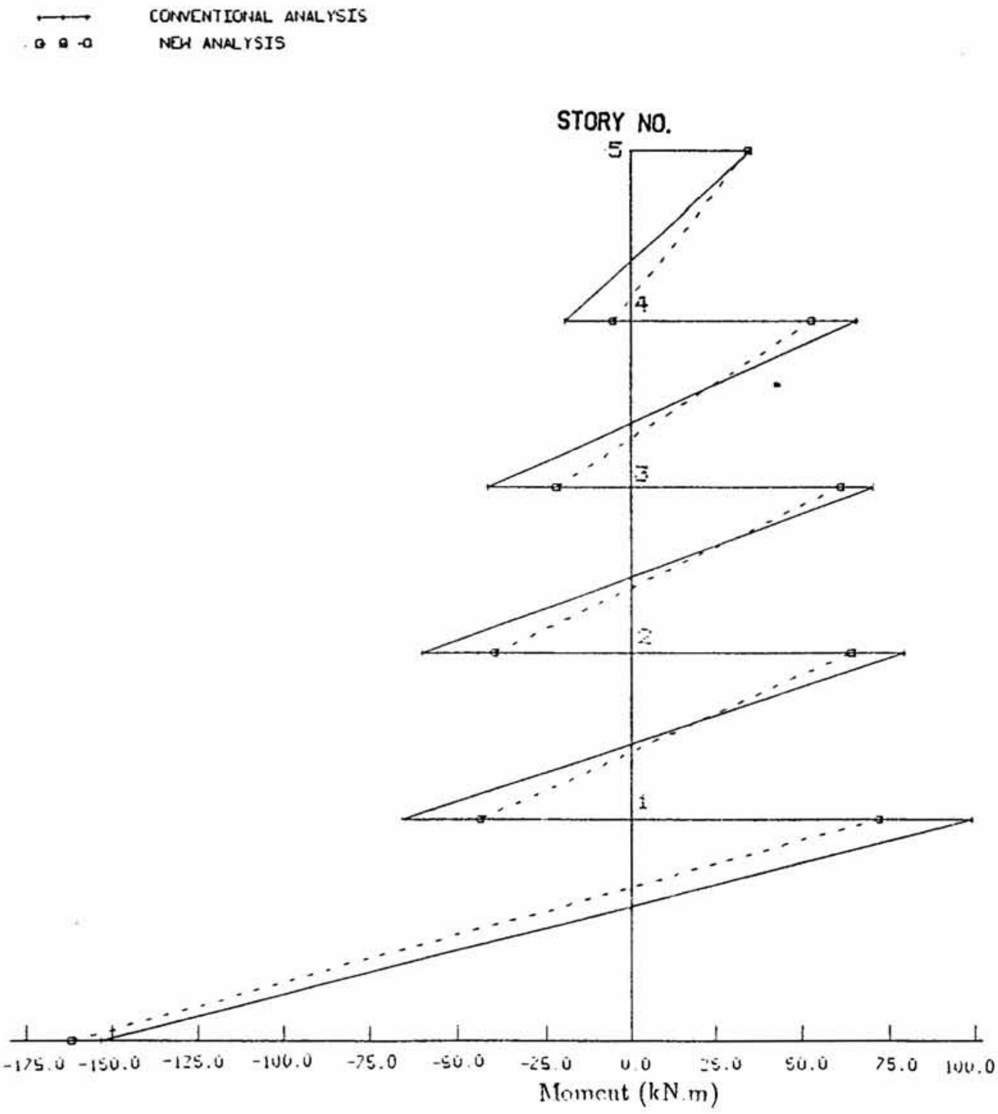

Figure 10 : Moment diagram for external columns of Frame 1 under equivalent earthquake loading.

TABLE 1 : Maximum Positive moment for beams of Frame 1 under gravity dead load using both approaches (Approximate New Analysis and New Analysis) in kN.m.

Story No.	External beam		
	Approximate	New	$\frac{\text{Approx.}}{\text{New}}$
1	120.62	119.95	1.0058
2	116.22	116.66	0.9962
3	117.15	117.85	0.9941
4	115.31	115.70	0.9966
5	131.12	120.99	1.0837