Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.IV)

6-6-9

RELOCATING BEAM PLASTIC HINGING ZONES FOR IMPROVED EARTHQUAKE RESISTANT OF REINFORCED CONCRETE BUILDINGS

James WIGHT,[1] B.A. ABDEL-FATTAH[2] and M.S. AL-HADDAD[3]

[1]Dept. of Civil Eng., The University of Michigan, Ann Arbor, MI, USA
[2]Dept. of Civil Eng., University of Jordan, Amman, Jordan
[3]Dept. of Civil Eng., King Saud University, Riyad, Saudi Arabia

SUMMARY

Results are presented from a combined experimental and analytical study of relocating beam plastic hinging zones away from column faces in reinforced concrete structures. The experimental study concentrated on simple modifications to standard reinforcing schemes and attempted to define key design parameters. The analytical portion included a parametric study of changes in dynamic response due to different beam spans and varying flexural strengths in the relocated beam plastic hinges.

INTRODUCTION

The strong column-weak beam design philosophy for earthquake resistant design of reinforced concrete frame buildings usually leads to formation of plastic hinges in the beam regions adjacent to the column face. Because the beam inelastic activity is adjacent to the connection, it is likely that some stiffness and strength deterioration will penetrate into the connection. In order to avoid or minimize such damage, current design recommendations (Refs. 1,2,3) 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.

An alternative approach for solving the beam to column connection problem is to relocate the beam plastic hinging zone some distance from the column face. Theoretically the joint will then be isolated from severe inelastic deformation and a reduction in the joint transverse reinforcement could be anticipated. In order to formulate definite conclusions and recommendations for this kind of design, a systematic experimental and analytical study was undertaken.

EXPERIMENTAL INVESTIGATION

The primary objective of the experimental study was to find a simple reinforcing scheme which would relocate the potential beam plastic hinging zone approximately one beam depth away from the column face. To accomplish this objective twelve beam-column subassemblies were constructed and tested. Eight of the specimens consisted of the beams and columns only while the other four test specimens had a slab and transverse stub beams to more closely approximate a real beam-column subassembly.

The test specimens essentially represented a column between inflection points above and below a floor level and a beam between inflection points on each side of the column, Fig. 1. The test specimen was loaded laterally at the top of the column and held by link type supports at each end of the beam. The base of the column simulated a pin connection. The applied lateral displacements were reversed during each load cycle and the magnitude of the displacements was increased in each successive cycle.

The use of extra top and bottom steel, which extended one beam depth away from the column face, combined with intermediate depth longitudinal bars, which extended over two beam depths, was found to be the most effective and easy to design reinforcing scheme for relocating the beam hinging zone, Fig. 2. The intermediate layers of longitudinal steel shown in Fig. 2 were added to improve the cyclic shear strength of the plastic hinging zone. The successful use of such bars has been reported previously (Refs. 4,5).

Table 1 lists all the test specimens and gives some of the key design parameters. The specimen name gives the following three pieces of information. The first letter gives an indication of the amount of inelastic flexural deformation anticipated in the beam at the column face; that is, M = none, hinging zone completely relocated, m = some, a spreading rather than a complete relocation of the hinging zone, and C = substantial, standard beam design. The second letter indicates the presence (S) or absense (X) of a floor slab in the test specimen. The third position indicates the number of the specimen within a particular series. Other parameters reported in Table 1 are the ratio between the sum of the moment capacities of the columns to that of the beams at the connection, M_r, the level of expected shear stress in the joint (based on gross column area), the percentage of transverse reinforcement in the joint, the ratio of intermediate longitudinal steel area, A_i, to that of the top steel, A_s, and the shear span to depth ratio for the beam.

Table 1 also gives a short summary of the behavior of each specimen. In almost all of the type m and M specimens the beam plastic hinging zone was initially moved from the column face, but subsequent problems developed in several specimens. For all of these specimens, relocating the beam plastic hinging zone lead to higher shear forces in the beams, columns and the beam-column joint. For some of the test specimens the increased shear forces in the joint caused severe diagonal cracking and eventual shear failure of the joint.

Relocating the beam plastic hinging zone also lead to higher rotational ductility demands in the beam plastic hinge. For some of the M specimens the relative moment capacity of the beam section at the proposed plastic hinge location was made less than half of that at the column face. For these specimens the hinging zone did not spread toward the joint and damage concentrated over a short segment of the beam. Because of the increased rotation demands and the higher shear forces mentioned above, these narrow plastic hinging zones quickly deteriorated and did not dissipate an adequate amount of energy.

For the successful test specimens, that is, specimens in which the beam plastic hinging zone was relocated and the subassembly still exhibited stable hysteretic behavior, the following criteria were adherred to: 1) the beam-column joint shear stresses were kept below recommended values from ACI Committee 352 (Ref. 1), 2) beam shears were kept below 2.0 MPa, and 3) the ratio of beam flexure strength at the hinge to that at the column face was approximately 0.6. A more complete description of this experimental study is given in Ref. 6.

Table 1 Specimen Design Parameters and Summary of Experimental Results

Specimen	Moment Ratio, M_r	Joint Shear Stress (MPa)	Joint Trans. Reinf. Ratio, (%)	$\frac{A_i}{A_s}$	$\frac{a}{d}$	Yielding at Column Face	Total Energy Dissip. / First Cycle Energy
CX1	1.62	7.8	0.95	0	2.6	Substantial	19
mX1	1.60	7.8	0.95	0.81	2.6	Substantial	28
mX2	1.60	7.8	0.61	0.81	2.6	Substantial	22
MX1	1.60	7.8	0.95	0.58	2.6	Minor	18
MX2	1.60	7.8	0.61	0.58	2.6	Substantial	5
MX3	2.60	6.1	1.18	0.30	3.3	None	41
MX4	2.60	6.1	0.76	0.30	3.3	None	24
MX5	3.50	4.5	1.18	0.32	3.3	None	30
CS1	1.40	8.7	0.95	0	2.6	Minor	14
mS1	1.50	8.3	0.95	0.73	2.6	Substantial	4
MS1	2.17	7.8	1.18	0.22	3.3	None	20
MS2	2.17	7.8	0.76	0.22	3.3	None	19

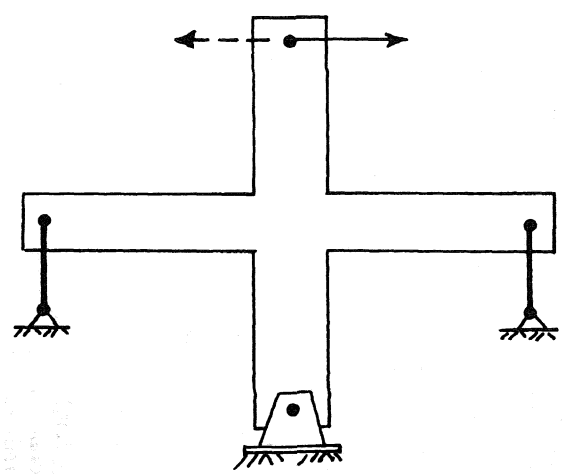

Fig. 1 Idealization of Test Setup

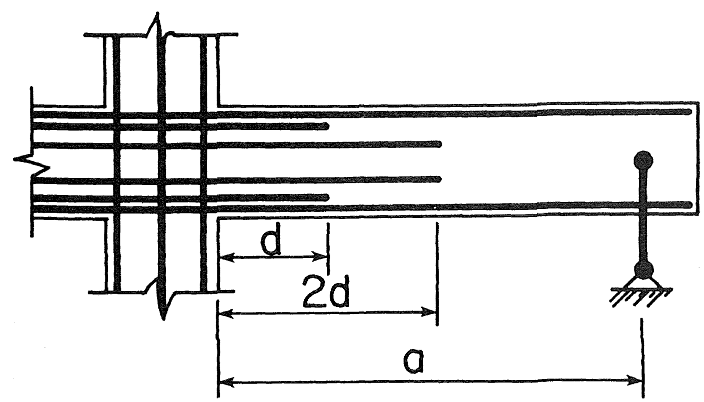

Fig. 2 Proposed Reinforcement Pattern

ANALYTICAL INVESTIGATION

The analytical portion of this study was essential for expanding the experimental recommendations by demonstrating practical applications of this design concept. Three reinforced concrete moment resisting frames were selected for this study. The first was a two bay frame (7.6 m spans) with five stories (3.65 m story height). The second was a two bay frame (4.6 m spans) with five stories as well. The third was a two bay frame (6.1 m spans) with ten stories. Initially they were designed according to the current provisions of the Uniform Building Code (Ref. 3) and the recommendations of ASCE-ACI Committee 352 (Ref. 1). These frames are referred to as the standard design frames, SDF. For each frame, the beam designs were then modified such that during overload conditions beam plastic hinges would form one beam depth away from the column face. The modified designs will be referred to as the modified design frames, MDF.

All analyses were performed using the DRAIN-2D inelastic dynamic analysis computer program. A two component beam-column element was used to simulate column dynamic behavior. This element yields on the basis of a moment-axial load interaction yield surface and assumes a simple bilinear hysteresis model with stable loops for the moment-rotation relationship. A single component beam element, consisting of an elastic line element, two inelastic rotation springs, and two rigid zones, was developed during this study to represent the beams. The location of the springs on the elastic line element can be specified. The moment-rotation relationship of the springs under load reversal was assumed to follow a modified version of Clough's hysteresis model, Fig. 3.

Parametric Analyses and Results Preliminary parametric analyses were conducted using only the first two study frames. The dynamic response of each of these frames for the first seven seconds of the El Centro 1940 NS earthquake was determined for both standard and modified designs. For the modified designs the beams were assumed to have different yield moment capacities at the relocated plastic hinges. Values were selected to be a certain ratio of that used for the standard hinging case. Strength ratio (SR) values used in the preliminary analyses were 1.0, 0.8, 0.7 and 0.6.

Preliminary results, which are summarized in Table 2, indicated that relocating the beam hinging zones away from the column face changed the overall dynamic response of the structures. The degree of change in the response parameters varied widely as the hinging zone was relocated and as the beam yield moment capacity in those hinging zones was changed.

Lateral story displacements were reduced and these reductions were only slightly sensitive to the variation in beam yield moment capacity. Changes in some of the other response parameters were sensitive to the variation in the beam yield moment capacity. For example, moving the beam hinging zone and maintaining a high beam yield moment capacity (SR=0.8) caused a reduction in energy absorbed by the beams and correspondingly caused an increase in member shear forces and column yielding. When the beam yield moment capacity at the relocated hinging zone was lower (SR=0.6), beam rotational ductility demand increased because the beams tended to absorb more of the input energy. In those cases member shear forces as well as column ductility demand were decreased. Based on the preliminary analysis a specific strength ratio (SR) was selected for each modified design frame. The SR values were based primarily on the beam span to depth ratio, using higher SR values for longer spans.

Final Analyses and Results Following the preliminary study, the dynamic response of each standard design frame (SDF) and its modified version (MDF) was determined for two earthquake records, El Centro 1940 NS and TAFT 1953 S21W. For each record the maximum acceleration was normalized to 0.5g. Except for the

Table 2 Results from Preliminary Analyses

Response Parameter	Study Frame 1: Conv. Design	Study Frame 1: Mod. Design, SR 0.8	Study Frame 1: Mod. Design, SR 0.6	Study Frame 2: Conv. Design	Study Frame 2: Mod. Design, SR 0.8	Study Frame 2: Mod. Design, SR 0.6
Roof displ. (in)	7.2	6.8	7.0	7.9	7.1	7.6
Beam plas. rot. (rad. 10^{-3})	13.3	12.8	13.2	11.5	9.7	11.2
Beam rot. ductility	4.0	5.9	6.0	4.5	8.2	9.4
Col. plas. rot.(rad. 10^{-3})	10.5	13.0	6.5	20.4	26.0	16.7
Col. rot. ductility	3.5	3.8	2.5	5.3	6.5	4.5
Beam shear force (kN)	218	245	187	151	200	160
Total base shear (kN)	1340	1420	1170	574	614	556

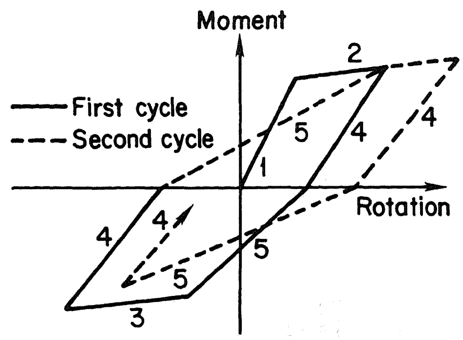

Fig. 3 Hinging Zone Hysteresis Model

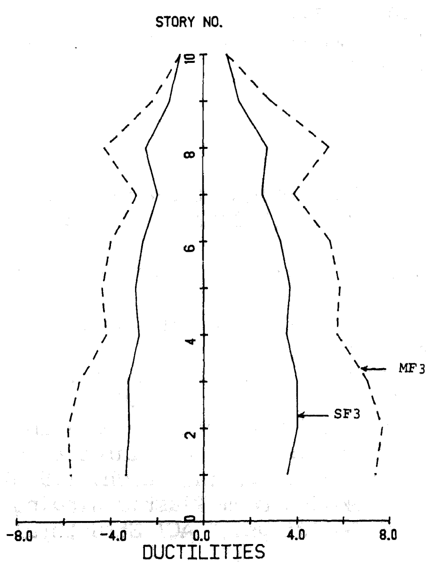

Fig. 4 Beam Ductility Demand

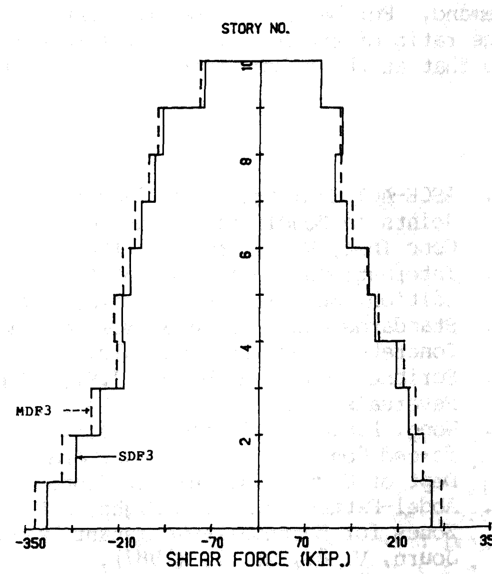

Fig. 5 Story Shear Distribution

effect of the column gravity loads acting through lateral displacements (P-delta effect), the beam moments and shear forces due to dead loads were specified as initial forces in the analyses. For this paper only the dynamic response of the frame SDF3 and its modified version MDF3, due to the modified El Centro record are compared. Results for frames SDF1 and SDF2 and their modified versions, MDF1 and MDF2, were similar for both input motions.

Roof displacement time histories for frames SDF3 and MDF3 were very similar for both earthquake records. Relocating the beam hinging zone made little difference in the peaks and frequency content of the response history. Relocating the beam hinging zone reduced the maximum story displacements an average of 10%. The maximum rotational ductility demand of the beams are presented in Fig. 4. The beam rotational ductility demand, which was the response parameter most sensitive to relocating the beam hinging zone, showed increases as high as 100% in two locations. Fig. 5 shows that only slightly higher story shears were induced in frame MDF3. A more complete description of this analytical investigation is given in Ref. 7.

CONCLUSIONS

Experimentally it was shown that a simple reinforcing detail could be used to move a beam plastic hinging zone away from a column face. This detail is recommended for use in normal span beams which have moderate shear forces acting during seismic overload conditions. The detail was not successful for specimens with low span to depth ratios.

The determination of the required design strength of the beam section at the proposed plastic hinging zone is the most important step in the design process. Two major criteria are to be considered: 1) excessive column yielding and significant increases in member shear forces should be avoided (this is done by reducing the beam flexural capacity at the potential plastic hinging zone), and 2) large reductions in the beam flexural strength at the relocated hinging zone should be avoided because of the resulting increase in beam rotation ductility demand. For beams with normal span to depth ratios the appropriate value for the ratio of the beam flexural strength at the relocated plastic hinge location to that at the column face was between 0.6 and 0.7.

REFERENCES

1. ASCE-ACI Committee 352, "Recommendations for the Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," Journ of the American Conc Inst, V. 82, No. 3, (1985).
2. International Conference of Building Officials, Uniform Building Code, 1985 edition, Whittier, California, (1985).
3. Standards Association of New Zealand, Code of Practice for The Design of Concrete Structures, NZS 3101, Wellington, (1982).
4. Scribner, C.F. and Wight, J.K., "Strength Decay in R/C Beams Under Load Reversals," ASCE Journ of the Struct Div, V. 106, No. ST4, (1980).
5. Wong, P.K.C., Priestley, M.J.N. and Park, R., "Seismic Behavior of Reinforced Concrete Frames Incorporating Beams with Distributed Reinforcement," Dept of Civil Eng, Univ of Canterbury, Res Rpt 85-6, Christchurch, (1985).
6. Abdel-Fattah, B.A. and Wight, J.K.,"Study of Moving Beam Plastic Hinging Zones for Earthquake-Resistant Design of R/C Buildings.," ACI Structural Journ, V. 84, No. 1, (1987).
7. Al-Haddad, M.S. and Wight, J.K., "Relocating Beam Plastic Hinging Zones For Earthquake Resistant Design of R/C Buildings," ACI Structural Journal, V. 85, No. 2, (1988).