Steel Moment Frames With Ductile Girder Web Opening

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ABSTRACT

An innovative way to improve the performance of moment-resisting steel frames is proposed. The upgrading scheme consists of creating rectangular (Vierendeel) openings reinforced with diagonal members in the web of girders near the mid span. Under severe seismic forces, inelastic activity will be limited only to yielding and buckling of diagonal web members and plastic hinging in T-shaped chord members of the openings. Through this scheme, not only is premature fracture of beam-to-column connections is avoided, but widespread and uncontrolled yielding, as observed in many moment frames, is also prevented. This results in improved dynamic response under severe ground motions. An actual existing moment frame building was taken as an example by applying the proposed modification scheme to the moment frames. The responses of the structure to a severe ground motion before and after modification were compared. Finally, a full scale testing of a one story subassemblage was carried out to verify the proposed modification procedure and to verify the results from computer analyses.

INTRODUCTION

Moment-resisting structural steel frames have long been regarded as one of the best structural systems to resist seismic forces. The performance of such frames under seismic forces depends primarily on the strength and ductility of their beam-to-column joints. Unfortunately, a large number of beam-to-column connection failures were observed in the 1994 Northridge Earthquake and the 1995 Kobe Earthquake. These failures clearly showed that typical beam-to-column moment connections as used in current practice possess far less ductility than expected. Moreover, studies have shown that moment resisting frames designed by elastic method using equivalent static forces undergo inelastic deformations in a rather uncontrolled manner resulting in uneven and widespread formation of plastic hinges.[6,9,10] This unevenly distributed yielding may lead to large story drifts and large rotational demands on beam-to-column connections under seismic forces. Thus, combined lack of ductility of the connections and the use of elastic design method could hold a major key in explaining the recently observed poor performance of steel moment frames.

It is obvious that there is an urgent need for methods to retrofit and upgrade existing moment-resisting steel frames. Two approaches are being employed in current design practice and research studies: (1) a strengthening strategy in which the beam-to-column connection is reinforced to meet the strength and ductility demand, and (2) a weakening strategy where the beam is weakened (away from the connection) in order to create a "fuse" that limit the force demand on the connection. The strengthening strategy also requires checking the adequacy of columns and other critical regions of the frame for increased force demands. For this reason, weakening strategy (such as "dog bone" solution) is becoming increasingly popular.

As part of the weakening strategy, a possible scheme to modify the behavior of moment resisting frames to have ductile yield mechanism is by creating rectangular Vierendeel openings in the girder web near the middle of the span. The shear capacity of the openings can be increased, if needed, by adding diagonal and vertical members into the openings to provide overall stiffness to the frame. The openings are designed such that, under a severe ground motion, the inelastic activity will be confined only to yielding, and buckling of the diagonal members and the plastic hinging of the chords of the opening while other members in the frame remain elastic. The concept and results from preliminary research work are briefly presented in this paper.

CONCEPT AND DESIGN PROCEDURE OF PROPOSED UPGRADING SCHEME

The concept of using openings as ductile segments is derived from a structural system known as Special Truss Moment Resisting Frames (STMF). This structural system has been studied both analytically and experimentally by Goel et al.[2,3,6] at The University of Michigan during the past ten years and

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has been recently accepted by the 1996 UBC Supplement and the next edition of the AISC-LRFD seismic provision.[1,14] The system consists of truss frames with special segments designed to behave inelastically under severe ground motions while the other structural members of the frame remain elastic. The special segments of the STMF can be either Vierendeel openings or Vierendeel openings with X-diagonal members, depending on the desired level of shear strength of the special panels.

When a moment frame with openings or a STMF structure is subjected to lateral forces induced from an earthquake, the shear in the floor girders is resisted by the chord members and the X-diagonals in the openings. After yielding and buckling of the X-diagonals, plastic hinges will form at the ends of the chord members. After the openings in all floor girders have yielded, complete mechanism is achieved when additional plastic hinges form at the base of the columns. Figure 1 shows a moment frame modified with girder web openings and a STMF at ultimate (mechanism) state.

In this proposed system, the chord, diagonal and vertical members are designed such that, under their fully yielded and strain-hardened condition at about 3 percent of story drift, the moment at the beam-column connections generated by the shear force in the opening will be smaller than their flexural yield strength, thus, reducing the risk of premature failure of those connections and confining all inelastic activity only to the openings. The maximum shear strength of an opening can be estimated by multiplying the nominal strength by a factor called "overstrength factor". This factor takes into account the increase in strength due to strain hardening and the difference in the nominal and actual yield strength.

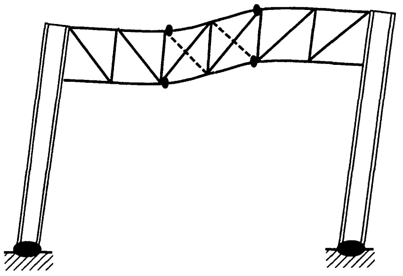
The design of an opening begins by calculating the maximum allowable shear force in the opening such so the moment created by the fully strain hardened shear force in the opening at the connection is approximately equal or less than the yield moment (or some acceptable strength level) of the connection. Generally, moment frames are exterior frames, therefore, the effect of gravity loads is small as compared to that of the lateral loads and is, therefore, neglected in the following design procedure. Figure 2 shows internal forces in a frame with opening due to lateral loads only. From the simplified moment and shear force diagrams shown in Figure 2a, assuming that the opening is placed at the mid span and neglecting the moment due to the vertical member and axial forces in the chord members, the maximum permissible shear force in the opening can be conservatively taken as (using center line dimensions):

$$V_o \leq \frac{2\phi M_y}{L} \tag{1}$$

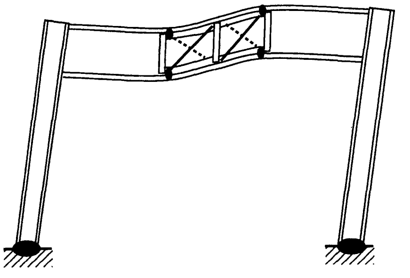
where

V_o = maximum permissible shear force in the opening
M_y = yield moment of the connection
ϕ = resistance factor = 0.90
L = span length

The shear strength of an opening consists of the shear force contributed by the chords, and the diagonal members. By

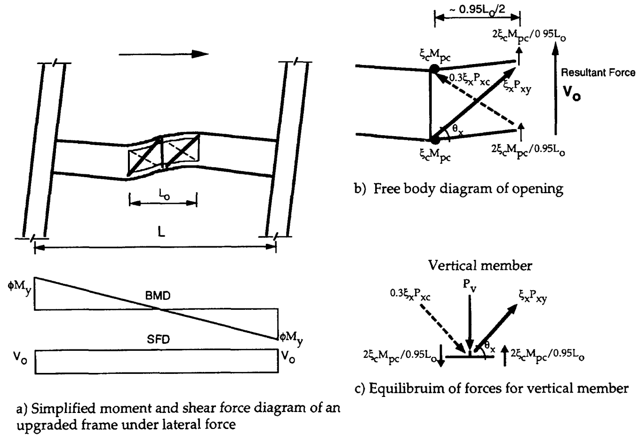


a) Special Truss Moment Frame



b) Moment Frame with Ductile Opening

Fig. 1. Moment frame modified with ductile opening and STMF at mechanism state.



a) Simplified moment and shear force diagram of an upgraded frame under lateral force

b) Free body diagram of opening

c) Equilibruim of forces for vertical member

Fig. 2. Internal forces in a frame modified with the proposed upgrading scheme.

assuming that the points of inflection of the chord members are at the middle of the opening, the vertical resultant force, V_o in the opening can be expressed as (From Figure 2b):

$$V_0 = \frac{4(\xi_c M_{pc})}{(0.95L_o)} + \xi_x(P_{xy} + 0.3P_{xc})\sin(\theta_x) \tag{2}$$

where

ξ_c, ξ_x = overstrength factors for the chord and the diagonal members, respectively
M_{pc} = plastic moment of the T-section chord member
L_o = length of the opening
P_{xy} = yield force of the diagonal member
P_{xc} = buckling force of the diagonal member
θ_x = angle between the diagonal and the chord member

In Equation 2, the first term represents the contribution from the chord members and the second term represents the contribution from the diagonal members. The factor 0.95 in the first term accounts for the rigid zone at the end of the chords and the factor 0.3 in the second term accounts for the post-buckling strength of the diagonal members. These factors have been validated with reduced scale tests of beams with openings conducted in earlier phase of this research. The overstrength factor for the diagonals, ξ_x, has been found to be approximately 1.4 in most of the experiments. Therefore, the value of 1.4 is suggested for design purpose. The overstrength factor for the chords, on the other hand, is a function of both the length of the opening and the section properties of the opening's chord members. Therefore, the design of the chords involves trial and error process and is discussed in the following section.

Design of Chord Members

The expression for overstrength factor was first proposed in the study of Special Truss Moment Resisting Frames by Basha[2] which was later modified for use with WF beams with opening by correlating the equation with the results from the small scale tests. The overstrength factor for the chords of WF beam with opening, ξ_c, can be expressed as:

$$\xi_c = \frac{0.02EI_c\left(\dfrac{L - 0.95L_o}{L_o^2}\right) + 0.90M_{pc}}{M_{pc}} \tag{3}$$

where

E = Young's Modulus of steel
I_c = moment of inertia of the chord member
M_{pc} = plastic moment of the T-section chord member
L_o = length of the opening

The overstrength factor is directly related to the rotational demand at the plastic hinges in the chords—larger the rotational demand on the chord members larger the overstrength factor. Therefore, in order to limit the rotation in the chords such that the chords would not be too severely damaged during major earthquakes, the overstrength factor with a value between 1.8–2.0 is desirable and has been found by experiments to be in a practical range. As mentioned earlier, the overstrength factor of the chords is a function of both the length of the opening and the section properties of the opening's chord members. The process of designing the openings is based primarily on trial and error approach to converge on a reasonable value of the overstrength factor.

The design begins by determining the length of the opening. The length of the opening in the order of about 0.20 to 0.25 of the span length has been found to perform well and provide a good combination between stiffness and strength of the frame. After the length of the opening has been selected, since I_c, M_{pc} can be expressed in terms of the depth of the chord, the chord of the opening can be designed by varying the depth of the chord until the overstrength factor calculated by Equation 3 comes out in target range of 1.8–2.0.

Design of Diagonal Bars

With known depth of the chord members, the shear contribution of the chords can be determined, and consequently, the size of the diagonal bars. From Equation 2:

$$1.4(P_{xy} + 0.3P_{xc})\sin(\theta_x) = V_o - \frac{4(\xi_c M_{pc})}{(0.95L_o)} \tag{4}$$

where P_{xy} and P_{xc} can be calculated by using the formulas given in the AISC-LRFD specifications.[1] By using clear length of the diagonal and taking the effective length factor k about 0.80, the buckling load P_{cr} was found to correlate well with experiments. Trial and error was used to determine the size of the diagonal bars to satisfy Equation 4.

Design of Vertical Members

With the designed chord members and diagonal bars, the force in the vertical bar P_v can be found by equilibrium. From Figure 2c:

$$P_v = 1.4(P_{xy} - 0.3P_{xc})\sin(\theta_x) \tag{5}$$

or conservatively, the compression force in vertical member can be taken as:

$$P_v = 1.4P_{xy}\sin(\theta_x) \tag{6}$$

Design of Welds

The welds for the diagonal bars should be designed to take the fully strained hardened force $1.4P_{xy}$. The welds for the vertical member should be designed to develop the full plastic moment of the vertical member.

Required Strength of Opening Under Gravity Load Condition

The previously mentioned design procedure for the girder web opening is a limit state design procedure which considers the force distribution at ultimate lateral load condition. It was assumed that the dead load is small. However, the opening should also be checked against the gravity load combination, $1.4DL + 1.6LL$, even though its effect may be small. Under this condition, all the members in the opening should remain in the elastic range. The design philosophy is based on permitting activity in the openings in the event of extreme earthquake lateral loads.

Detailing of Opening

Stress concentration is a major factor in initiating cracks in steel structures. In the case of the proposed upgrading scheme, the stress concentration is highest at the corners of the opening. Therefore special detailing should be provided to minimize stress concentration and also to move the location of plastic hinges away from the critical corners. During the small scale tests carried out to determine the feasibility of using the proposed system, many detailing patterns were investigated to find the best scheme. The detailing scheme shown in Figure 12 was found to behave well. It provided the needed ductility and was also practical. The opening can be flame-cut but the surface of the cut close to the corners should be smoothed out by grinding. It is desirable to have radius in all corners, although test results suggested that it might not be necessary. The vertical members at the ends of the opening should be placed at an offset about 0.5 inches from the ends of the opening. This is done so that the plastic hinges are pushed away from the critical areas. The diagonal bars as shown in Figure 12 also help in reinforcing the critical areas. All welds should have at least 0.25-in. clearance from the edges to allow plastic flow, thus increasing local ductility.

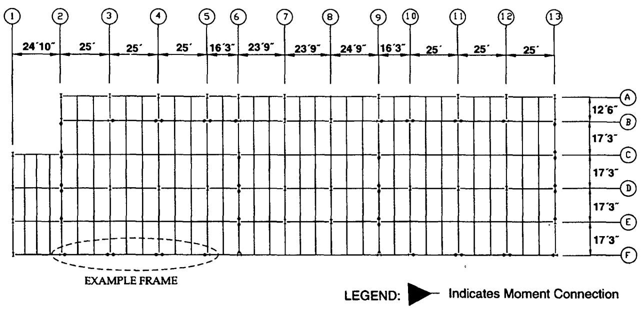


Fig. 3. Plan view of the example building.

DESIGN EXAMPLE

The building selected for this study is a typical six story moment frame structure which was located near the epicenter of the January 17, 1994, Northridge Earthquake. The perimeter moment frames in the North-South direction suffered significant damage to the welded moment connections during the earthquake. The building was the subject of an earlier study by Hart et al.[4] The plan view of the structure is shown in Figure 3. The primary lateral force resisting system in the N-S direction consists of four identical three-bay moment frames above the ground level. The bottom story is below grade with extensive outside and interior basement walls. Therefore, for purposes of this study only the five stories above ground level were considered. The effect of gravity load was neglected since the moment frames are perimeter frames, therefore, the effect of gravity load is significantly smaller than the effect of lateral load. A typical three-bay frame along with member sizes is shown in Figure 4.

In order to study the viability of the above mentioned scheme, the moment frame structure of the example building was modified according to the proposed procedure. Taking the fourth floor girder (W36×150) for an example, and assuming grade 50 steel, the maximum allowable shear at mid span is:

$$\frac{2\phi M_v}{L} = \frac{2(0.9)(504)(50)}{300} = 151.2 \text{ kips}$$

Choosing the length of the opening as 0.20 times the span length and with a target overstrength factor of 2.0 for the chord members, the appropriate depth of the chord members

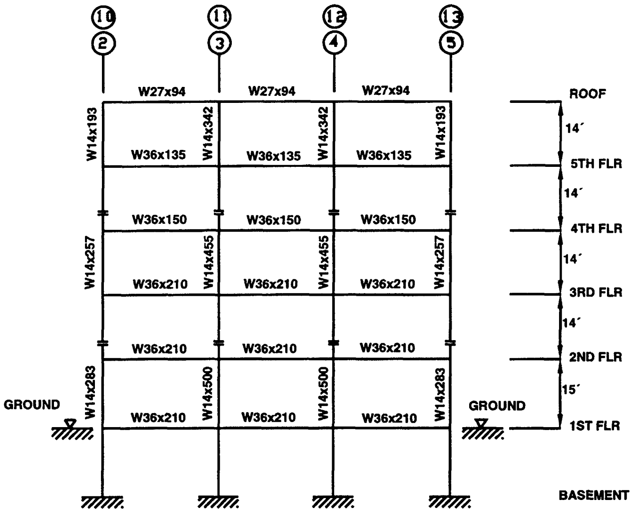


Fig. 4. Typical three-bay moment frame in N-S direction.

Table 1.
Design of Web Opening

Beam Size	$\phi_b M_y$ (kip-in.)	V-allowable (kips)	Depth of Chord (in.)	Overstrength Factor	V-chord (kips)	Diagonal Members	V_x (kips)
W27×94	10,935	72.9	3.75	2.00	28.7	1½×¾	42.5
W36×135	19,775	131.7	3.75	1.97	35.5	1×1⅞	88.0
W36×150	22,680	151.2	4.25	2.09	51.0	1⅞×1⅛	100.2
W36×210	32,335	215.7	4.50	2.01	80.0	2×1⅜	133.0

Note: Calculations based on $F_y = 50$ ksi for chord members and $F_y = 36$ ksi for diagonal members.

Table 2.
Member Sizes of the Modified Frame with Web Openings

Floor	Beam Size	Opening Length (in.)	Depth of Chord Sections (in.)	Diagonal Members (in. × in.)	Vertical Members
Roof	W27×94	60	3.75	1½×¾	2L2×2×3⁄16
5	W36×135	60	3.75	1×1⅞	2L2×2×⅜
4	W36×150	60	4.25	1⅞×1⅛	2L2½×2½×5⁄6
3	W36×210	60	4.50	2×1⅜	2L2½×2½×½
2	W36×210	60	4.50	2×1⅜	2L2½×2½×½

was found by trial and error to be 4.25 inches, with an exact overstrength value of 2.09 and plastic moment M_{pc} of 347.6 k-in. Therefore the shear provided by the chords from Equation 2 is:

$$\frac{4(\xi_c M_{pc})}{(0.95L_o)} = \frac{4(2.09)(347.6)}{0.95(0.20)(300)} = 51.02 \text{ kips}$$

By taking $\xi_x = 1.4$, the shear contribution from the diagonal members should be, from Equation 4:

$$1.4(P_{xy} + 0.3P_{xc})\sin(\theta_x) = 151.2 - 51.02 = 100.18 \text{ kips}$$

Using 1⅞-in. × 1⅛-in. bars interconnected at the mid length, the yield force and buckling force (with $k = 0.80$ and $l_x = 23$ in.) were found to be 76.0 kips and 64.3 kips, respectively. Taking θ_x to be approximately as 49 degrees (Figure 5b), the contribution for diagonal members is:

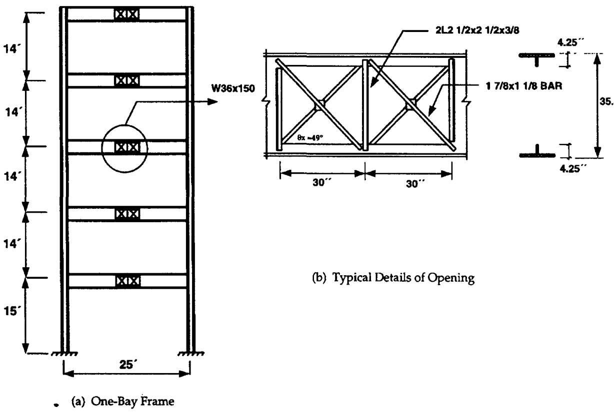
$$1.4(P_{xy} + 0.3P_{xc})\sin(\theta_x) = 1.4(76.0 + 0.3 \times 64.3)\sin(49°)$$

$$= 100.2 \text{ kips}$$

The total shear force provided by the opening is 100.2 + 51.02 = 151.2 which is adequate. With the selected bar size, the compression force in the vertical member is:

$$P_v = 1.4P_{xy}\sin(\theta_x) = 79.9 \text{ kips}$$

Therefore, double angles 2L2½×2½×⅜ with calculated critical load $\phi P_{vc} = 95$ kips were used. Calculations for the other floor girders are summarized in Tables 1 and 2. The modified frame with openings is shown in Figure 5.



(a) One-Bay Frame

(b) Typical Details of Opening

Fig. 5. Modified frame with girder web openings.

NON-LINEAR ANALYSIS OF THE EXAMPLE BUILDING

In order to study the behavior of the structure modified by the proposed scheme, non-linear analyses were performed in order to compare the behavior before and after the modification. One-bay, five story models of the original three-bay moment frame and the modified frame with web openings were prepared for inelastic static ("pushover") and an inelastic dynamic analysis. SNAP-2DX computer program was used for the inelastic analyses.[11] Modeling assumptions as used in previous studies[2,3,6] were used in this study as well. Floor masses of the frame were lumped at the beam-to-column connection nodes. The damping was taken as 2 percent and proportional to the mass matrix in the dynamic analysis using the estimated period calculated from the 1994 UBC. Beam-column elements were used to model the girders, columns, and chord members of the openings. Jain's buckling elements[7] were used for diagonal web members. Centerline dimensions were used along with rigid end offsets at the element connections with lengths equal to one-fourth of the beam and column depths. Gravity loads were small and, therefore, were neglected in design and analysis. The yield stress for the girders and columns was taken as 55 ksi (expected for grade 50 steel) and, for diagonal members the yield stress, was taken as 49 ksi (expected for grade 36 steel). The panel zone deformations of these frames were not considered since the main purpose was to compare the overall behavior of the modified frame to that of the original frame.

The static "pushover" analysis was carried out by applying lateral forces representing the distribution of UBC design lateral forces. For the inelastic dynamic analysis the two models were subjected to the N-S component of the 1978 Miyagi-ken-oki accelerogram with peak ground acceleration equal to 0.4g. This scaled ground motion record has been used in some previous studies to represent a ground motion on soft soil whose damped elastic response spectrum matched with the idealized elastic design spectra specified in the 1994 UBC.[2,3,6] The results from the above two analyses are presented and discussed in the following sections.

Table 3.
Floor Masses of the Original Building

Floor	Floor Mass (kip-in./sec/sec)	Weight (kips)
Roof	6.26	2,416.4
5	5.45	2,103.6
4	5.45	2,103.6
3	5.45	2,103.6
2	6.93	2,675.0

Inelastic Static "Pushover" Analysis

Figure 6 shows base shear versus roof displacement plot for the two models. Also shown in the figure are the sequences of inelastic activity in the two frames and the 94 UBC design base shear for the structure with the soil factor, S, taken equal to 1.5. The horizontal drift at design lateral forces in both frames satisfied the UBC limits. Calculation of UBC design lateral forces and drifts are shown in Tables 3, 4 and 5. The strength corresponding to first significant non-linearity in the force-displacement plot for the original frame is approximately twice that of the modified frame. Both frames pos-

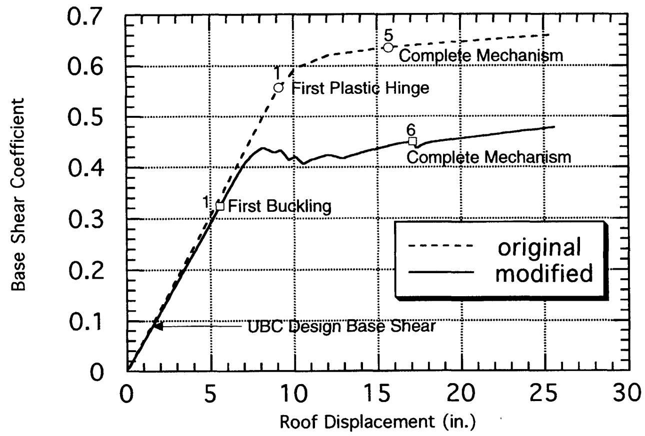
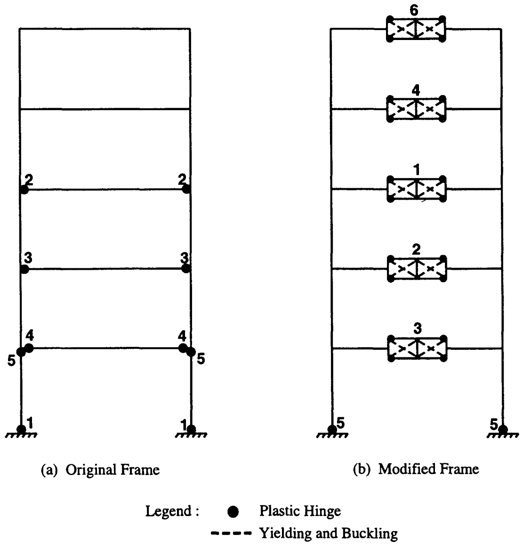


Fig. 6. Force-displacement response from non-linear static (pushover) analysis.



(a) Original Frame

(b) Modified Frame

Fig. 7. Yielding and mechanism under static lateral forces.

Table 4.
UBC Design Lateral Forces for the Original Frame

Floor	h_x (ft)	$W_x h_x$	$W_x h_x / \Sigma W_i h_i$	F_t (kip)	F_x (kip)	F_x/frame (kips)	Ftorsion + 5% Ecc.	Total F_x (kips)
Roof	71	171,564.4	0.36	47.4	266.2	78.4	23.5	101.9
5	57	119,905.2	0.25	—	184.9	46.2	13.8	60.0
4	43	90,454.8	0.19	—	140.5	35.1	10.5	45.6
3	29	61,004.4	0.13	—	96.1	24.0	7.2	31.2
2	15	40,125.0	0.08	—	59.2	14.8	4.4	19.2

Note: Calculations based on $Z = 0.4$, $I = 1.0$, $S = 1.5$ (Soil Type S3), $T = 0.035h^{3/4} = 0.86$ sec, and $R_w = 12$ according to 1994 UBC.

sess significant overstrength above the design force level—approximately 7 times for the original frame and 5 times for the modified frame. The occurrence of inelastic activity in the two frames was quite different too, as seen from Figure 7. In the original frame the first set of plastic hinges to form was at the column base and the yield mechanism was that of story type in the first story, both of which are not considered as good behavior. Early formation of plastic hinges at the column base can mean large ductility demands at a rather critical location and the formation of story mechanism can lead to more serious consequences including collapse in some cases. The modified frame, on the other hand, behaved in a truly strong column fashion as intended with inelastic activity essentially limited to the openings in the girders and minor flexural yielding at the column base forming almost last at much larger frame drift.

Inelastic Time-history Dynamic Analysis

Some selected results from the inelastic dynamic analysis of the two frames are presented briefly in Figures 8 through 11. The envelopes of maximum floor displacements of the two frames are shown in Figure 8. The floor displacements of the modified frame with web openings are consistently smaller than those in the original frame. The maximum story drifts, shown in Figure 9 are similar, but more significantly, the maximum story drift in the first story is approximately half of that in the original frame. This is because a story mechanism in the first story formed in the original frame as seen in Figure 10 where the inelastic activity in the two frames is shown. It can also be noticed from Figure 10 that there is widespread distribution of plastic hinges in the original structure with large plastic rotation demand at the column bases. In contrast, the inelastic activity in the modified frame is much more controlled and limited to the openings as was intended in the design.

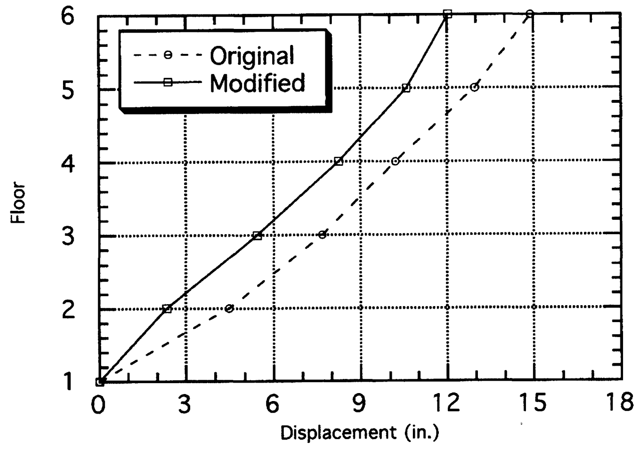


Fig. 8. Maximum floor displacements under Miyagi-Ken-Oki (PGA = 0.4g).

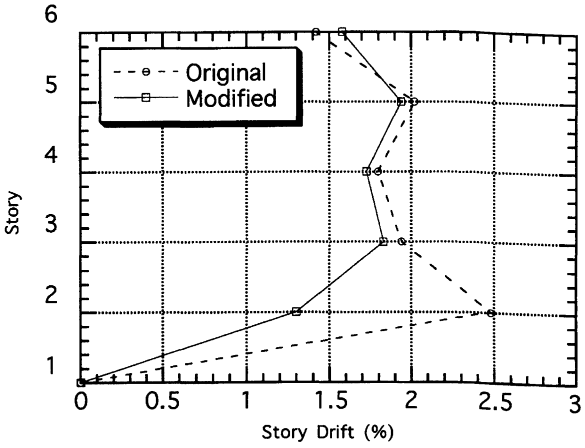


Fig. 9. Maximum relative story drifts under Miyagi-Ken-Oki(PGA = 0.4g).

Figure 11 shows the time-history of horizontal displacements at the second floor and the roof levels. Not only are the floor displacements in the modified frame smaller than those in the original structure, but also the excursions with larger amplitude are much fewer in the former. Formation of story mechanism in the first story after ten seconds into the response resulted in larger displacements in later cycles. Thus, controlled inelastic activity in the structure to follow a desired yield mechanism results in better response as well as the damage inspection and repair work after the earthquake should also be relatively easy and less costly effort.

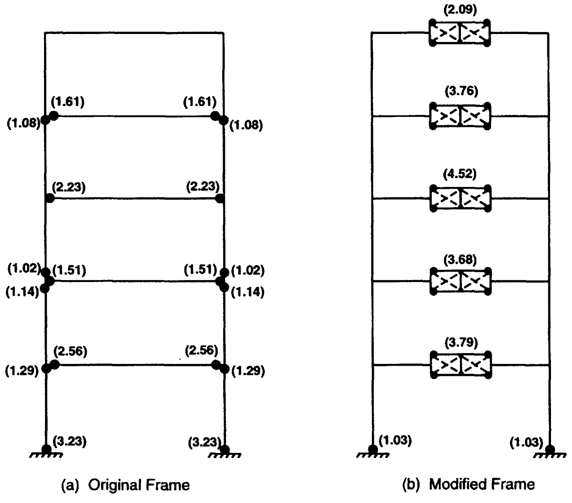
EXPERIMENTAL PROGRAM

In order to validate the proposed upgrading scheme, the behavior under cyclic load of a one-story subassemblage consisting of a full scale 28 ft. long W24×62 beam with a web opening, and two 13 ft W14×82 columns at the ends of the beam was investigated experimentally. The columns were half-story high above and below the beam with pinned ends at both the top and bottom of the columns. The frame represented a story in a one bay frame assuming that inflection points were at the mid height of the story. Lateral braces were provided at the third points of the span length. The opening was designed according to the previously mentioned procedures such that the maximum applied force at 3 percent story drift was approximately 65 kips. The shear force in the opening resulting from this force would create moment at the end connections corresponding to about 85 percent of the yield moment. This maximum force was chosen due to the capacity of the actuator as well as the capacity of the columns which were previously designed and used for other tests. The chords of the opening in the specimen were designed to have an expected overstrength factor of about 1.80 at 3 percent story drift. The dimensions of the test frame and the opening are shown in Figure 12.

Table 5.
Design Story Shears and Story Drifts

Story	Story Shear (kips)	Story Drift (%)
6	101.9	0.15
5	161.9	0.19
4	207.5	0.18
3	238.7	0.18
2	257.9	0.16
Design Base Shear Coefficient (V/W) = 0.090.		

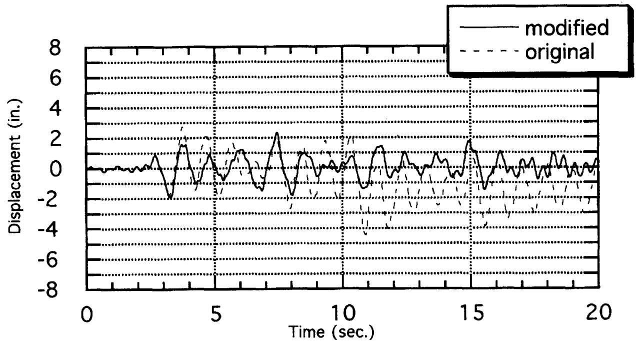
The experiment consisted of applying two loading histories one immediately after another to the frame. The first loading



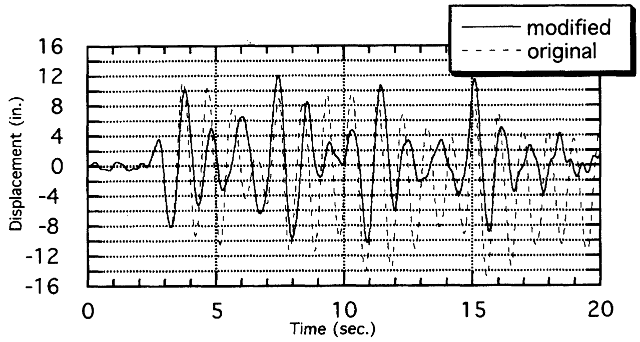
(a) Original Frame (b) Modified Frame

Note: Ductility ratios at plastic hinges shown in parentheses

Fig. 10. Inelastic activity under Miyagi-Ken-Oki accelerogram (PGA = 0.4g).



a) Time-History of Displacements of the Second Floor



b) Time-History of Displacements of the Roof

Fig. 11. Time-history of displacements under Miyagi-Ken-Oki (PGA = 0.4g).

history consisted of cycles of increasing displacements up to about 0.9 percent story drift where buckling and yielding started. The second loading history consisted of cycles of large displacement amplitudes up to 3 percent story drift. The 3 percent story drift was used because many studies have shown that, during a severe earthquake, steel structures usually experience story drifts about 2 percent to 3 percent and also because the maximum stroke of the actuator was in the order of 3 percent story drift of the frame. The first and the second loading history are shown in Figure 13.

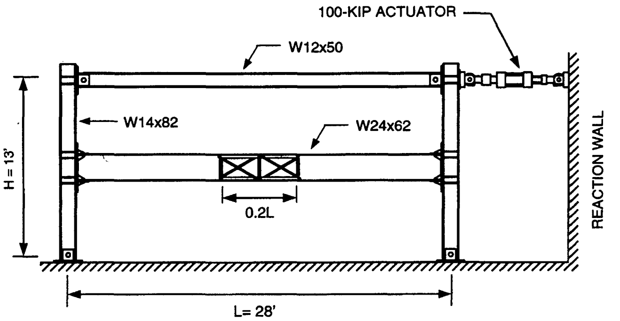
Evaluation of Test Results

The hysteretic loops from the first and second loading histories as well as the analytical loops from computer analyses using similar modeling assumptions as discussed earlier are shown in Figure 14. The only difference in modeling assumptions is that the panel zone deformation was included as suggested by Krawinkler.[8] As can be seen from the first loading history, the response started to deviate from elastic behavior at a drift of about 0.75 percent. Two of the diagonal members buckled at this displacement and some yielding in the diagonal members was observed when story drift was about 0.9 percent. During the second loading history, after the diagonals had completely buckled and yielded, the chords of the opening started to plastify and plastic hinges formed visibly at the end of 1.8 percent story drift displacement cycle. Progressing further into later displacement cycles showed mechanism pattern as intended in the design, i.e., yielding and buckling of the diagonal members followed by plastic hinges at the ends of the chord members. The complete mechanism of the test frame is shown in Figures 15 and 16.

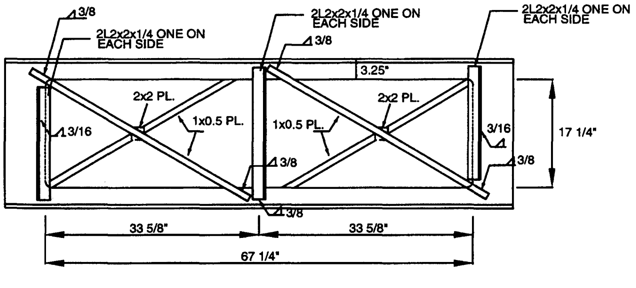
The test specimen was able to sustain many cycles of large displacements without any fracture. Only local buckling in the chords and local necking in the diagonal bars due to very high local strain were observed. The effect of these local instabilities resulted in small reduction in the load carrying capacity of the frame during the 3 percent story drift cycles (Figure 14b). The maximum load obtained from the test was 65.8 kips at the first 3 percent story drift cycle. This maximum load agreed very well with the design value of 65 kips. The chord member later fractured after the frame was subjected to additional decreasing displacement cycles which are not shown here. Overall, the proposed upgrading system showed a very ductile behavior and all inelastic behavior was confined to only the designated elements of the web opening.

CONCLUSIONS

An upgrading scheme for steel moment resisting frames was proposed in this paper. The results of both analytical and experimental studies of the response of the frame modified with the proposed system were presented. The upgrading scheme consists of creating a ductile rectangular opening in the middle of the beam web to control the yield mechanism of the frame. The detailed design procedure was developed

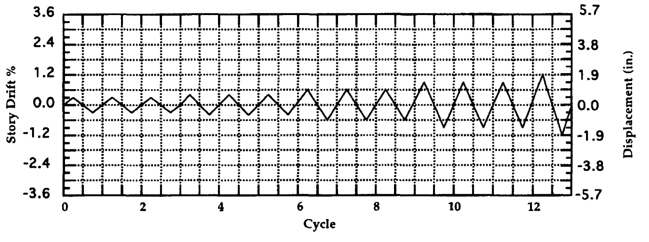


a) Dimensions of Test Frame

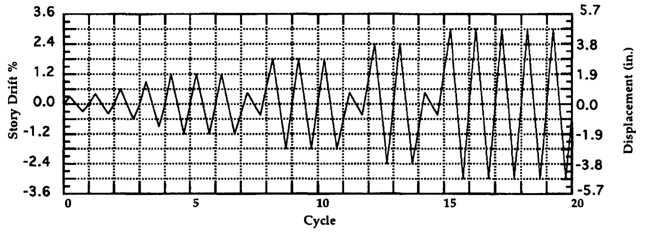


b) Details of Opening

Fig. 12. Dimensions and opening details of the test subassemblage.



a) First Loading History



b) Second Loading History

Fig. 13. Loading histories used in the test.

based on small scale tests of beams with opening conducted in earlier phase of this study. In the analytical parts, an actual existing frame was modified according to the design procedure presented herein as an example. The results of both non-linear static and dynamic analyses of the modified frame showed that the modified frame responded as expected with an excellent behavior. From static analysis, the modified frame showed somewhat less global stiffness because of the presence of the openings. However, the modified frame responded to a severe ground motion with less story drifts which were the result of controlled yield mechanism. Moreover, the risk of having premature failure of beam-to-column connections was much reduced since no plastic hinge formed at the connections due to openings in the beams. A full scale test was finally carried out to validate the key assumptions used in analyses and design. The test results confirmed the validity of the proposed analysis and design procedure.

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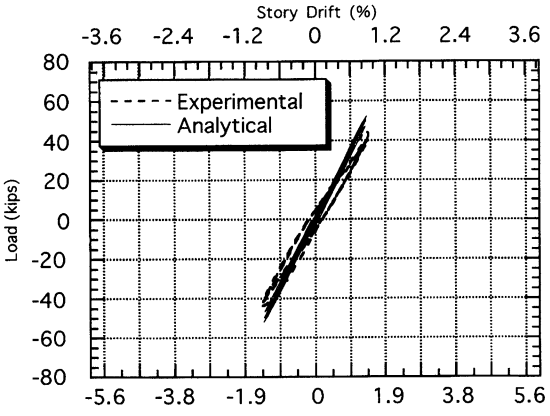
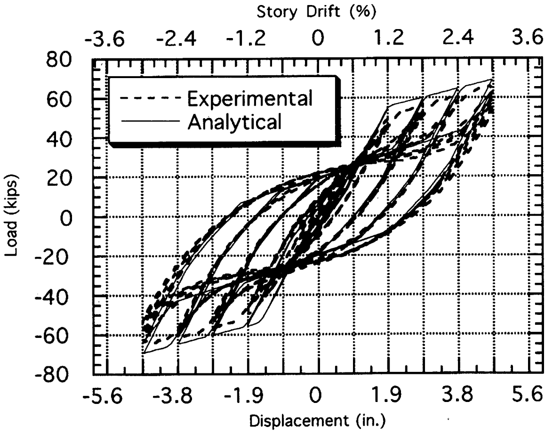


a) Hysteretic Loops from First Loading History



b) Hysteretic Loops from Second Loading History

Fig. 14. Experimental and analytical hysteretic loops.

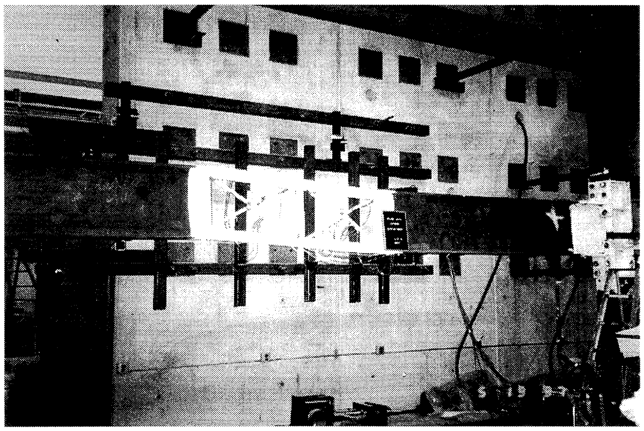


Fig. 15. Deformation of the girder with ductile opening.

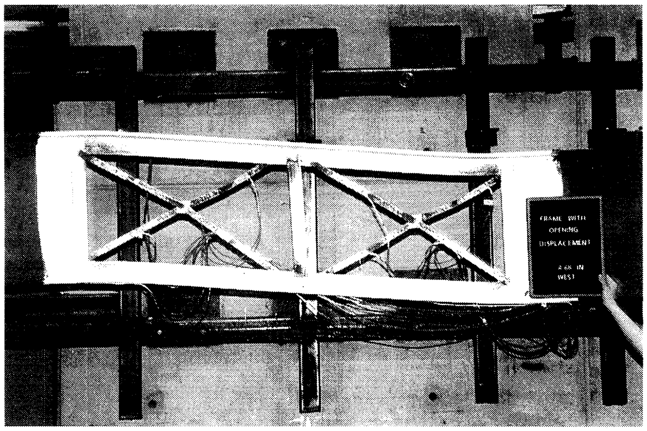


Fig. 16. Yielding, buckling and plastic hinges in the opening.

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