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A Study on a Repair Technique for a Reinforced Concrete Frame Subjected to Seismic Damage Using Prestressing Cable Bracing

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Abstract

The proposed building upgrading technique employs prestressing cables to function as bracing to improve the seismic performance during future events. A four-story reinforced concrete moment resisting frame damaged from an ultimate limit state earthquake is assessed and upgraded using the proposed technique. Both existing and upgraded buildings are evaluated in regard of seismic performance parameters performing static lateral load to collapse analysis and dynamic nonlinear time history analysis as well. To obtain realistic comparison of seismic performance between existing and upgraded frames, each frame is subjected to its critical ground motion that has strength demand exceeding the building strength supply. Furthermore, reliability of static lateral load to collapse analysis as a substitute to time history analysis is evaluated. The results reveal that the proposed upgrading technique improves the stiffness distribution compared to the ideal distribution that gives equal inter-story drift. As a result, the upgraded building retains more stories that contribute to energy dissipation. The overall behavior of upgraded building beyond yield is also enhanced due to the gradual change of building stiffness as the lateral load increases.

Keywords: upgrading technique, earthquake, seismic performance

1. INTRODUCTION

The lessons learned from recent seismic events have refined the building design codes for new construction. Yet, a large number of structures currently in use were built under older codes and pose substantial risk of earthquake-caused damage.[1),5)] It has been revealed that the major source for loss of life and economic loss comes from poorly designed and poorly constructed existing buildings.[3),13)] In addition to mitigating life hazard, the fundamental goal of repairing lies in economic reason. The continuous rise in construction cost, will provide people with great incentive for upgrading old existing buildings rather than demolishing them for new construction.

Recent research on seismic hazard abatement is two fold: the first is the assessment of building performance characteristics; and the second is to develop retrofitting approaches for deficient building.

This study deals with the second task, a retrofitting technique in which prestressing cables are used to act as bracing for a reinforced concrete moment resisting frame to enhance the seismic performance during future events. It is expected that such structures will experience inelastic deformation at certain critical location in strong or moderate seismic disturbance. Thus, the nonlinear response of the structure and of its components in which it invalidates all the theories based on elastic analysis is significant.

2. OBJECTIVES AND SCOPE

The purpose of this study is to evaluate an upgrading technique by which reinforced concrete moment resisting frame damaged from an ultimate limit state earthquake can be strengthened and repaired. The technique uses prestressing cables to function as bracing to enhance the performance of building behavior during future events.

Performing nonlinear time history analysis, the behavior of the original and upgraded building under the effect of the critical ground motion is also studied.

Building seismic performance parameters are given as strength, stiffness, and ductility.[9)] These are only applicable to building type structures. Consequently, the method is intended for upgrading the building performance parameters as pertaining to the superstructure only. Characteristics and behavior of substructure, soil and foundation are not implemented at this stage. The soil-structure interaction that may affect the building behavior significantly will be included in the future study.

3. UPGRADING REINFORCED CONCRETE FRAME

A 4-story moment resisting reinforced concrete building is assessed and upgraded using the proposed technique. Seismic performance parameters of the building are evaluated performing both static lateral load to collapse analysis and dynamic time history analysis. Reliability of static lateral load to collapse analysis as a substitute to time history analysis is evaluated.

3.1 Building Appraisal

The building is a 4-story office building, and has 7 bays with a total length of 62.9 m in the EW direction and 3 bays with a total length of 29.3 m in the NS direction as shown in Figure 1.[6] The building frame is reinforced concrete with 4.26 m story heights and its total height is 17.1 m. A typical frame from a cross section of building in the NS direction is selected for this study. The geometries and reinforcement of beams and columns are shown in Figure 2.

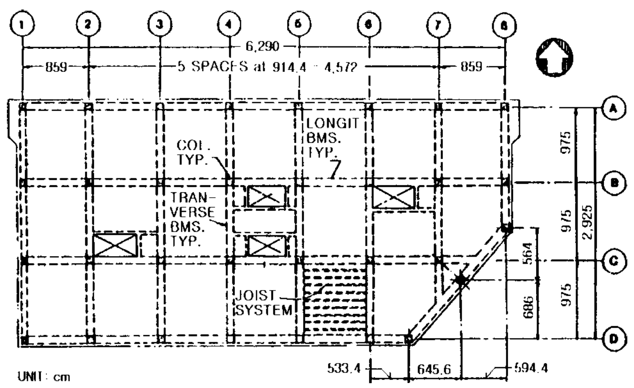

Figure 1. Building Plan

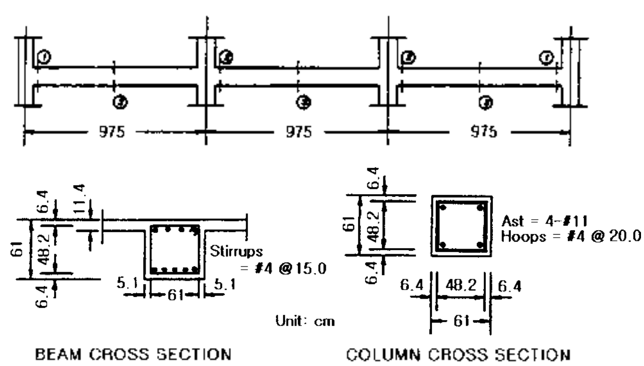

Figure 2. Typical Beam and Column Cross Section

The Kent & Park model is used to represent the concrete stress-strain curve. The 28-day uniaxial compressive strength of concrete (f_c') is 27.6 MPa for all beams and 34.5 MPa for all columns in the frame. The strain at the maximum stress $\varepsilon_o = 0.002$ is used.[1] $7.5\sqrt{f_c'}$ (in psi) is the assumed tensile rupture stress of concrete, and 0.01 is used as the concrete crushing strain (ε_{cr}).[4] Steel rebars are assumed to have elastic perfectly plastic stress-strain properties. Rebars have a yield stress of 413.4 MPa and a Young's modulus of 203.2 GPa.

Cross sectional properties are calculated as moment-curvature relationships using a program developed for reinforced concrete section analysis (i.e. home made program).[10] The effect of confined concrete is taken into account by the program. Moment-curvature relationships for all unique cross section in the frame are given in Figures 3 and 4. The axial load effects are ignored when the moment-curvature relationships are computed for the cross section of the beams. However, axial forces induced from gravity load are included in calculating these relationships for the columns. The sections are assumed to reach their ultimate moment capacity when concrete strain reaches a value of 0.01.

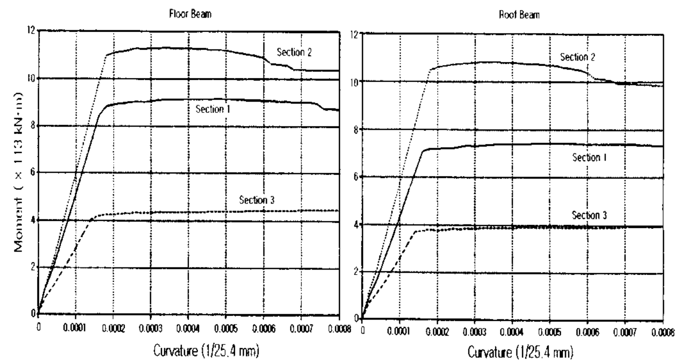

Figure 3. Moment vs. Curvature for Beams

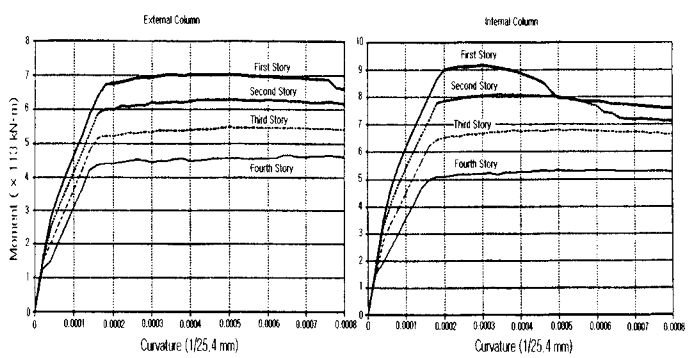

Figure 4. Moment vs. Curvature for Columns

Member properties for the frame are determined as moment-rotation relationships with the assumption: an inflection point is assigned to the middle of the beam due to lateral load. Conjugate beam method is used to obtain moment-rotation relationship for each beam, applying the yield moment and the ultimate moment at the end one after another. In order to find the member stiffness, moment-rotation curves are constructed for each end. From each end moment-rotation curve, two stiffness can be distinguished: (1) the elastic stiffness (k_e), (2) the plastic stiffness (k_p). The strain hardening (SH) is obtained by dividing the plastic stiffness (k_p) by the elastic stiffness (k_e). The computed rotations (θ_y, θ_u), the elastic stiffness, plastic stiffness, and the strain hardening for all cross-sections of beams and columns are shown in Tables 1, 2 respectively.[2,12]

Table 1. Stiffness and Strain Hardening for Beams

BEAM	Sec. No.	M_y kN·m	ϕ yield	M_u kN·m	ϕ ultimate
FLOOR M -ve	Sec. 1	951.2	7.007E-6	977.2	4.417E-5
	Sec. 2	1181.2	7.598E-6	1196.0	1.063E-5
ROOF M -ve	Sec. 1	687.4	6.653E-6	987.0	1.614E-5
	Sec. 2	1130.5	7.559E-6	1143.9	1.024E-5
M +ve	FLOOR	268.0	4.331E-6	281.4	5.512E-6
	ROOF	211.8	3.543E-5	222.3	4.724E-6

BEAM	Sec. No.	θ yield	θ ultimate	K_e	K_p	SH
FLOOR M -ve	Sec. 1	0.00854	0.0180	111382	2785	0.025
	Sec. 2	0.00926	0.0189	127559	1531	0.012
ROOF M -ve	Sec. 1	0.00811	0.0176	84760	2628	0.031
	Sec. 2	0.00922	0.0187	122614	2330	0.019
M +ve	FLOOR	0.00114	0.0185	235087	705	0.003
	ROOF	0.00110	0.0170	192545	578	0.003

Table 2. Stiffness and Strain Hardening for Columns

	FLOOR	P kN	M_y kN·m	ϕ yield	M_u kN·m	ϕ ultimate
Exterior Column	1st	1521.3	730.4	7.007E-6	756.6	1.260E-5
	2nd	1134.3	652.9	6.693E-6	687.4	1.417E-5
	3rd	742.8	566.8	6.339E-6	599.0	1.654E-5
	4th	347.0	478.1	5.984E-6	509.6	2.520E-5
Interior Column	1st	2855.7	984.9	8.150E-6	985.7	1.023E-5
	2nd	2126.2	855.1	7.480E-6	877.1	1.220E-5
	3rd	1396.7	705.9	6.890E-6	738.9	1.181E-5
	4th	671.7	552.0	6.230E-6	585.0	2.008E-5

	FLOOR	θ yield	θ ultimate	K_e	K_p	SH
Exterior Column	1st	0.00373	0.00810	195818	7833	0.040
	2nd	0.00357	0.00790	182885	7864	0.043
	3rd	0.00338	0.00772	167692	7378	0.044
	4th	0.00318	0.00803	150346	6615	0.044
Interior Column	1st	0.00435	0.00870	226413	226	0.001
	2nd	0.00400	0.00831	213775	5131	0.024
	3rd	0.00368	0.00793	191821	7673	0.040
	4th	0.00334	0.00789	165270	7272	0.044

3.2 Analysis of the Original Frame

From static lateral load to collapse analysis performed by "ULARC", the sequence of plastic hinge formation and its location are obtained as lateral loads increase monotonically as shown in Figure 5, with which dead load for roof and for the typical floor is 5.62 kN/m^2 and 6.34 kN/m^2, respectively. The total dead load of the typical frame is computed as 7339.5 kN. The live load for roof and for the typical floor is 960 N/m^2 and 2400 N/m^2, respectively. As shown in the figure, the plastic hinges are concentrated in the first and second stories, and the fourth story has remained essentially elastic. The frame response (base shear vs. roof drift) under the effect of two lateral loads is shown in Figure 6. As shown in this figure, the yield occurs at base shear of 0.126W and 0.135W for the case of triangular and rectangular load respectively. Thus, the triangular lateral load distribution is applied to the frame throughout this study.

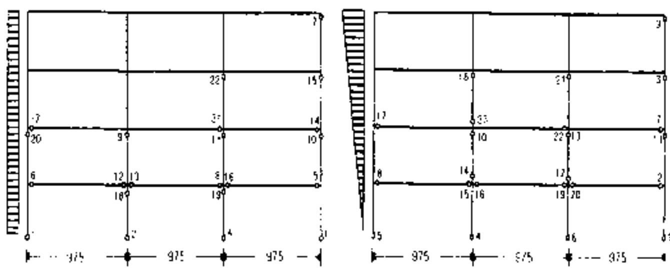

Figure 5. Sequence of Plastic Hinges

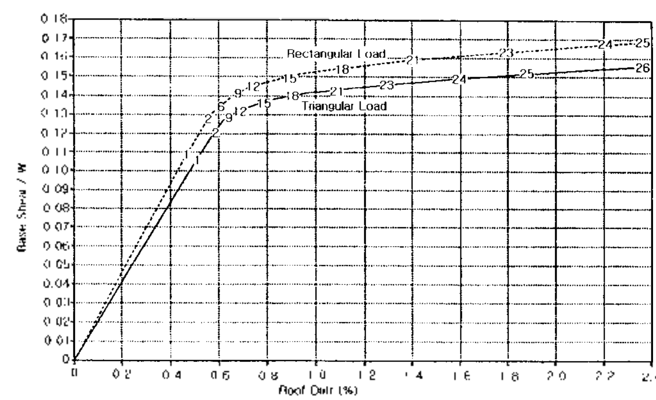

Figure 6. Base Shear vs. Roof Drift

Figure 7 represents the relationships between base shear and inter-story drift. It is obvious that the first and second story yielded at base shear of 0.126W and the story drift at yield is 0.55% and 0.78% for the first and second story, respectively. The third story yields at base shear of 0.136W and story drift equal to 0.75%. On the other hand, no yield occurs in the fourth story. Story shear vs. inter-story drift is also shown in Figure 7. Story strength of first, second, third, and fourth from the figure is 0.126W, 0.11W, 0.085W, and 0.052W respectively.

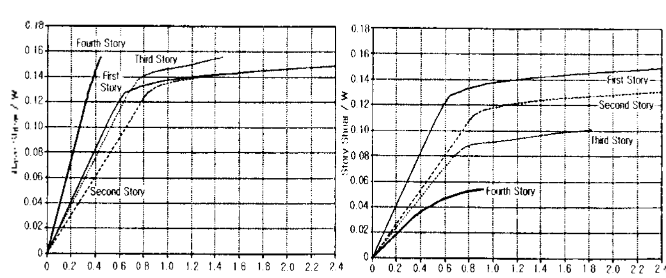

Figure 7. Base Shear vs. Inter-story Drift and Story Shear vs. Inter-Story Drift

After static analysis is performed, the building is subjected to TAFT earthquake (California, 1952) for dynamic time history analysis. The TAFT record is given at every 0.02 second and has a duration of 54.38 seconds with a maximum acceleration of 0.18g at 3.7 second. The first 7 seconds of the record that includes the maximum acceleration is taken into consideration in this study.

The elastic response spectra for TAFT are generated using "SPECTR" program and shown in Figure 8. As shown in this figure, the fundamental period of the original frame (T_{OR}) lies at the constant speed region of the spectra. To insure plastification of the building, response spectra are required to be shifted along the constant acceleration region of the spectra towards higher fundamental period. At this location, when the fundamental period increases due to the effect of the first plastic hinge, the acceleration which affects the building increases and allows to form another plastic hinge. Thus, the response spectra are shifted from $T_{OR}=1.17$ seconds to $T_{OR}=0.15$ by modifying recorded time increment as shown in Figure 8. The time increment has changed from 0.02 second to 0.156 second to shift the response spectra to the desired location. The maximum ground acceleration ($\ddot{y}_{g\,max}$) is also scaled to be a ground motion with a strength demand that meets or just exceeds the building's strength supply. From the static lateral load to collapse analysis, the yield base shear (C_y) of the original frame is 0.126W. Thus, the maximum acceleration is scaled (k_a) down to: $k_a = 0.126g/0.40g = 0.315$.

Then, the maximum ground acceleration ($\ddot{y}_{g\,max}$) of scaled TAFT earthquake becomes 0.0567g. However, 0.065g with base shear 0.15W is selected since the relationship between the maximum ground acceleration and the acceleration that affects the building is not linear. The designed elastic response spectra for the acceleration record with $\ddot{y}_{g\,max} = 0.065g$ and $\Delta t = 0.156$ second are also given in Figure 8.

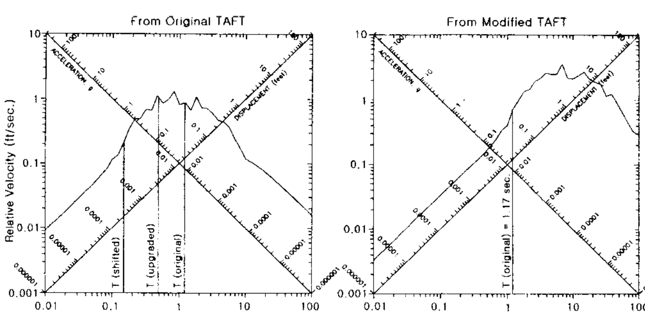

Figure 8. Response Spectrum of the Original and Modified TAFT

The building is subjected to the modified TAFT earthquake and the responses are obtained using "ABAQUS" program.[7] Base shear vs. time is shown in Figure 9. The maximum base shear that affects the building during the ground motion is 0.15W at 37.5 seconds. Figure 9 also shows the roof drift vs. time. The maximum roof drift is 0.8% and occurs at 37.5 second. Story shears vs. inter-story drifts are calculated for each story and shown in Figure 10. It is clear that the third and fourth stories have remained essentially elastic.

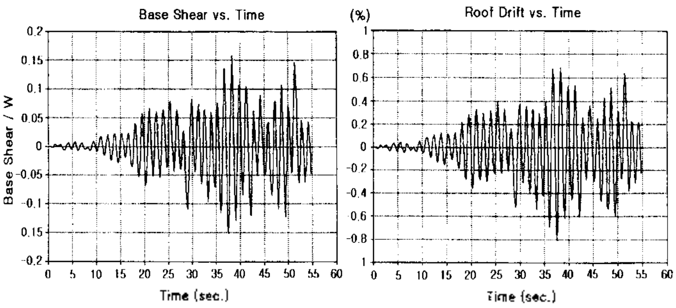

Figure 9. Base Shear vs. Time and Roof Drift vs. Time

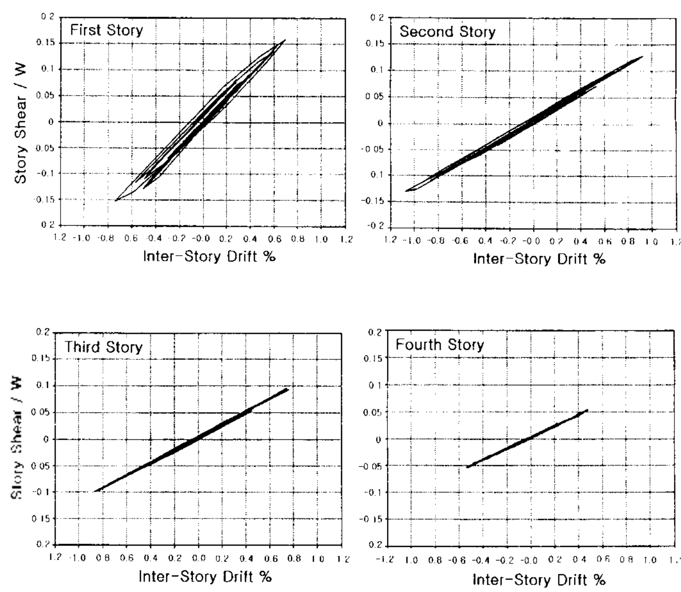

Figure 10. Story Shear vs. Inter-Story Drift

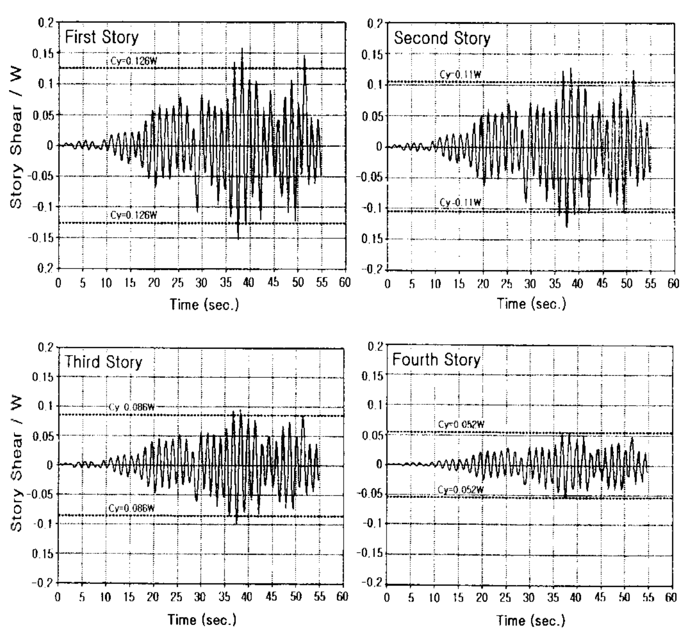

Figure 11. Story Shear vs. Time

The first story suffers the most plastification and reaches its potential strength supply. The second story suffers some small amount of plastification. This agrees well with the results of the static lateral load to collapse analysis. Figure 11 shows story shear vs. time for each story. Projecting C_i values obtained from static lateral load to collapse analysis onto the figure, it shows that the results from time history analysis match well with those extracted from static lateral load to collapse analysis. Figures 12 shows inter-story drift vs. time for each story. As shown, the first and second stories suffer the most plastification and the fourth story remains practically elastic. Again, the results match well with those obtained from static lateral load to collapse analysis.

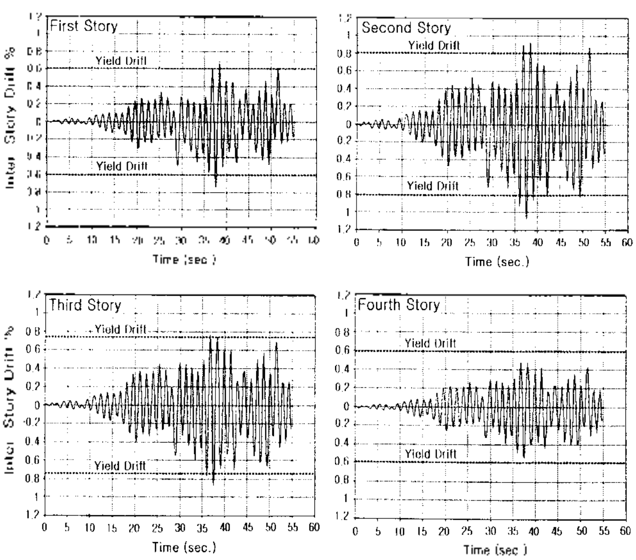

Figure 12. Inter- Story Drift vs. Time

3.3 Assessment and Upgrading

The analysis of the original frame indicates deficiencies in strength and stiffness supply distributions along the building height. The first and second stories are subjected to shear demands above their yield shear supplies and fourth story is subjected to shear demand below its respective yield supply. The ability of the structure to resist forces beyond the elastic limit is confined to the first and second stories.

It is assumed that the original frame has been damaged during an ultimate limit state earthquake with base shear equal to 0.15W. This damaged frame is upgraded by applying the proposed technique to enhance its seismic performance using prestressing cable bracing (PSC). The bracing system is designed to resist a triangular lateral load distribution. The building is assumed to be in a highest seismicity zone. Thus, the total horizontal force is calculated as a base shear equal to 20% of the total frame weight (0.2W). This value of base shear is greater than the required value in current US codes. The force, calculated in the bracing when the building frame is subjected to a lateral load corresponding to a base shear of 0.2W, is applied as the prestressing force. Thus, 0.2W base shear is the ultimate limit of the proposed technique without any factor of safety. The best location of the PSC is the middle bay due to the distribution of plastic hinges revealed from static lateral load to collapse analysis. The upgraded frame with the expected location of plastic hinges and the required lateral load distribution to be resisted are shown in Figure 13.

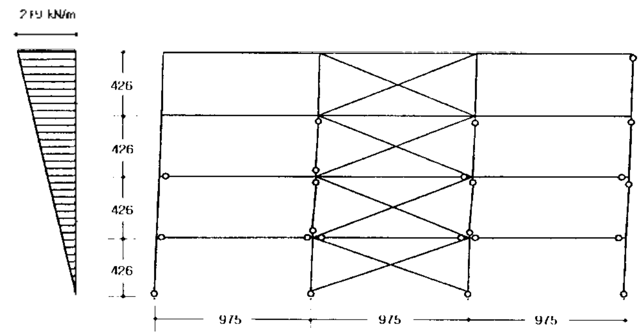

Figure 13. Upgraded Frame After the Damage

The prestressing forces, bracing design forces, and bracing cross-sectional area are shown in Table 3. Strand seven-wire cable of Grade 270 is used for prestressed members in this study.

Table 3. Bracing Design

STORY NO.	Prestressing Force (kN)	Brace Tensile Capacity (kN)	Cross-Sectional Area (cm^2)
1st	1124.1	1734.8	30.32
2nd	1245.5	1632.5	26.45
2.8	629.6	1116.5	18.07
4th	240.6	364.8	5.87

Static lateral load to collapse analysis is performed for the upgraded frame. The relationship between base shear and roof drift is shown in Figure 14. As illustrated in this figure, the initial roof drift is 0.18% due to the effect of previous lateral load. The frame stiffness is reduced at a base shear of 0.2W due to the compression members loosing effectiveness by the loss of all prestressing force.[8]

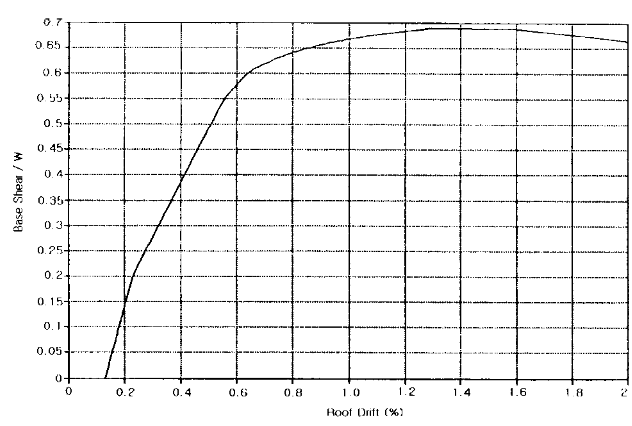

Figure 14. Base Shear vs. Roof Drift for Upgraded Frame

It is also obvious from this figure that after 0.2W base shear the lateral load is resisted by the tension members only which start to yield at base shear equal to 0.55W. The roof drift at base shear 0.2W and 0.55W are 0.22% and 0.59% respectively.

The relationship between base shear and inter-story drift is shown in Figure 15. This figure shows that the compression members in all stories have no effect, losing their prestressing force simultaneously at base shear 0.2W. Figure 15 also presents the relationship between story shear and inter-story drift. Story strength and story stiffness can be calculated from this figure. As shown, the strength of the first, second, third, and fourth story are 0.205W, 0.175W, 0.125W, and 0.053W respectively. The corresponding inter-story drifts for each story are 0.1%, 0.101% 0.102%, and 0.105%, which are approximately equal. This indicates that the story drifts of the upgraded frame under the effect of its critical lateral load are more uniform than those of the original frame.

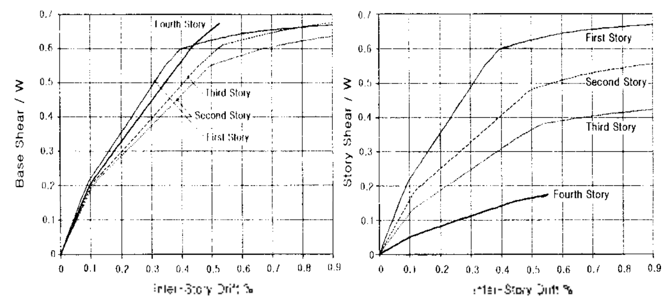

Figure 15. Base Shear vs. Inter-Story Drift and Story Shear vs. Inter-Story Drift for Upgraded Frame

The critical ground motion must be designed before dynamic nonlinear time history analysis is performed for the upgraded frame. The fundamental period of the upgraded frame (T_{up}) is 0.484 seconds located at the constant speed region of "TAFT" response spectrum as shown in Figure 16.

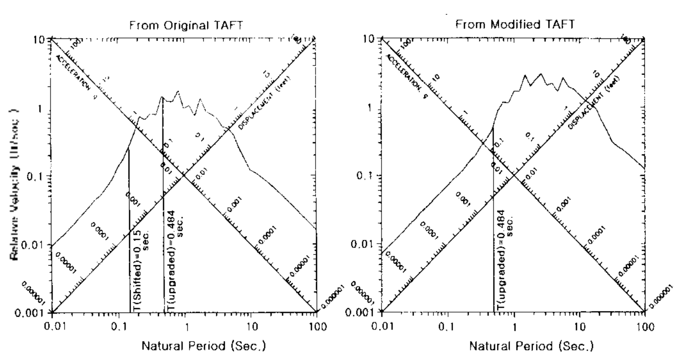

Figure 16. Response Spectrum of Original and Modified TAFT for Upgraded Building

To ensure the plastification of the upgraded frame, both time increment and maximum ground acceleration are adjusted and scaled as done for the original frame. The designed elastic response spectra for the acceleration record with $\ddot{y}_{g_{max}} = 0.14g$ and $\Delta t = 0.0645$ second are also given in Figure 16. Then, time history analysis with the modified TAFT earthquake is performed.

3.4 Results

Base shear vs. time is given in Figure 17. The maximum base shear that affects the building is 0.475W at 15.0 seconds. Figure 17 also shows roof drift vs. time. The maximum roof drift is 0.41% occurred at 15.0 seconds. Story shear vs. inter-story drift are calculated for each story and given in Figure 18. It is obvious from the figures that all stories contribute to energy dissipation. This agrees well with the results of static lateral load to collapse analysis.

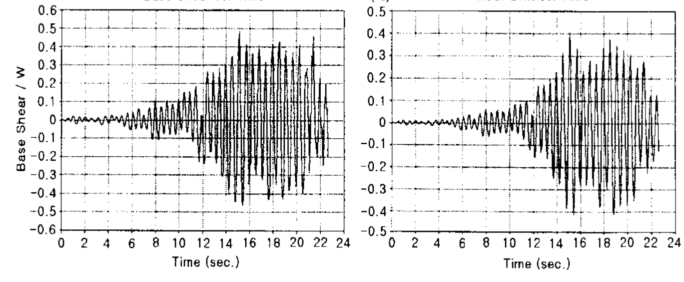

Figure 17. Base Shear vs. Time and Roof Drift vs. Time

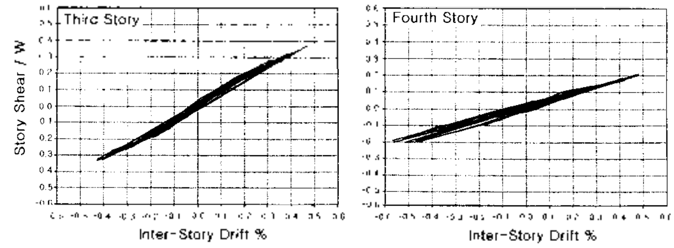

Figure 18. Story Shear vs. Inter-Story Drift for Upgraded Frame

Figures 19 shows story shear vs. time for each story. Story shears at which the compression members lose their prestressing force and the tension members yield determined from static lateral load to collapse analysis are projected onto the figure. As shown, story shear for all stories exceeds the buckling limit. Moreover no story reaches the yield limit of tension member except the fourth story that just reaches this limit. Figures 20 shows inter-story drift vs. time for each story. The inter-story drifts at which the compression members lose their prestressing force calculated from static lateral load to collapse analysis are projected onto the figures. As shown, all stories exceed the limit.

Figure 21 shows the strength distribution of the original and upgraded frames with the ideal strength distribution obtained from the critical lateral load distribution shown in Figure 13. As shown in Figure 21, the first and second stories of the original frame are subjected to shear demands above their yield shear supplies but the fourth story is subjected to shear demand below its respective yield supply. Strength distribution of upgraded frame, however, matches well with the ideal distribution.

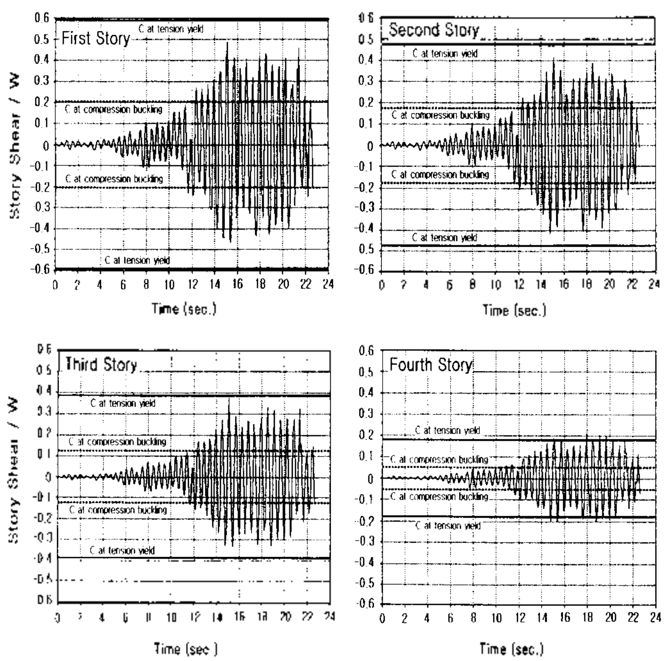

Figure 19. Story Shear vs. Time for Upgraded Frame

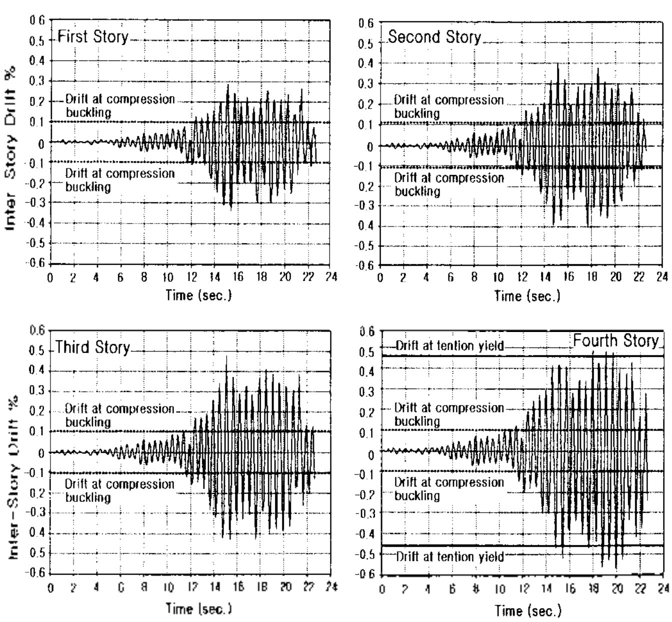

Figure 20. Inter-Story Drift vs. Time for Upgraded Frame

Stiffness distributions of the original and upgraded frames are shown in Figure 22. It shows that the proposed technique increases the building stiffness. Figure 22 also presents the stiffness of each story as a ratio of the first story stiffness. These relative values of story stiffness are compared to the ideal relative stiffness distribution in the same figure. It indicates that the proposed technique for upgraded frame improves the stiffness distribution compared to the original frame. The inter-story drifts of the upgraded frame are more uniform than those of the original frame.

Figure 23 shows the maximum story displacement for the original and upgraded frames subjected to the designed critical ground motion. The displacement values of upgraded frame are smaller and more uniform than those of the original frame.

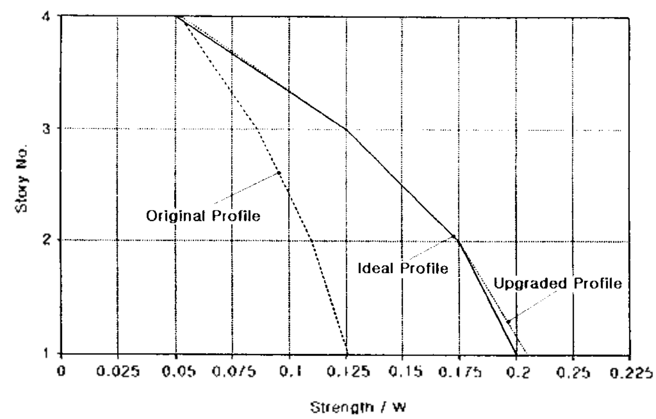

Figure 21. Strength Profile

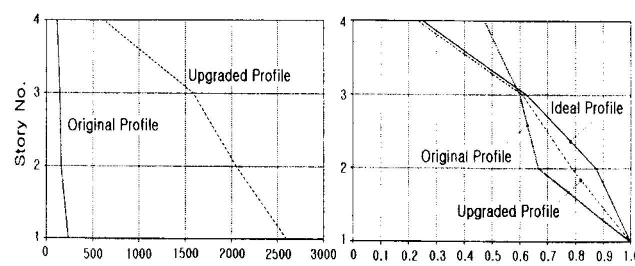

Figure 22. Stiffness and Relative Stiffness Profile

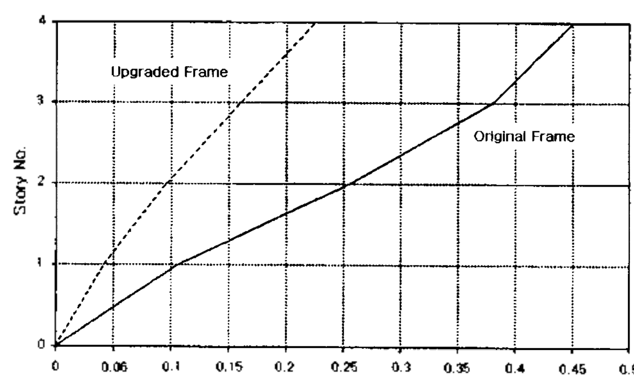

Figure 23. Strength Profile

4. CONCLUSIONS

The proposed upgrading technique can increase strength and stiffness of existing building considerably. Any upgrading technique may alter the building dynamic properties (mass and stiffness). The advantage of the proposed technique is the comparatively small increase in mass associated with the retrofitting.

The proposed technique improves the building behavior beyond yield. No sudden failure occurs in this system that enhances the ductility of the upgraded building due to the gradual change of building stiffness as the lateral load increases. On the other hand inelastic buckling of the traditional steel bracing is the main problem in achieving good hysteretic ductility.

The uniformity of damage distribution is thought to be a desirable goal of earthquake resistant design. Story stiffness can be controlled and changed from level to level to match with ideal stiffness distribution which gives equal inter-story drift by using prestressing cables as bracing system. Thus, the upgraded building has more stories that contribute to energy dissipation.

As demonstrated in this study, static lateral load to collapse analysis provides a very good alternative analysis method for time history analysis. Global performance characteristics, strength, stiffness, and ductility along with behavior characteristics such as the sequence of plastic hinge formation obtained from static lateral load to collapse analysis match well with those extracted by time history analysis.

The proposed upgrading technique provides a new tool that can be used to upgrade building with seismic deficiencies. The technique should be supplemented by experimental works.

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