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U.S. General Services Administration Progressive Collapse Design Guidelines Applied to Concrete Moment-Resisting Frame Buildings

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ABSTRACT

This paper discusses a study performed by the Portland Cement Association (PCA) to examine the application of the progressive collapse analysis and design guidelines included in the U.S. General Services Administration (GSA) publication[1] "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" to concrete frame buildings. Three reinforced cast in place concrete moment-resisting frame buildings, each 12 stories high and each with different seismic design categories (SDC), were analyzed and designed using the ETABS Nonlinear version 8.11[2] structural analyses and design software. The flexural and shear reinforcement for each building was calculated according to the strength requirements of the 2000 International Building Code[3] (2000 IBC). The seismic use group, site class definition, and spectral response accelerations were selected to represent seismic design categories A, C, and D for each building used in the study.

Each SDC requires the use of different types of concrete frames; ordinary moment frame, intermediate moment frame, and special moment frame as defined in the 2000 IBC[3]. However, for this study, only strength requirements are evaluated and compared for the three frames. Reinforcement detailing and ductility requirements of the three types of frames are not evaluated in this study.

The study showed that the building columns in each of the three seismic categories do not require additional reinforcement to prevent progressive collapse. Also, the study showed that the beams proportioned and reinforced according to the strength requirements for the most severe seismic category, SDC D, have sufficient strength to resist progressive collapse. The perimeter beams designed to satisfy the strength requirements for SDC C need additional reinforcing only for the beams in the lower four stories. The perimeter beams designed for SDC A need additional flexural reinforcement in the stories one through eleven in order to prevent progressive collapse. The cost of the additional reinforcement required to satisfy the GSA criteria is quite nominal.

INTRODUCTION

Following the Alfred P. Murrah Federal Building bombing on April 19, 1995, President Clinton issued an executive order to establish construction standards for federal buildings subject to terrorist attack. The Interagency Security Committee (ISC) was organized to respond to the executive order and developed the "Security Design Criteria for New Federal Office Buildings and Major Modernization Projects" for federal buildings of which the latest version was published in 2001 for <u>official use only</u>. "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects[1]" was published in November 2000 by the General Services Administration (GSA) to meet the progressive collapse requirements of the ISC criteria.

The GSA publication is a threat independent method to reduce the potential for progressive collapse. The application of the guideline is not an explicit part of blast design and its use is limited to buildings without unusual structural configurations. The method discussed in the GSA publication is normally used for buildings 10 stories above grade and less, but can be applied to taller buildings.

Early in 2003 the Portland Cement Association (PCA) initiated a study of the use of the GSA method for analysis and design against progressive collapse as applied to a 12-story concrete frame building. It is assumed that the building fits in a category of building, which is not exempted from the progressive collapse analysis. The PCA study is limited to an evaluation of concrete building structures from a strength perspective, and uses a static linear elastic analysis of a three dimensional model of the structure using the ETABS[2] structural analysis and design program.

To resist progressive collapse, in addition to strength requirements, the concrete building structure reinforcement must be detailed in such a way as to behave in a ductile fashion. However, reinforcement detailing for ductility is not discussed in this paper.

GSA PROGRESSIVE COLLAPSE ANALYSIS AND DESIGN CRITERIA

The GSA criteria for new and existing structures which do not qualify for exemption from consideration of progressive collapse, contains guidelines for the analyses of "typical" and "atypical" structural systems. A typical structure is defined as having relatively simple layout with no unusual structural configurations. Only typical structures are discussed in this paper. To determine the potential of progressive collapse for a typical structure, designers can perform structural analyses in which the instantaneous loss of one of the following first floor columns at a time is assumed:

1. An exterior column near the middle of the long side of the building.
2. An exterior column near the middle of the short side of the building.
3. A column located at the corner of the building.
4. A column interior to the perimeter column lines for facilities that have underground parking and/or uncontrolled public ground floor areas.

The GSA criterion utilizes the alternate path method to ensure that progressive collapse does not occur. Designers may use linear elastic static analyses or non-linear dynamic analysis to check structural members in the alternate path structure, i.e. the structure after removal of a single column. For this paper only linear elastic static analysis is discussed. For static analysis purposes the following gravity load is applied to each structural member of the alternate path structure:

Load = 2(DL + 0.25LL)

Where,

DL = Dead load
LL = Floor Live load

The Demand Capacity Ratio (DCR) of each primary and secondary member of the alternate path structure is calculated from the following equation:

$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

Where,

Q_{UD} = Acting force determined in the structural element.
Q_{CE} = Expected ultimate, un-factored capacity of the structural element.

To determine the ultimate capacity of the structural component, a material strength increase of 25% is allowed for concrete and reinforcing steel.

In order to prevent collapse of the alternate path structure, the DCR values for each structural element must be less than or equal to the following:

DCR $\leq$ 2.0 for typical structural configuration
DCR $\leq$ 1.5 for atypical structural configuration

Structural elements that have DCR values exceeding the above limits will not have additional capacity for effectively redistributing loads, are considered failed, and can, therefore, result in collapse of the entire structure. The above DCR methodology is based on NEHRP[4] Guidelines for the Seismic Rehabilitation of Buildings issued by FEMA in 1997.

THE BUILDING MODEL

The building used in the study is a twelve-story cast-in-place reinforced concrete moment-resisting frame structure. The plan of the building and the bay dimensions are uniform as shown in Figure 1 and, therefore, the structural members are considered typical. The floor live load is 50 psf and the superimposed dead load is 30 psf. Three building structures were designed, one for each of three different seismic design categories (A, C and D). The structural design is in accordance with the 2000 International Building Code[3] seismic design provisions and the seismic design parameters for the three designs are shown in Table 1. The building structure is modeled as a three-dimensional structure and includes consideration of P-Delta effects. Using the computer program ETABS Nonlinear version 8.11[2], member forces and the required reinforcement to resist normal dead, live, wind, and seismic loads were determined. The design of each critical member for each of the three structures is shown in Table 2.

PROGRESSIVE COLLAPSE ANALYSIS RESULTS

Following the design of each of the three structures (A, C, and D) for dead, live, wind, and seismic loads, first story columns were removed at each of the four locations for each of the three buildings as specified by the GSA criteria. The specified GSA load combination was applied and the demand forces were calculated for each member again using the ETABS program.

In order to calculate the demand capacity ratio for each member, the section ultimate capacity was recalculated considering the actual area of steel provided in the design. Also, the material strength and strength reduction factors were set equal to one, as specified by the GSA provisions, and the computer program PCACOL[5] was used to calculate the ultimate capacity. For each beam the demand capacity ratio was calculated for the top and bottom reinforcement for each section along the beam in addition to the demand capacity ratio for the shear. Spreadsheets were developed to analyze the results from the computer program ETABS. For each beam the maximum DCR was determined. For each column the demand capacity ratio was calculated directly using the results from ETABS.

The demand capacity ratios (DCR) for the first story columns for buildings A, C, and D are summarized in Table 3. The table shows that the demand capacity ratios for the remaining columns (un-removed) are below the GSA limit of DCR = 2 for the three moment resisting frame buildings. The flexural and shear demand capacity ratios for the beams in the vicinity of the eliminated columns for the three buildings in the study are summarized in Tables 4 and 5. The following is a discussion of the analysis results for flexural and shear demand capacity ratio calculations and progressive collapse potential for the three buildings subject to removal of first floor columns.

EXTERIOR COLUMN NEAR THE MIDDLE OF THE LONG SIDE OF THE BUILDING REMOVED

The removal of exterior column C9 near the middle of the long side of the building caused moment reversal in the beams intersecting at the removed support, beams B2, B3, and B27. The flexural resistance in these beams depends on the bottom reinforcement provided at the support. Figure 2 shows the distribution of the moment and shear after removal of the column for the two column lines intersecting at the removed support. The figure shows that the values of the reversed moment diminish in the upper floors and for beams away from the vicinity of the removed column. A comparison of the flexural demand capacity ratios for the three buildings studied for the beams in the vicinity of the removed column at beams B2, B3, and B27 for the twelve stories is presented in Figure 3. The figure shows that the perimeter beams (B2 and B3) are more critical for resisting progressive collapse than beam B27. The following is a summary of the analysis results:

1. For the building designed for SDC D
    a. Beams B2, B3, and B27 in all levels have flexural demand capacity ratios (DCR's) less than the GSA limit of 2 and, therefore, do not need additional reinforcement to resist progressive collapse.
    b. All other beams do not need additional reinforcement.

2. For the building designed for SDC C
    a. Beams B2 and B3 in levels 1, 2, 3 and 4 have flexural DCR's greater than 2.0 and therefore need additional reinforcement to prevent progressive collapse.
    b. All other beams do not need additional reinforcement.

3. For the building designed for SDC A
    a. Beams B2 and B3 in levels 1 through 11 have flexural DCR's greater than 2.0 and therefore need additional reinforcement to prevent progressive collapse.
    b. All other beams do not need additional reinforcement.

The shear DCR's are shown in Table 5 for critical beams B2, B3, and B27. The table shows that all the SDR's are below the GSA limit of 2 and therefore additional shear reinforcement is not needed to prevent progressive collapse.

Columns C13, C12, and C16 are symmetrical to the removed column C9 and, therefore, would also be removed one at a time. Consequently, beams B4, B17, B18, and B19 would have DCR's equal to those for beams B2 and B3 and need additional reinforcement as discussed in 2 and 3 above.

EXTERIOR COLUMN LOCATED NEAR THE MIDDLE OF THE SHORT SIDE OF THE BUILDING REMOVED

The removal of exterior column C2 near the middle of the short side of the building caused moment reversal in the beams intersecting at the removed support, beams B6, B21, and B22. The beam flexural resistance depends on the bottom reinforcement provided at the support. Figure 4 shows the distribution of the moment and shear after the removal of the column for ordinary moment frame for the two column lines intersecting at the removed support. A comparison of the flexural demand capacity ratios for the three buildings studied for the beams in the vicinity of the removed column at beams B6, B21, and B22 for the twelve stories is presented in Figure 5. The following is a summary of the analysis results:

1. For the building designed for SDC D
    a. Beams B6, B21, and B22 in all levels have flexural demand capacity ratios (DCR's) less than the GSA limit of 2 and therefore do not need additional reinforcement to resist progressive collapse.
    b. All other beams do not need additional reinforcement.

2. For the building designed for SDC C
    a. Beams B21 and B22 in levels 1, 2, and 3 have flexural DCR's greater than 2.0 and therefore need additional reinforcement to prevent progressive collapse.
    b. All other beams do not need additional reinforcement.

3. For the building designed for SDC A
    a. Beams B21 and B22 in levels 1 through 11 have flexural DCR's greater than 2.0 and therefore need additional reinforcement to prevent progressive collapse.
    b. All other beams do not need additional reinforcement.

The shear DCR's are shown in Table 5 for beams B6, B21, and B22. The table shows that all the SDR's are below the GSA limit of 2 and therefore additional shear reinforcement is not needed to prevent progressive collapse.

Columns C3, C22, and C23 are symmetrical to the removed column C2 and, therefore, would also be removed one at a time. Consequently, beams B23, B36, B37, and B38 would have DCR's equal to those for beams B21 and B22 and would need additional reinforcement as discussed in 2 and 3 immediately above.

COLUMN LOCATED AT THE CORNER OF THE BUILDING REMOVED

The removal of the corner column C1 caused moment reversal in the beams intersecting at the removed support, beams B1 and B21. Figure 6 shows the distribution of the moment and shear after the removal of the column for the two column lines intersecting at the removed support. A comparison of the flexural demand capacity ratios for the three buildings studied for the beams in the vicinity of the removed column at beams B1 and B21 for the twelve stories is presented in Figure 7. The following is a summary of the analysis results:

1. For the building designed for SDC D

    a. Beams B1 and B21 in all levels have flexural demand capacity ratios (DCR's) less than the GSA limit of 2 and therefore do not need additional reinforcement to resist progressive collapse.
    b. All other beams do not need additional reinforcement.

2. For the building designed for SDC C
    a. Beams B1 and B21 in levels 1 and2 have flexural DCR's greater than 2.0 and therefore need additional reinforcement to prevent progressive collapse.
    b. All other beams do not need additional reinforcement.

3. For the building designed for SDC A
    a. Beam B1 in levels 1 through 8 and beam B21 in levels 1 through 4 have flexural DCR's greater than 2.0 and therefore need additional reinforcement to prevent progressive collapse.
    b. All other beams do not need additional reinforcement.

The shear DCR's are shown in Table 5 for beams B1 and B21. The table shows that all the SDR's are below the GSA limit of 2 and, therefore, additional shear reinforcement is not needed to prevent progressive collapse.

Columns C4, C21, and C24 are symmetrical to the removed column C1 and, therefore, would also be removed one at a time. Consequently, beams B23, B16, B20, B9, B36 and B38 would have DCR's equal to those for beams B1 and B21 and would need additional reinforcement as discussed in 2 and 3 immediately above.

INTERIOR COLUMN REMOVED

The removal of interior column C6 caused moment reversal in the beams intersecting at the removed support in beams B6, B7, B24, and B25. Figure 8 shows the distribution of the moment and shear after the removal of the column for the two column lines intersecting at the removed support. A comparison of the flexural demand capacity ratios for the three buildings studied for the beams in the vicinity of the removed column at beams B6, B7, B24, and B25 for the twelve stories is presented in Figure 9. The following is a summary of the analysis results:

1. For the building designed for SDC D
    a. Beams B6, B7, B24, and B25 in all levels have flexural demand capacity ratios (DCR's) less than the GSA limit of 2 and, therefore, do not need additional reinforcement to resist progressive collapse.
    b. All other beams do not need additional reinforcement.

2. For the building designed for SDC C
    a. Beams B6, B7, B24, and B25 in all levels have flexural demand capacity ratios (DCR's) less than the GSA limit of 2 and, therefore, do not need additional reinforcement to resist progressive collapse.
    b. All other beams do not need additional reinforcement.

3. For the building designed for SDC A

a. Beams B6 and B24 in levels 1 through 3 and beams B7 and B25 in levels 1 through 2 have flexural DCR's greater than 2.0 and, therefore, need additional reinforcement to prevent progressive collapse.
b. All other beams do not need additional reinforcement.

The shear DCR's are shown in Table 5 for beams B6, B7, B24, and B25. The table shows that all the SDR's are below the GSA limit of 2 and therefore additional shear reinforcement is not needed to prevent progressive collapse.

Columns C7, C10, C11, C14, C15, C18, and C19 are symmetrical to the removed column C6 and, therefore, would also be removed one at a time. Consequently, beams B8, B9, B10, B11, B12, B13, B14, and B15 would have DCR's equal to those for beams B6, B7, B24, and B25 and would need additional reinforcement as discussed in 3 immediately above.

ESTIMATE OF ADDITIONAL REINFORCEMENT REQUIRED TO SATISFY THE GSA CRITERIA

For concrete buildings designed for SDC A, 235 beams out of a total of 456 beams need additional reinforcement to satisfy the GSA limit of DCR = 2. A brief analysis shows that in the worst case, i.e. beams with the largest value of DCR, the reinforcement will have to be doubled. It is estimated that the total amount of additional reinforcement for the 12 story building is 15 tons. At an average cost of labor and material of $775 per ton, the cost of reinforcement to satisfy the strength requirements of the GSA criteria will only be $12,000. If the total construction cost of the 104,000 square foot building is approximately $9,000,000, the cost of satisfying the GSA criteria is only a small fraction of the total cost.

CONCLUSIONS

The objective of this study was to examine the application of the U.S. General Services Administration (GSA) progressive collapse analysis and design guidelines as applied to moment resisting frame reinforced concrete buildings. The main parameters studied were the axial load, flexure, and shear reinforcement required for the moment resisting concrete framed buildings designed for seismic design categories A, C, and D and for column removal and DCR per the GSA criteria. The building structures were designed in accordance with the 2000 International Building Code. Only strength requirements were evaluated and compared for the three buildings.

Conclusions for the 12-story building studied are as follows:

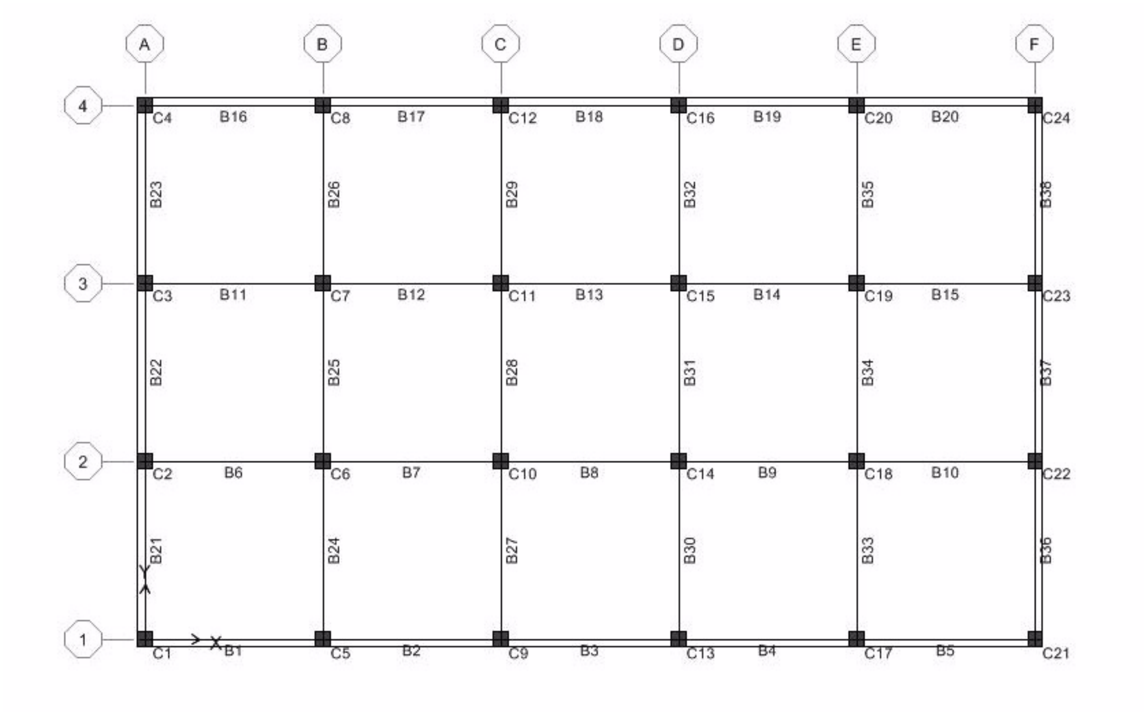
1. Since shear DCR's are less than 2 in all cases studied, shear reinforcement is adequate and does not have to be increased to meet the GSA criteria for buildings designed for SDC A, C, or D.
2. Since column DCR'sare less than 2 in all cases studied, the columns are adequate and do not need to be changed to meet the GSA criteria for buildings designed for SDC A, C, or D.
3. For the building designed for SDC C, 55 beams out of a total of 456 beams need additional reinforcement to satisfy the GSA criteria.
4. For the building designed for SDC A, 235 beams out of a total of 456 beams need additional reinforcement to satisfy the GSA criteria.

5. For concrete buildings designed for SDC C and D, progressive collapse prevention per the GSA criteria can be achieved with only a very minor increase in cost.
6. For concrete buildings designed for SDC A, progressive collapse prevention per the GSA criteria can be achieved with only a small increase in cost ($12,000).

Applying the GSA criteria to prevent progressive collapse for concrete buildings can be accomplished by the structural engineer using readily available software and for little additional construction cost.

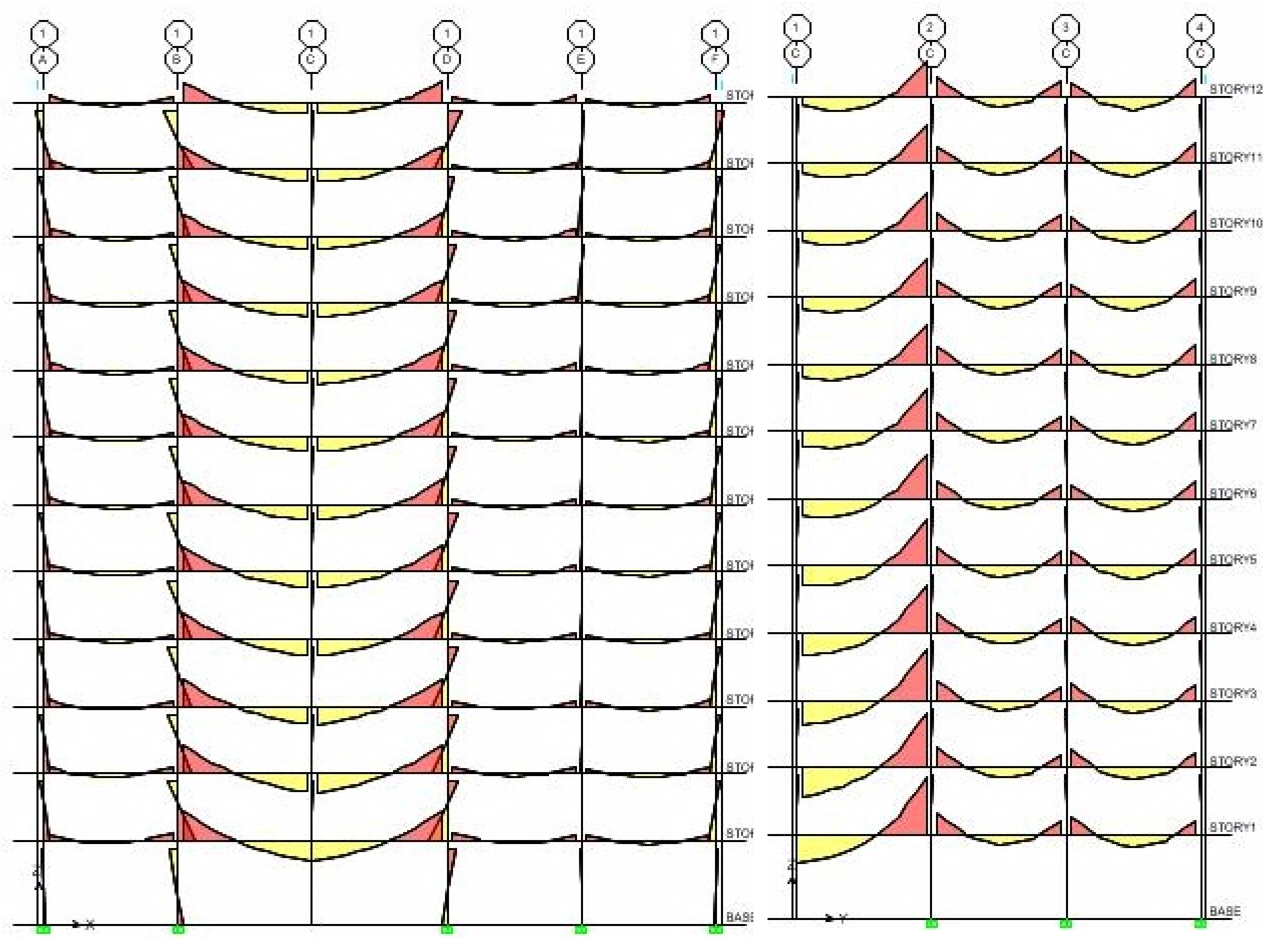
REFERENCES:

1. *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*, U.S. General Services Administration, November 2000.
2. *ETABS Nonlinear Version 8.11*, *Extended 3-D Analysis of Building Systems*, Computers and Structures, Inc. Berkeley, California.
3. *International Building Code*, International Code Council, 2000
4. *NEHARP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA, 1997
5. *PCACOL Version 3.0b*, *Design and Investigation of Column Sections*, PCA 2000
6. *Seismic and Wind Design of Concrete Buildings*, S.K. Ghosh and David Fanella, 2003



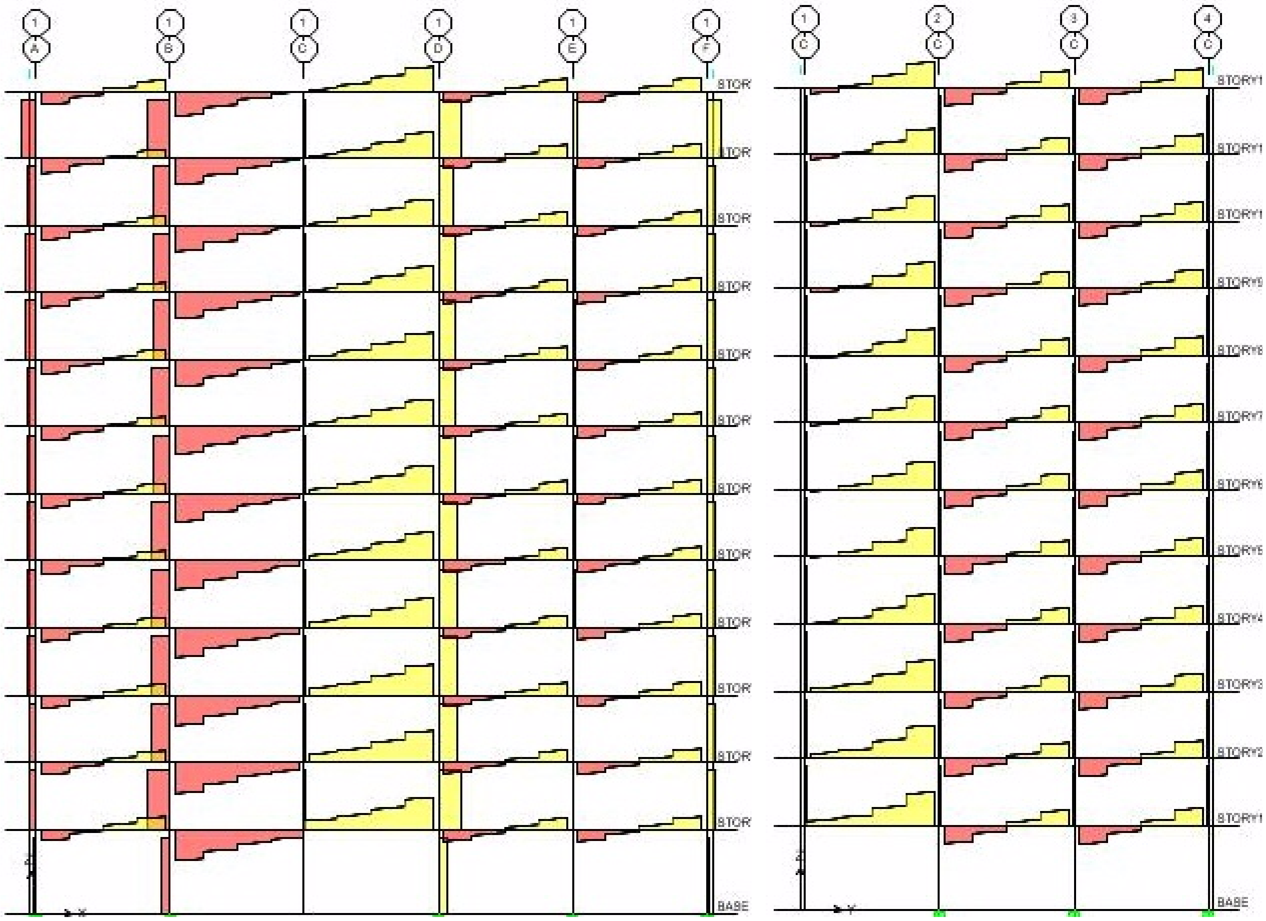
Number of stories = 12
First story height = 15 feet
Typical story height = 12 feet

Figure 1 Building Plan



Column line 1 Column line C

Figure 2 a – Bending moment due to GSA load combination (SDC A) after removing column C9



Column line 1 Column line C

Figure 2 b – Shear force due to GSA load combination (SDC A) after removing column C9

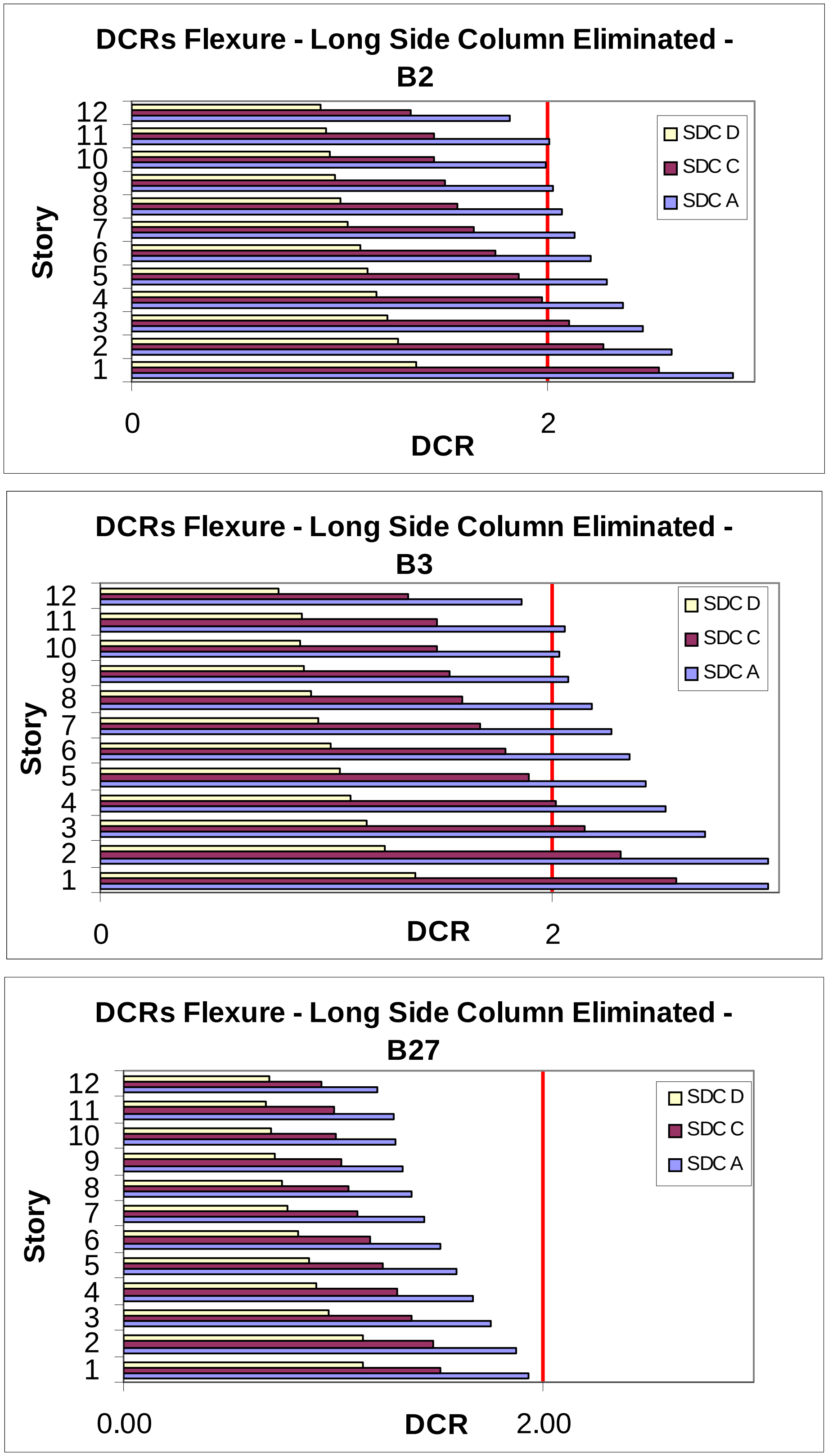
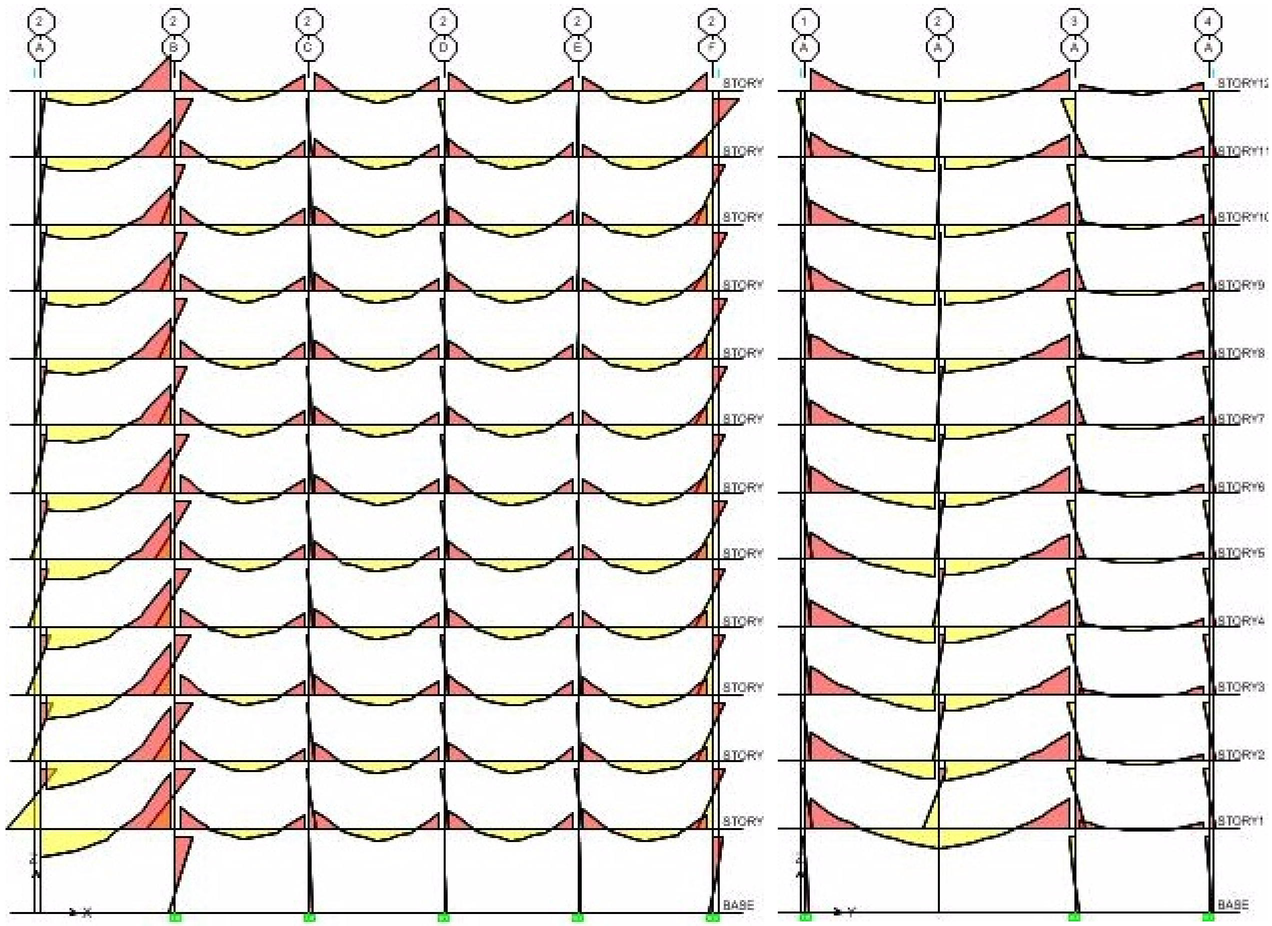
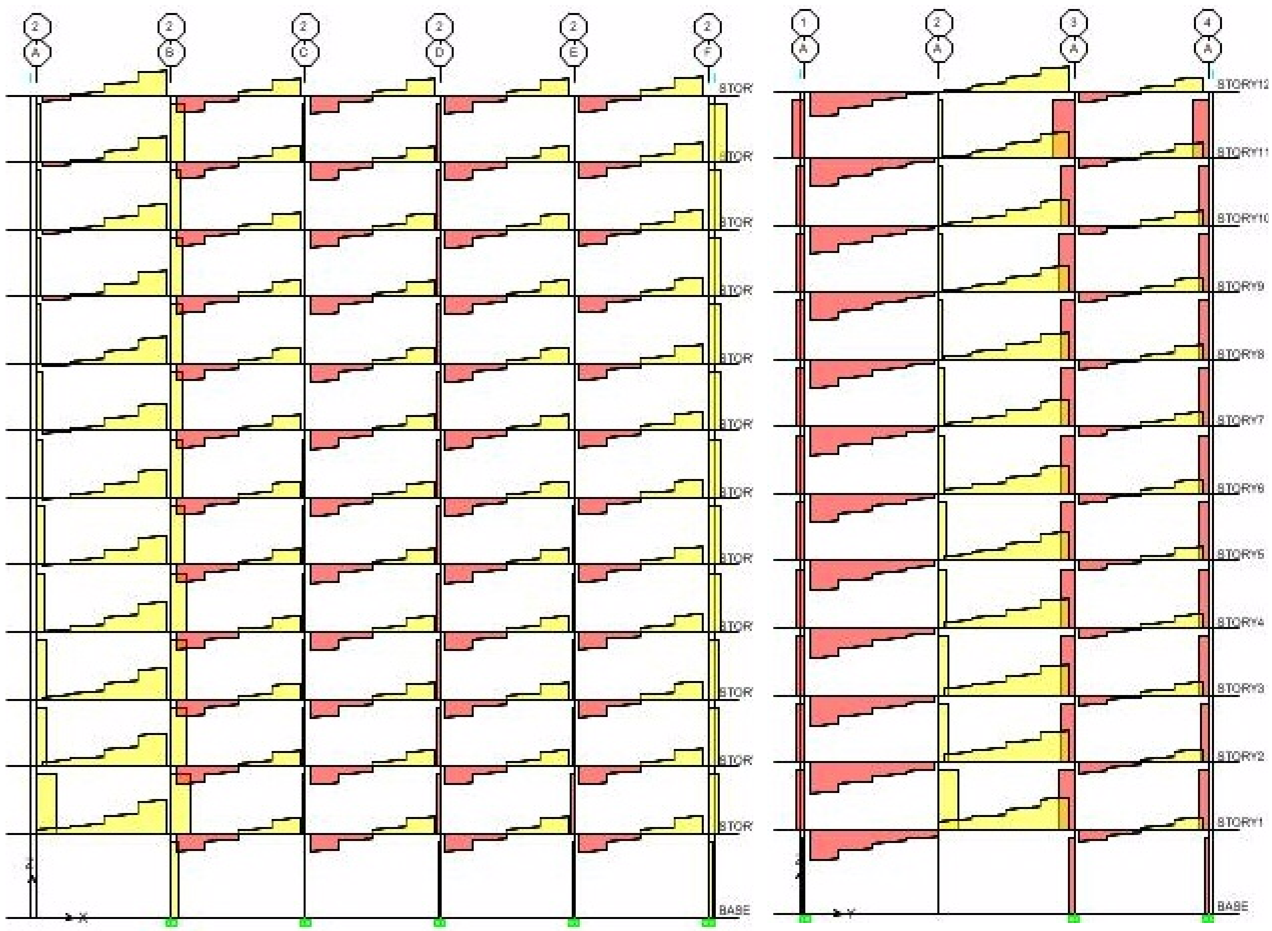


Figure 3 Flexural DCR for beams in the vicinity of the removed column (Exterior near the middle of the long side)



Column line 2 Column line A

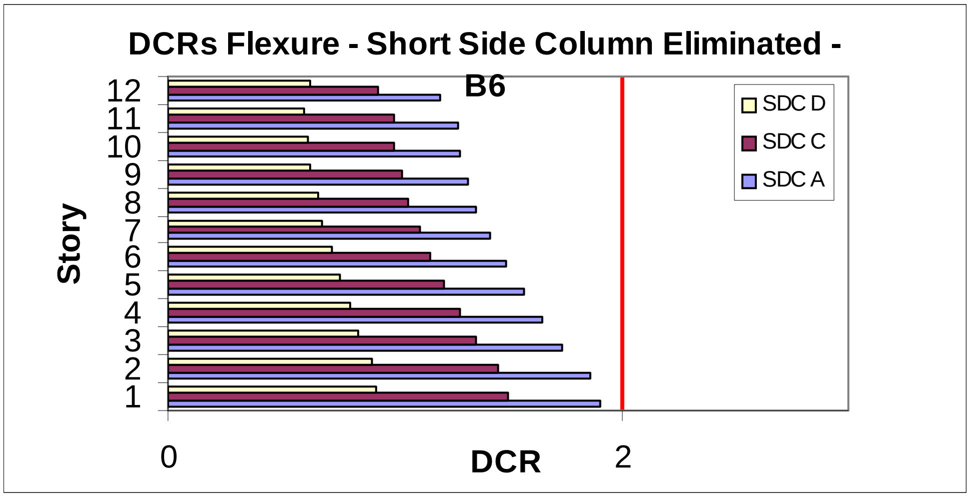
Figure 4 a – Bending moment due to GSA load combination (SDC A) after removing column C2

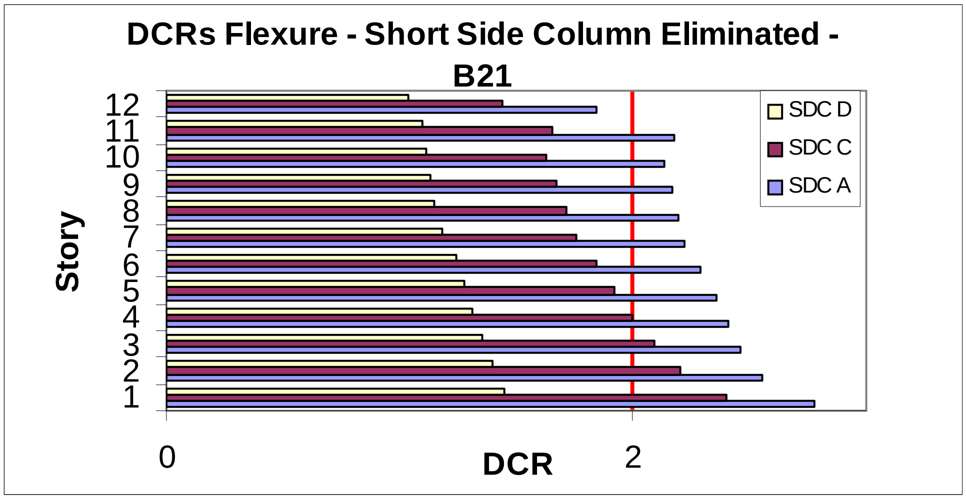


Column line 2

Column line A

Figure 4 b – Shear force due to GSA load combination (SDC A) after removing column C2





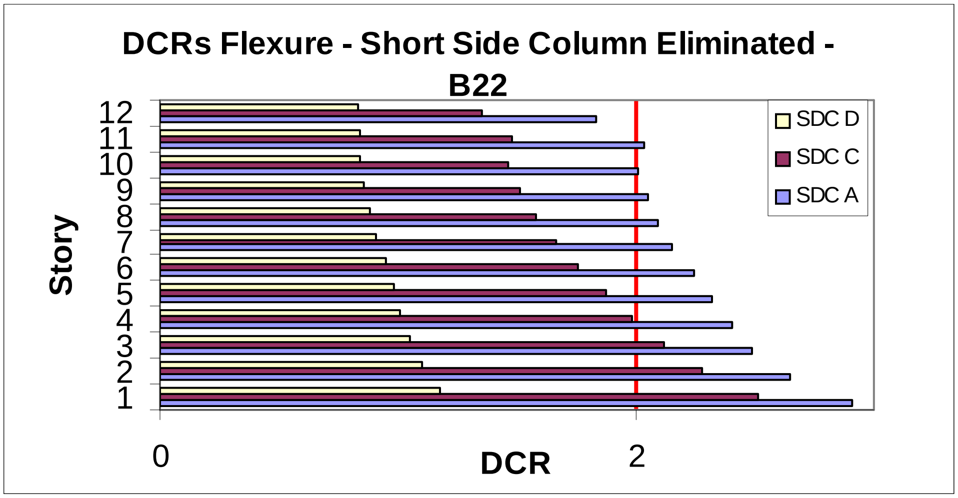
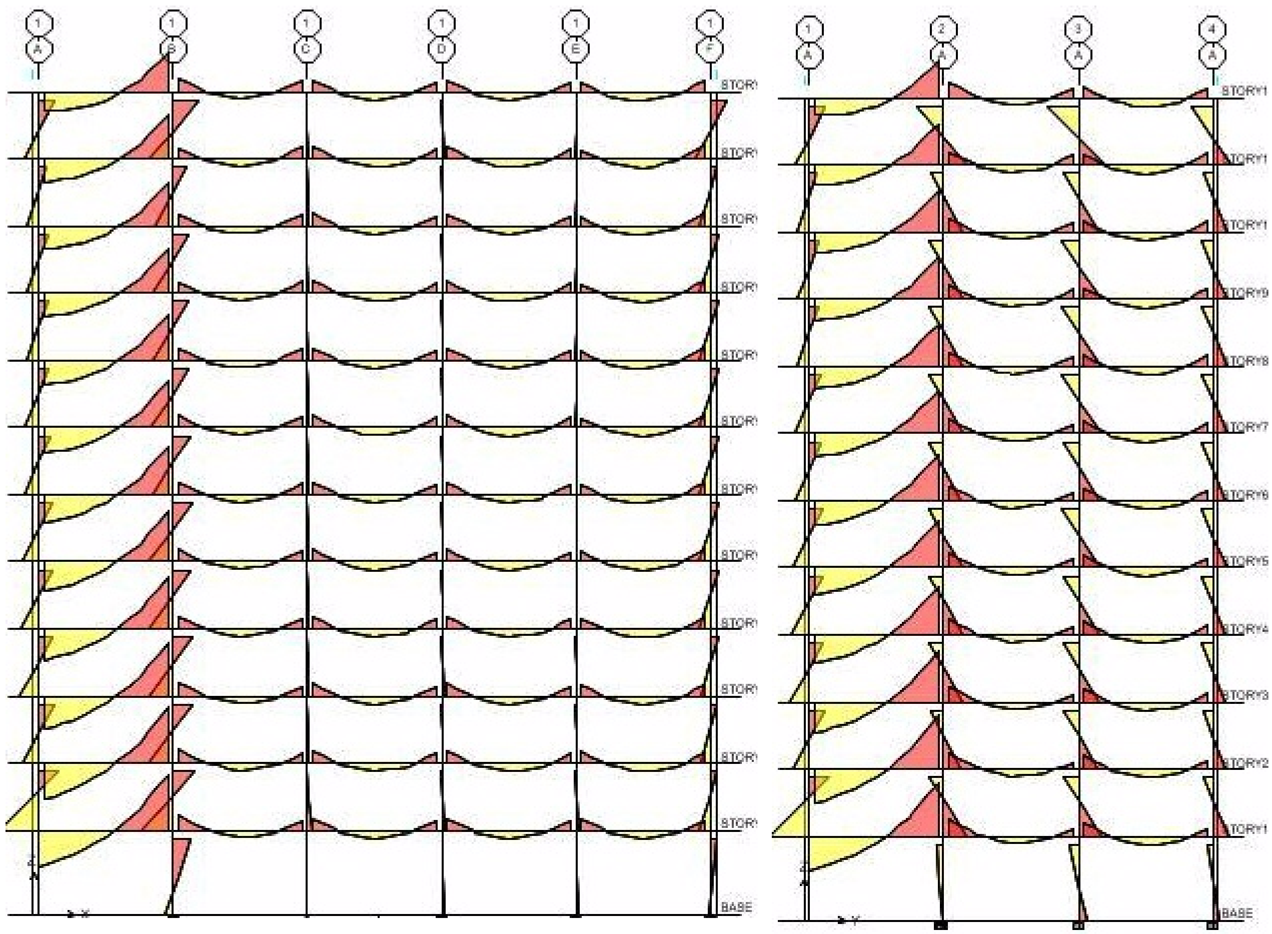
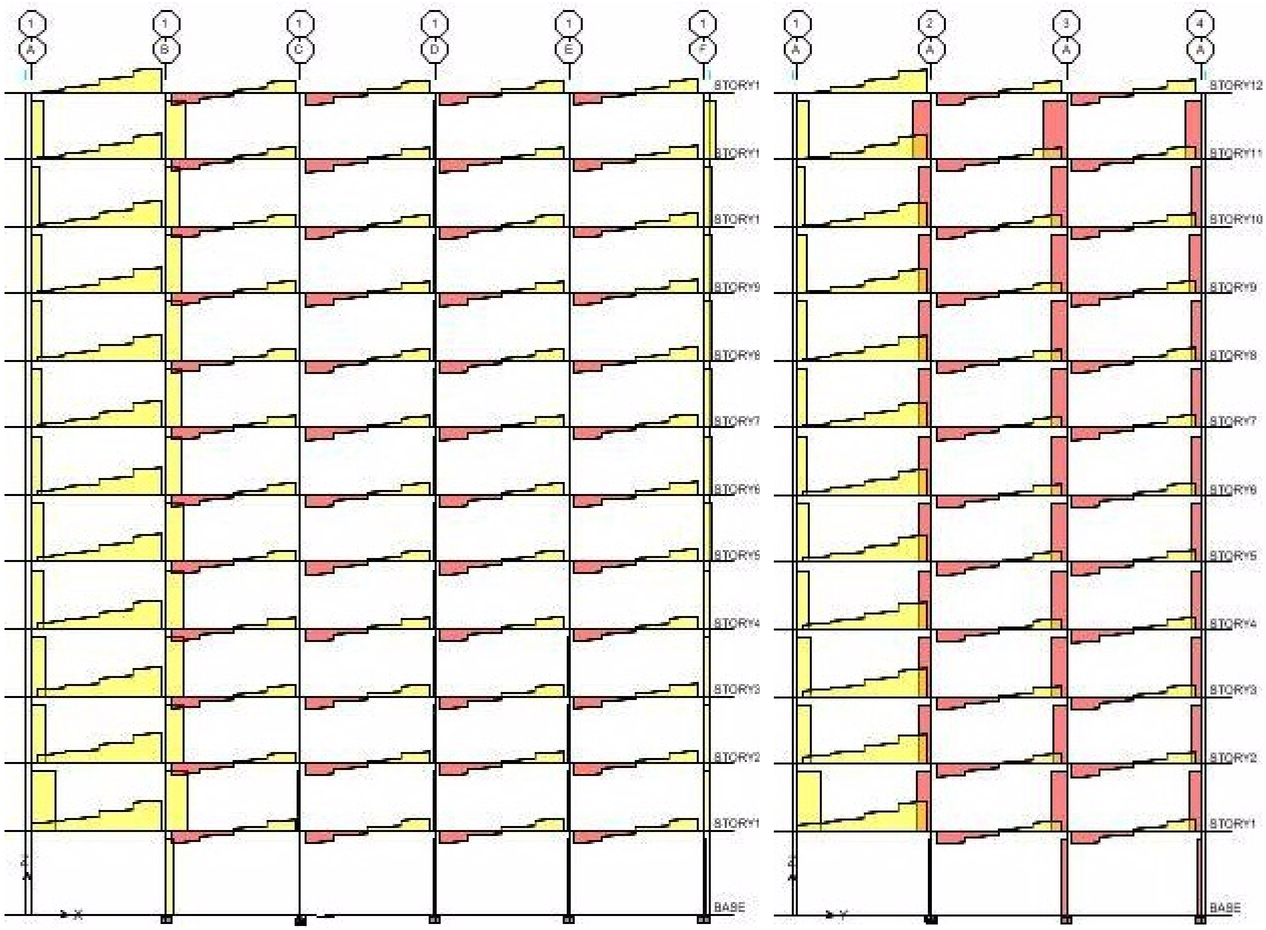


Figure 5 Flexural DCR for beams in the vicinity of the removed column (Exterior near the middle of the short side)



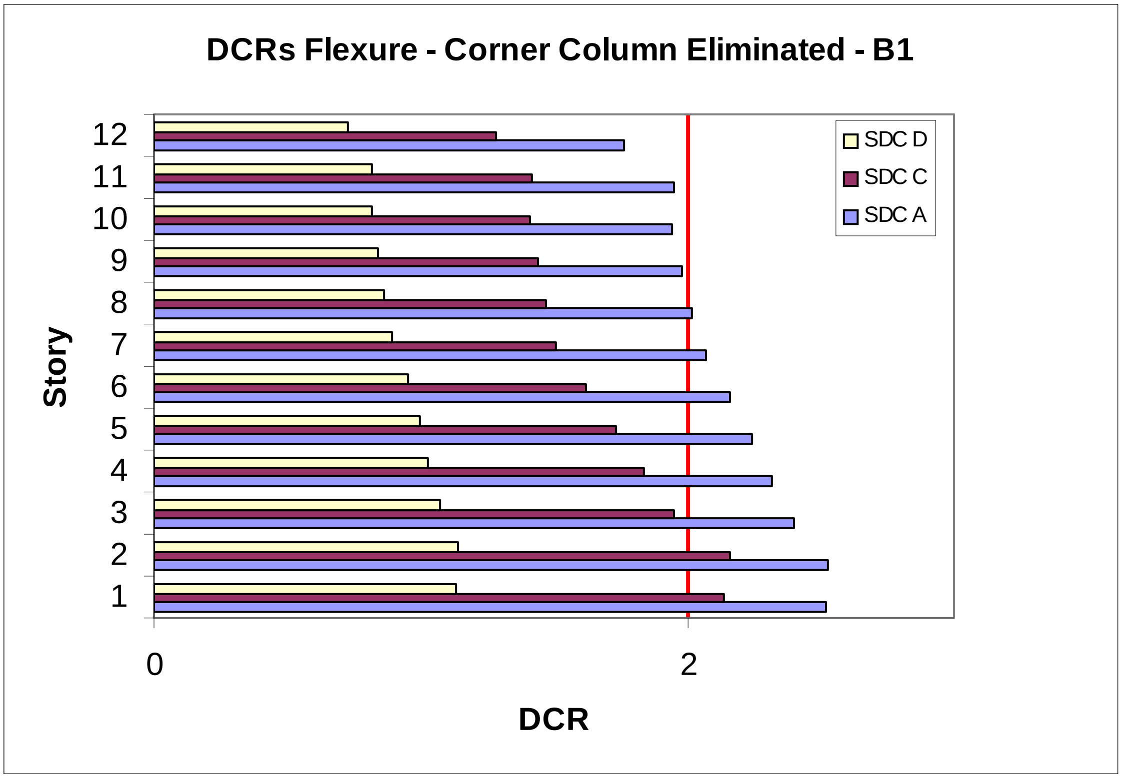
Column line 1 Column line A

Figure 6 a – Bending moment due to GSA load combination (SDC A) after removing column C1



Column line 1 Column line A

Figure 6 b – Shear force due to GSA load combination (SDC A) after removing column C1



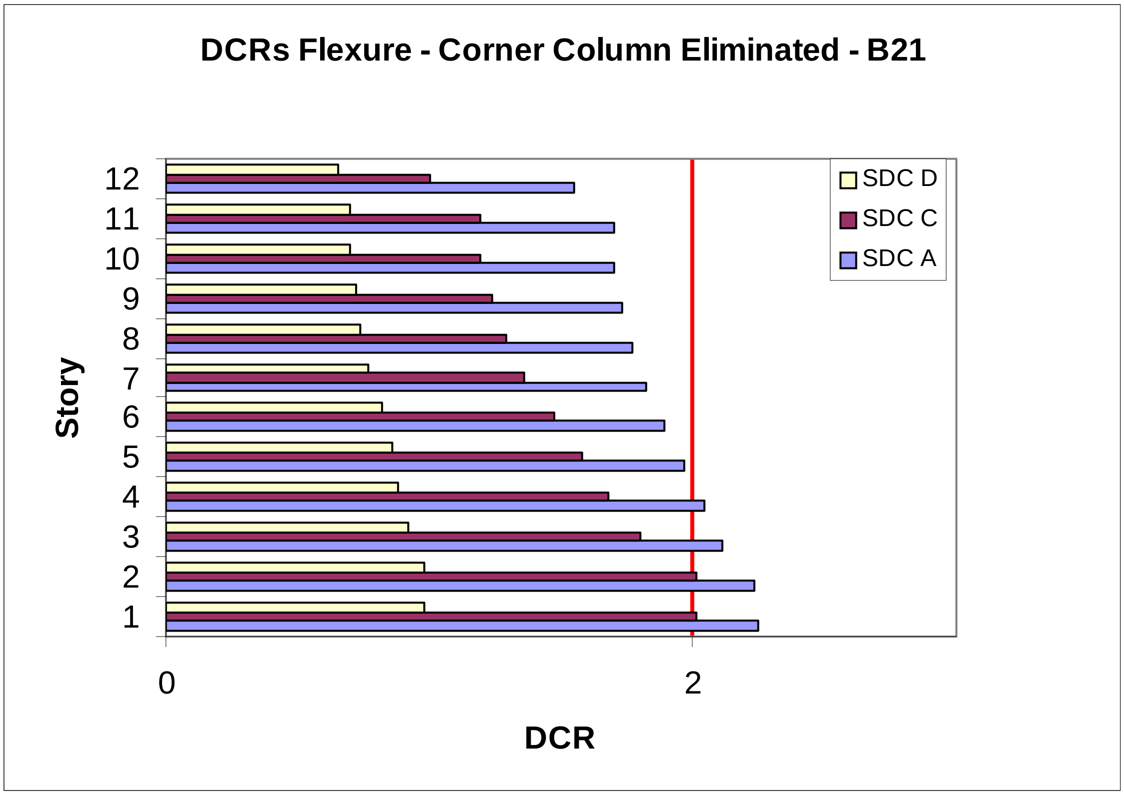
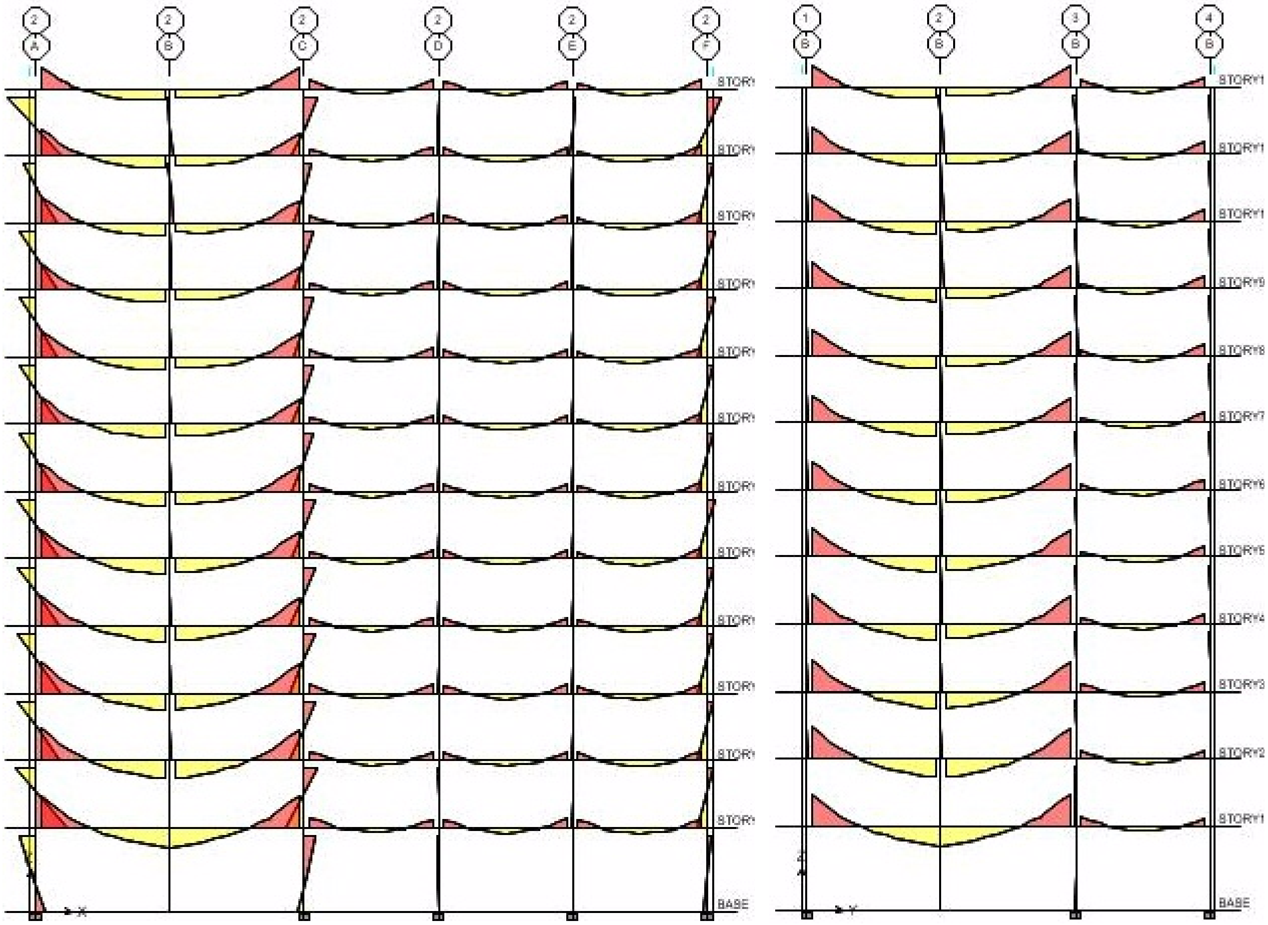


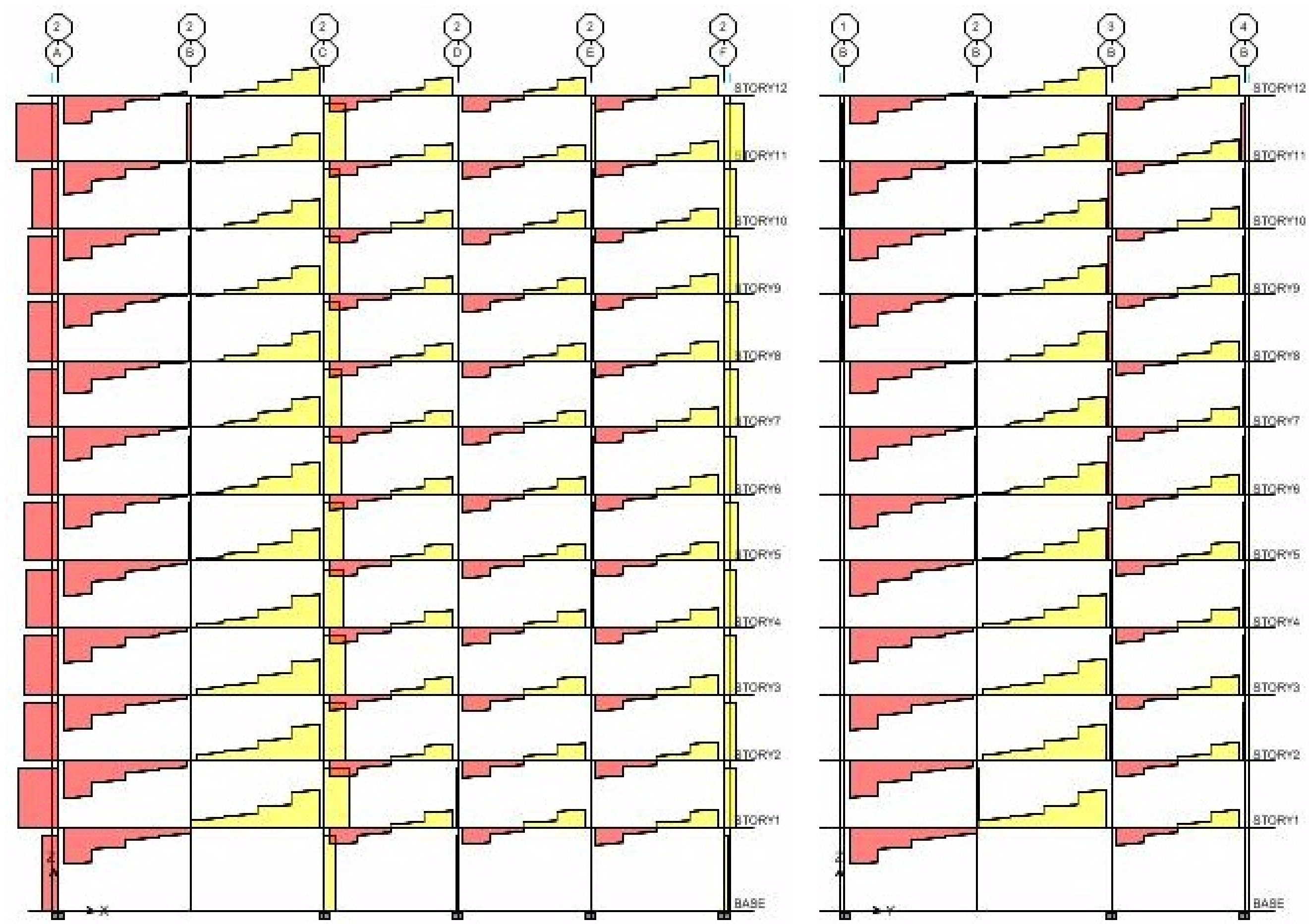
Figure 7 - Flexural DCR for beams in the vicinity of the removed column (Corner column



Column line 2

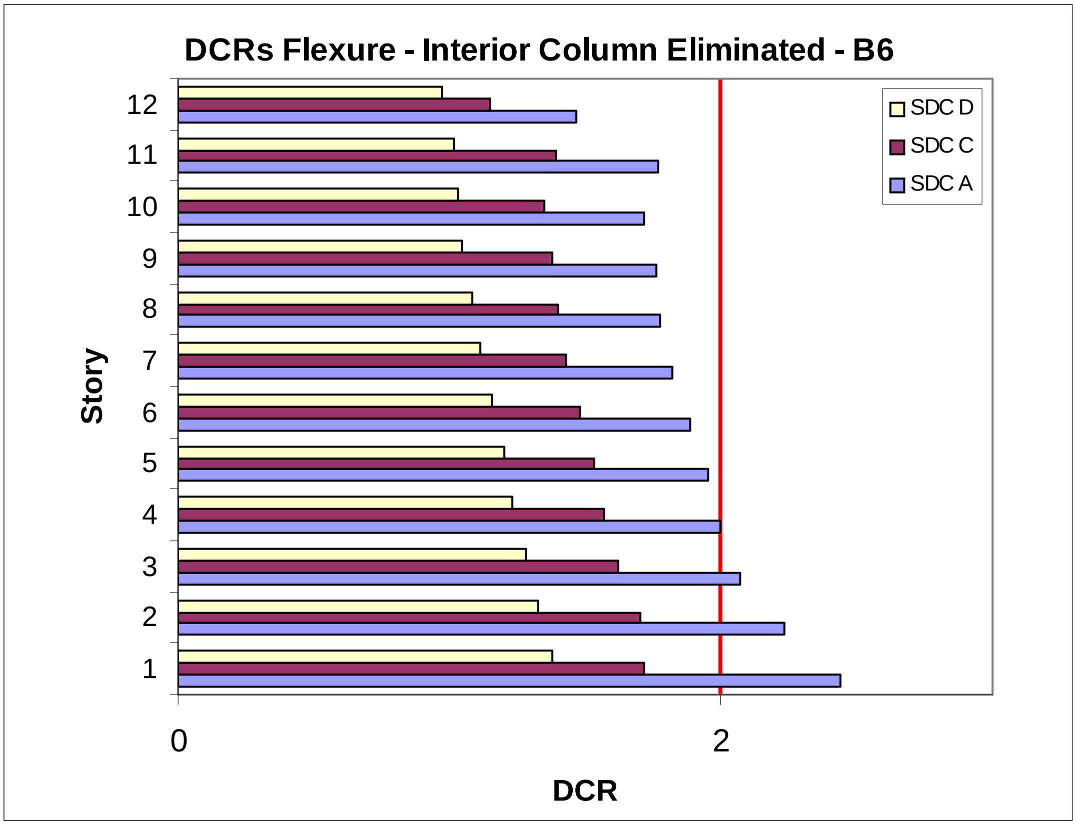
Column line B

Figure 8 a – Bending moment due to GSA load combination (SDC A) after removing column C6



Column line 2 Column line B

Figure 8 a – Shear force due to GSA load combination (SDC A) after removing column C6



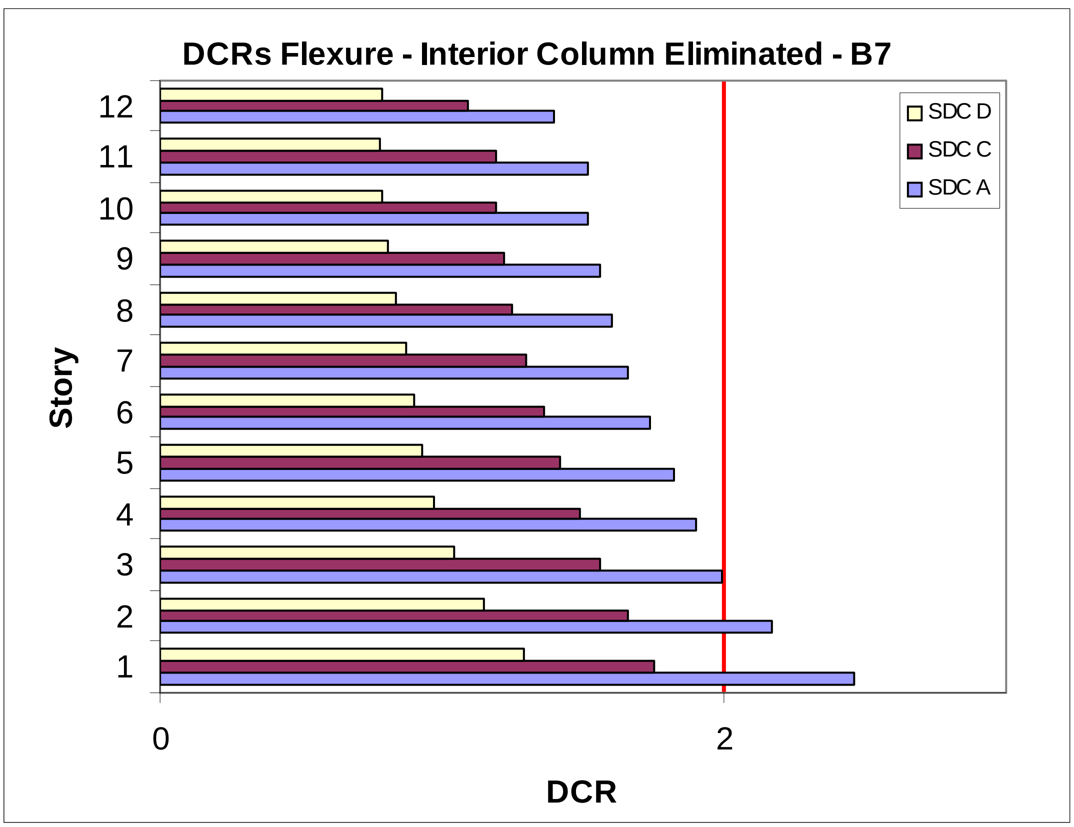
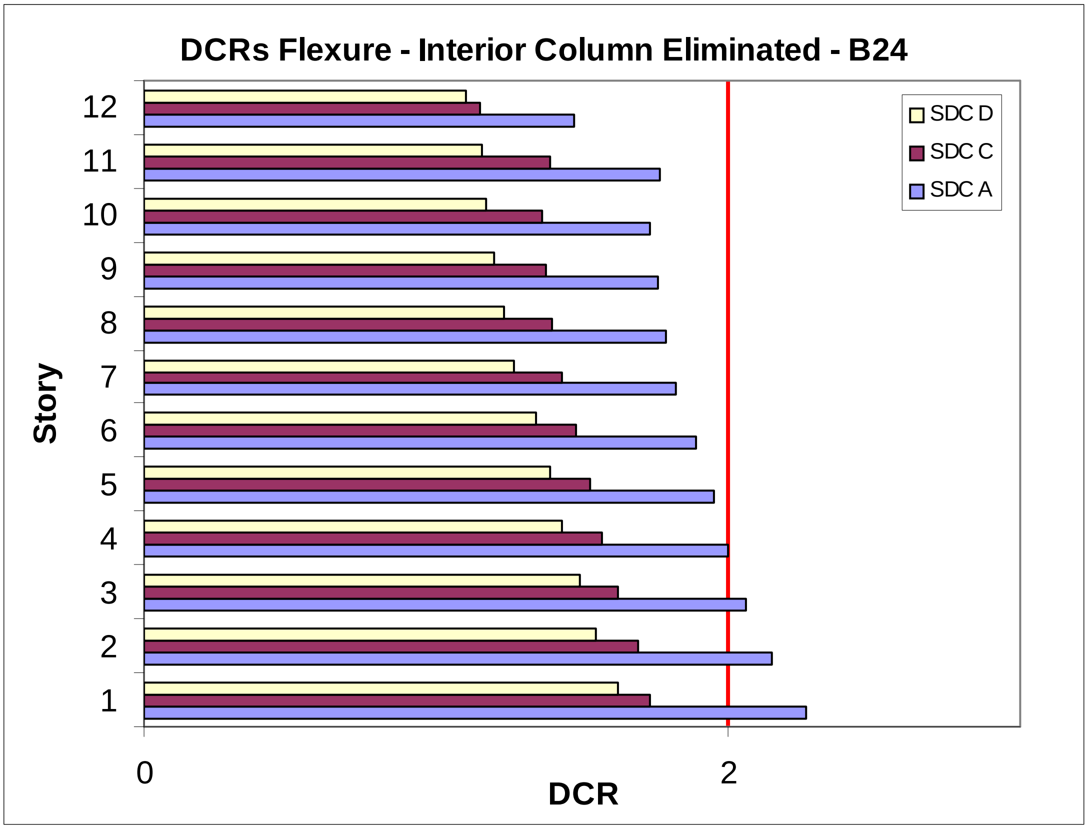


Figure 9a - Flexural DCR for beams in the vicinity of the removed column (Interior column)



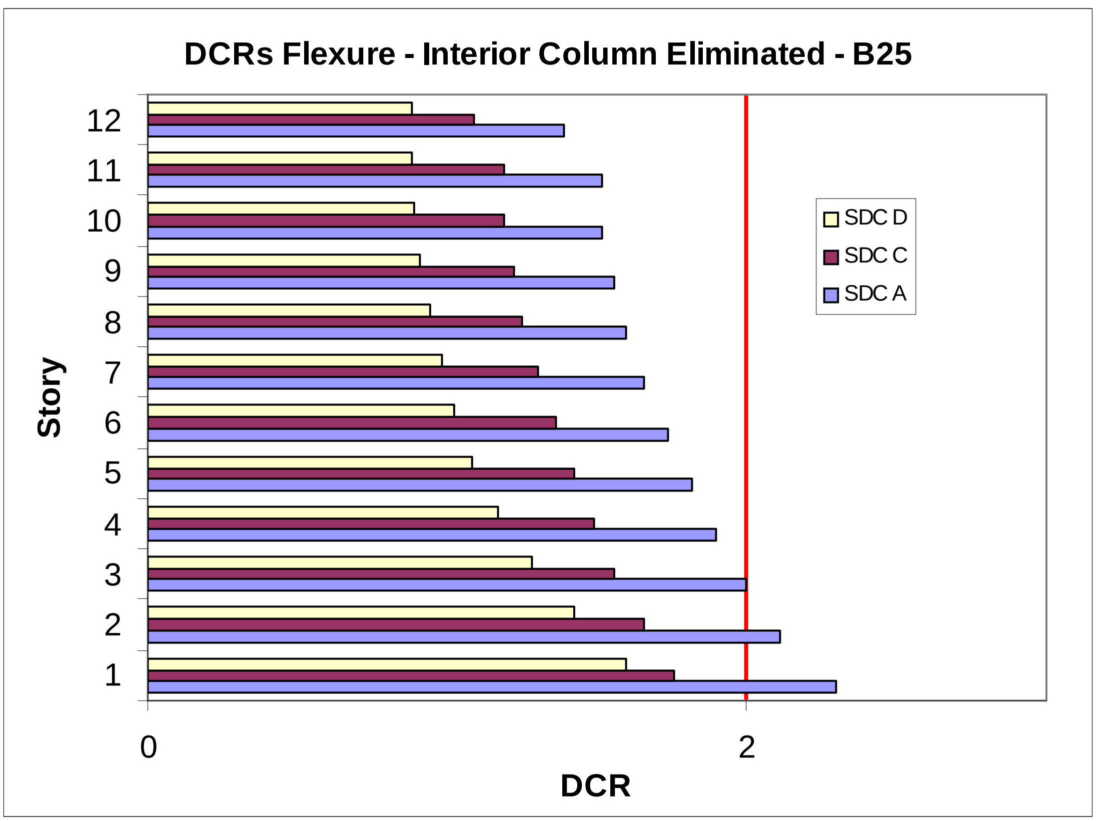


Figure 9b - Flexural DCR for beams in the vicinity of the removed column (Interior column)

Table 1 Seismic Design Parameters*

	SDC A Ordinary Moment Frame	SDC C Intermediate Moment Frame	SDC D Special Moment Frame
Zip Code	33122	10013	94105
Response acceleration Ss	0.065g	0.424g	1.50g
Response acceleration S1	0.024g	0.094g	0.61g
Seismic use group	I	I	I
Site class	D	D	D
Fa	1.6	1.4608	1
Fv	2.4	2.4	1.5
Response modification R	3	5	8

*Reference 6

Note: Design for wind was performed for wind speed 70 mph for each of the three buildings

Table 2 – Building Design

		SDC A	SDC C	SDC D
Transverse beams (Exterior)	Section	24"X18"	24"X18"	24"X22"
	Top Longitudinal Steel	5#6	5#8	6#9
	Bottom Longitudinal Steel	3#6	3#8	3#9
	Stirrups	#3 @ 7.75 2 Legs	#3 @ 3.75 2 Legs	#4 @ 4.75 2 Legs
Transverse beams (Interior)	Section	24"X18"	24"X18"	24"X22"
	Top Longitudinal Steel	5#7	5#8	6#9
	Bottom Longitudinal Steel	4#7	3#8	3#9
	Stirrups	#3 @ 7.75 2 Legs	#3 @ 3.75 2 Legs	#4 @ 4.75 2 Legs
Longitudinal beams (Exterior)	Section	24"X18"	24"X18"	24"X20"
	Top Longitudinal Steel	5#6	4#8	5#9
	Bottom Longitudinal Steel	3#6	3#8	3#9
	Stirrups	3 @ 7.75 2 Legs	#3 @ 3.75 2 Legs	#4 @ 4.375 2 Legs
Longitudinal beams (Interior)	Section	24"X18"	24"X18"	24"X20"
	Top Longitudinal Steel	5#7	5#8	6#9
	Bottom Longitudinal Steel	3#7	3#8	3#9
	Stirrups	3 @ 7.75 2 Legs	#3 @ 3.75 2 Legs	#4 @ 4 2 Legs
First floor interior columns	Section	24"X24"	24"X24"	24"X24"
	Reinforcement	16#11	16#11	16#11

Note: Concrete strength F'c is 6 ksi for the first six floor columns. All other members F'c = 4 ksi
Fy = 60 ksi
Slab thickness = 7"

Table 3 – Summary DCR for First Story Columns

		Long side column eliminated			Short side column eliminated			Corner column eliminated			Interior column eliminated		
	Column	A	C	D	A	C	D	A	C	D	A	C	D
Line A	C1	0.67	0.63	0.31	0.85	0.71	0.48	X	X	X	0.59	0.51	0.32
	C2	0.84	0.78	0.45	X	X	X	1.13	0.96	0.61	1.16	0.96	0.58
	C3	0.85	0.77	0.45	1.13	0.92	0.68	0.87	0.91	0.46	0.89	0.73	0.45
	C4	0.63	0.50	0.29	0.55	0.47	0.30	0.64	0.56	0.29	0.55	0.48	0.29
LineB	C5	1.14	1.11	0.59	0.88	0.68	0.46	1.14	0.95	0.59	1.16	0.98	0.61
	C6	1.03	0.81	0.60	1.20	1.01	0.69	1.03	0.91	0.60	X	X	X
	C7	1.00	0.79	0.58	1.03	0.97	0.60	1.00	0.89	0.58	1.22	1.08	0.73
	C8	0.82	0.75	0.43	0.85	0.52	0.45	0.82	0.62	0.43	0.85	0.72	0.44
Line C	C9	X	X	X	0.86	0.53	0.46	0.86	0.92	0.46	0.90	0.78	0.47
	C10	1.23	0.88	0.73	1.02	0.56	0.59	1.01	0.89	0.59	1.24	1.10	0.71
	C11	1.02	0.76	0.59	1.03	0.56	0.59	1.02	0.89	0.59	1.05	0.92	0.61
	C12	0.84	0.65	0.44	0.86	0.47	0.45	0.84	0.71	0.44	0.86	0.72	0.45
Line D	C13	1.15	1.12	0.60	0.86	0.47	0.45	0.88	0.73	0.46	0.86	0.72	0.45
	C14	1.05	0.79	0.61	1.02	0.63	0.59	1.02	0.92	0.59	1.02	0.92	0.59
	C15	1.01	0.78	0.60	1.02	0.63	0.59	1.02	0.92	0.59	1.01	0.93	0.60
	C16	0.84	0.62	0.44	0.86	0.53	0.45	0.85	0.88	0.44	0.86	0.73	0.45
Line E	C17	0.86	0.84	0.46	0.86	0.53	0.45	0.86	0.71	0.45	0.85	0.74	0.44
	C18	1.00	0.79	0.58	1.00	0.64	0.58	1.00	0.89	0.58	1.00	0.85	0.58
	C19	1.00	0.77	0.58	1.00	0.64	0.58	1.00	0.90	0.58	1.00	0.85	0.58
	C20	0.84	0.66	0.44	0.84	0.55	0.44	0.84	0.88	0.44	0.85	0.72	0.44
Line F	C21	0.57	0.55	0.30	0.54	0.33	0.29	0.55	0.46	0.29	0.55	0.49	0.29
	C22	0.85	0.65	0.45	0.83	0.54	0.44	0.83	0.65	0.44	0.85	0.72	0.44
	C23	0.85	0.66	0.44	0.83	0.54	0.44	0.83	0.65	0.44	0.85	0.72	0.44
	C24	0.54	0.38	0.29	0.53	0.39	0.29	0.52	0.42	0.28	0.55	0.48	0.29

Table 4 – Summary of Flexural DCRs for Beams Adjacent to Removed Columns

		Long Side Column Eliminated			Short Side Column Eliminated			Corner Column Eliminated		Interior Column Eliminated			
	Story	**B2**	**B3**	**B27**	**B6**	**B21**	**B22**	**B1**	**B21**	**B6**	**B7**	**B24**	**B25**
SDC A	12	1.82	1.86	1.20	1.20	1.84	1.83	1.76	1.55	1.47	1.40	1.47	1.39
	11	2.01	2.05	1.29	1.28	2.18	2.03	1.95	1.70	1.77	1.52	1.77	1.52
	10	1.99	2.03	1.30	1.29	2.13	2.01	1.94	1.70	1.72	1.52	1.73	1.52
	9	2.03	2.07	1.33	1.32	2.17	2.05	1.98	1.73	1.76	1.56	1.76	1.56
	8	2.07	2.12	1.37	1.36	2.19	2.09	2.02	1.77	1.78	1.60	1.79	1.60
	7	2.13	2.17	1.43	1.42	2.22	2.15	2.07	1.82	1.82	1.66	1.82	1.66
	6	2.21	2.26	1.51	1.49	2.29	2.24	2.16	1.89	1.89	1.74	1.89	1.74
	5	2.29	2.34	1.58	1.57	2.36	2.32	2.24	1.97	1.95	1.82	1.95	1.82
	4	2.37	2.41	1.66	1.65	2.41	2.40	2.32	2.04	2.00	1.90	2.00	1.90
	3	2.46	2.50	1.75	1.74	2.46	2.49	2.40	2.11	2.07	1.99	2.06	2.00
	2	2.60	2.67	1.87	1.86	2.55	2.65	2.53	2.23	2.23	2.17	2.15	2.11
	1	2.90	2.95	1.92	1.91	2.78	2.91	2.52	2.25	2.44	2.46	2.27	2.30
SDC C	12	1.34	1.36	0.94	0.93	1.44	1.35	1.28	1.00	1.15	1.09	1.15	1.09
	11	1.46	1.49	1.00	1.00	1.65	1.48	1.42	1.19	1.39	1.19	1.39	1.19
	10	1.46	1.49	1.01	1.00	1.63	1.46	1.41	1.19	1.35	1.19	1.36	1.19
	9	1.51	1.54	1.04	1.03	1.67	1.51	1.44	1.24	1.38	1.22	1.38	1.22
	8	1.57	1.60	1.07	1.06	1.71	1.58	1.47	1.29	1.40	1.25	1.40	1.25
	7	1.65	1.68	1.11	1.11	1.76	1.66	1.51	1.36	1.43	1.30	1.43	1.30
	6	1.75	1.79	1.17	1.16	1.84	1.76	1.62	1.47	1.48	1.36	1.48	1.36
	5	1.86	1.89	1.23	1.22	1.92	1.87	1.73	1.58	1.53	1.42	1.53	1.42
	4	1.98	2.01	1.30	1.29	2.00	1.98	1.84	1.68	1.57	1.49	1.57	1.49
	3	2.11	2.14	1.37	1.36	2.09	2.12	1.95	1.80	1.62	1.56	1.62	1.56
	2	2.27	2.30	1.47	1.46	2.20	2.28	2.16	2.01	1.70	1.66	1.69	1.66
	1	2.54	2.55	1.51	1.50	2.40	2.51	2.14	2.01	1.72	1.75	1.73	1.76
SDC D	12	0.91	0.79	0.69	0.63	1.04	0.83	0.73	0.65	0.97	0.79	1.10	0.88
	11	0.94	0.89	0.68	0.60	1.10	0.84	0.82	0.70	1.02	0.78	1.16	0.88
	10	0.95	0.88	0.70	0.62	1.11	0.84	0.82	0.70	1.03	0.79	1.17	0.89
	9	0.98	0.90	0.72	0.63	1.13	0.86	0.84	0.72	1.05	0.81	1.20	0.91
	8	1.01	0.93	0.75	0.66	1.15	0.88	0.86	0.74	1.08	0.84	1.23	0.94
	7	1.04	0.96	0.78	0.68	1.18	0.91	0.89	0.77	1.11	0.87	1.27	0.98
	6	1.10	1.02	0.83	0.72	1.24	0.95	0.95	0.82	1.16	0.90	1.34	1.02
	5	1.14	1.06	0.88	0.76	1.28	0.98	1.00	0.86	1.20	0.93	1.39	1.08
	4	1.18	1.11	0.92	0.80	1.31	1.01	1.03	0.88	1.23	0.97	1.43	1.17
	3	1.23	1.18	0.98	0.84	1.35	1.05	1.07	0.92	1.28	1.04	1.49	1.28
	2	1.28	1.26	1.14	0.90	1.40	1.10	1.14	0.98	1.33	1.15	1.55	1.42
	1	1.37	1.39	1.14	0.92	1.45	1.18	1.13	0.98	1.38	1.29	1.62	1.60

Table 5 – Summary of Shear DCRs for Beams Adjacent to Eliminated Columns

		Long Side Column Eliminated			Short Side Column Eliminated			Corner Column Eliminated		Interior Column Eliminated			
	Story	**B2**	**B3**	**B27**	**B6**	**B21**	**B22**	**B1**	**B21**	**B6**	**B7**	**B24**	**B25**
SDCA	12	0.85	0.86	0.86	0.86	0.88	0.86	0.82	0.79	1.02	0.95	1.02	0.96
	11	0.90	0.91	0.89	0.88	0.97	0.91	0.87	0.83	1.12	0.99	1.12	1.00
	10	0.90	0.91	0.89	0.88	0.95	0.91	0.87	0.83	1.11	0.99	1.11	1.00
	9	0.91	0.92	0.91	0.90	0.96	0.92	0.88	0.84	1.12	1.01	1.12	1.02
	8	0.92	0.93	0.93	0.92	0.97	0.93	0.89	0.86	1.13	1.03	1.13	1.04
	7	0.94	0.95	0.95	0.94	0.98	0.95	0.91	0.87	1.15	1.05	1.15	1.06
	6	0.96	0.97	0.98	0.97	1.00	0.97	0.94	0.90	1.18	1.08	1.18	1.09
	5	0.99	1.00	1.01	1.00	1.02	1.00	0.96	0.92	1.20	1.12	1.20	1.12
	4	1.01	1.02	1.04	1.03	1.04	1.02	0.98	0.94	1.23	1.15	1.23	1.16
	3	1.03	1.05	1.08	1.07	1.06	1.05	1.01	0.97	1.25	1.19	1.25	1.20
	2	1.07	1.08	1.02	1.12	1.08	1.08	1.05	1.01	1.29	1.24	1.29	1.24
	1	1.09	1.08	1.09	1.11	1.09	1.07	1.01	0.98	1.30	1.25	1.30	1.25
SDCC	12	0.61	0.62	0.62	0.62	0.64	0.62	1.04	0.67	0.74	0.69	0.74	0.69
	11	0.65	0.66	0.64	0.63	0.70	0.66	1.11	0.72	0.81	0.71	0.81	0.72
	10	0.65	0.66	0.64	0.64	0.69	0.65	1.11	0.72	0.80	0.72	0.80	0.72
	9	0.66	0.66	0.65	0.65	0.70	0.66	1.13	0.73	0.81	0.73	0.81	0.73
	8	0.67	0.67	0.67	0.66	0.70	0.67	1.15	0.75	0.82	0.74	0.82	0.75
	7	0.68	0.69	0.68	0.68	0.71	0.68	1.18	0.76	0.83	0.76	0.83	0.76
	6	0.70	0.70	0.70	0.70	0.73	0.70	1.23	0.81	0.85	0.78	0.85	0.79
	5	0.71	0.72	0.73	0.72	0.74	0.72	1.27	0.84	0.87	0.80	0.87	0.81
	4	0.73	0.74	0.75	0.75	0.75	0.74	1.30	0.86	0.89	0.83	0.89	0.83
	3	0.75	0.76	0.78	0.77	0.76	0.76	1.33	0.88	0.91	0.86	0.91	0.86
	2	0.77	0.78	0.82	0.81	0.78	0.78	1.38	0.93	0.94	0.90	0.93	0.90
	1	0.79	0.78	0.81	0.81	0.79	0.78	1.34	0.89	0.93	0.90	0.94	0.91
SDCD	12	1.10	1.11	0.76	0.66	0.73	0.72	1.04	0.67	0.78	0.74	0.72	0.84
	11	1.17	1.18	0.79	0.67	0.83	0.77	1.11	0.72	0.87	0.77	0.75	0.88
	10	1.17	1.18	0.79	0.68	0.82	0.77	1.11	0.72	0.86	0.78	0.76	0.89
	9	1.19	1.20	0.81	0.69	0.83	0.79	1.13	0.73	0.87	0.79	0.77	0.90
	8	1.21	1.22	0.83	0.71	0.84	0.80	1.15	0.75	0.88	0.81	0.79	0.93
	7	1.23	1.25	0.86	0.73	0.85	0.82	1.18	0.76	0.89	0.83	0.81	0.96
	6	1.28	1.30	0.91	0.76	0.89	0.86	1.23	0.81	0.93	0.86	0.84	1.00
	5	1.32	1.34	0.94	0.79	0.92	0.89	1.27	0.84	0.95	0.89	0.87	1.04
	4	1.35	1.37	0.97	0.81	0.93	0.92	1.30	0.86	0.97	0.92	0.89	1.07
	3	1.39	1.40	1.01	0.84	0.95	0.94	1.33	0.88	0.99	0.95	0.92	1.11
	2	1.43	1.45	1.06	0.88	0.98	0.98	1.38	0.93	1.02	0.98	0.96	1.16
	1	1.46	1.44	1.04	0.87	0.98	0.97	1.34	0.89	1.02	0.99	0.96	1.17