

# Simplified Analysis of Reinforced Concrete Framed Structures for Progressive Collapse

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**Abstract**—Significance of designing structures to resist progressive collapse has been recognised following the collapse of several structures, such as that at Ronan Point in 1968 and the collapse of World Trade Centre in 2001, where the consequences were deemed unacceptable relative to the initial damage. Progressive collapse requirements, however, must be incorporated into a structure without substantial increase in the cost of the structural system. This paper illustrates how progressive collapse analysis can be done and design criteria economically incorporated into typical structural systems. A variety of research efforts in the past decades have attempted to quantify aspects of robustness (such as redundancy) and identify design principles that can improve robustness. Only very few standards like British Standard, Eurocode and GSA incorporated clauses for the robustness (or integrity) of structure. In this paper, a six storey building is considered and simplified analysis is done by removing one column at a time as per GSA standard. Modifications to GSA clauses have been proposed by introducing dynamic increase factor for simplified analysis. Also, modification in factor of safety is suggested while analysing models with removed columns. Strain rate effect is also considered to make the progressive collapse resistance design economical. Also, this study illustrates the inherent ability of seismically designed RC beam column frames to resist progressive collapse.

**Key Words**— Alternative load paths, Building design, Progressive collapse, Robustness of structure, Reinforced concrete, structural integrity, demand-capacity ratio.

## INTRODUCTION

**R**OBUSTNESS of a building is the characteristic which defines the structure's strength in terms of integrity and redundancy. This characteristic of a structure makes it resistant to progressive collapse. Progressive collapse is defined as the situation where local failure of primary structural component(s) leads to the collapse of adjoining members, which in turn leads to additional collapse. Hence, the extent of total damage is disproportionate to the initial damage. Another way of describing progressive collapse is that it is a chain reaction or propagation of failures following damage to a relatively small portion of a structure. However, the term disproportionate collapse is used when the collapse is out of proportion to the event that triggers it. Basically, a disproportionate collapse is always a progressive collapse but a progressive collapse is not always a disproportionate one.

This phenomenon was first realized after the progressive as well as disproportionate collapse of the Ronan Point apartment tower in England in 1968. The building had load bearing walls without any structural frame. A small explosion led to the failure of a load bearing wall which resulted in progressive collapse. The Skyline Plaza (March, 1973), a large complex located in Virginia, was another case of such collapse. In midst of construction, one apartment building and the parking garage adjoining it collapsed.

The root cause of the failure was premature removal of the form work triggering shear failure around a number of columns on the 23<sup>rd</sup> storey which led to the failure of 23<sup>rd</sup> storey followed by 22<sup>nd</sup> and so on causing progressive collapse. WTC is another important example of progressive collapse. Once the columns were destroyed by the plane crash, the alternative load paths were formed through the trusses. The columns were probably near, but not over, their ultimate load capacity. However, the fires proved fatal as the structural steel began to lose its strength at high temperature and after sometime, a complete structural collapse occurred.

## CODES AND STANDARDS

Prevention of progressive collapse became one of the unchallenged imperatives in structural engineering after such accidents. And, code-writing bodies and governmental user agencies attempted to develop design guidelines and criteria that would reduce or eliminate the susceptibility of buildings to this form of failure.

British Standard Code (BS 8110-1 1997 & BS 8110-2 1985) and Eurocode (EN 1992-1-1: 2004) have similar guidelines. They suggest designing the structure for accidental load and if not, then avoiding the situation where damage to small areas of a structure or failure of single element may lead to progressive collapse. The layout of the building is checked to identify any key elements, the failure of which would cause the collapse of more than a limited portion close to the element in question. This key element design must be taken into consideration. Elements other than key elements are provided with vertical ties in accordance with the code provisions.

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Although there is no explicit mention of redundancy or alternate load paths in the American Code, it states to improve redundancy (ACI: 318-02). But U.S. General Services Administration (GSA Progressive Collapse Guidelines for New Federal Office Buildings and Major Modernization Projects 2003) and American Society of Civil Engineering (ASCE 7-02 Section 1.4) are two bodies which provide design guidelines for progressive collapse.

## IMPORTANCE OF INTEGRITY

A building with interior core wall for lateral load resistance and ordinary moment resisting frame for gravity loads, for example, may have a very limited ability to redistribute loads and prevent progressive collapse. Gravity load designed structures are incompetent to develop alternate load path after the removal of a vertical loadbearing member. As illustrated in Fig. 1(a), lack of continuous bottom reinforcement in the beam over the removed column will cause brittle failure of the resulting two-bay beam. Buildings having special moment resisting frames (SMRF) for their lateral systems Fig. 1(b), however, can provide both the ductility and the capacity required to prevent progressive collapse.

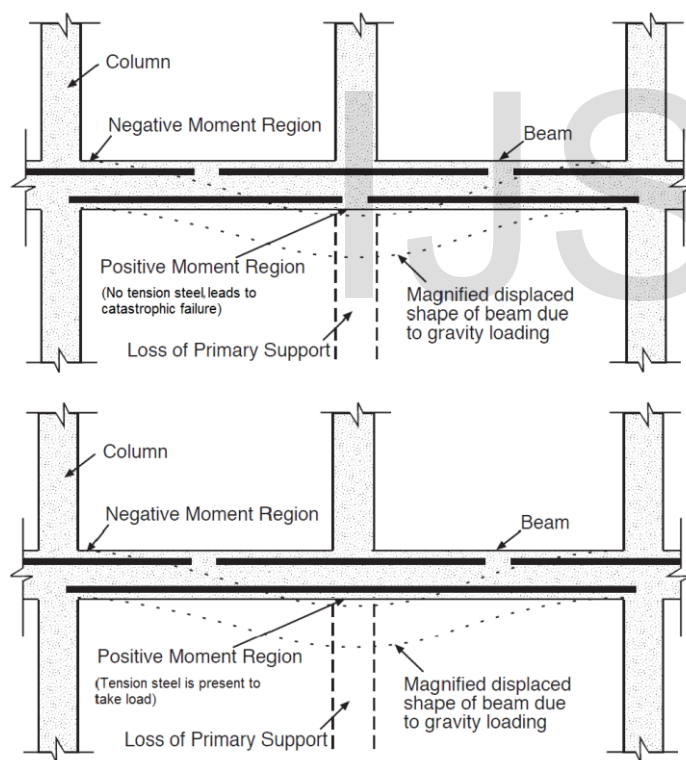


Figure 1: Response of a 'missing column' scenario: (a) gravity load design, (b) seismically designed.

## MODEL STUDIED

The GSA progressive collapse guideline provides a detailed methodology and performance criteria needed to assess the vulnerability of new and existing buildings to progressive collapse. For typical structural configurations, framed structures shall consider the instantaneous loss of a column for one story above

grade located near the middle of the long side of the building; near the middle of the short side of the building; and at the corner of the building. A separate analysis must be performed for each case.

To avoid an overly conservative design under normal service-load conditions, it is recognized that full live load is unlikely. Thus, in the GSA guidelines, live load is reduced to 25% of the full design live load. Multiplying the load combination by a factor Dynamic Increase Factor (DIF) of 2 is the GSA's simplified approach to account for amplification in the response from dynamic effects that can occur when a structural element is violently removed from a structure. In addition, strength increase factors are applied to the properties of construction materials to account for strain rate effects and material over-strength. To determine expected material strengths, the concrete compressive strength  $f'_c$  and the yield strength of the reinforcing steel  $f_y$  are increased by a factor of 1.25.

But the DIF recommended by GSA is overly conservative (as found in the study done in paper [14]) and hence a value of 1.5 can be considered.

To evaluate the results of a linear elastic analysis, the concept of demand-capacity ratio (DCR) is used in this paper. The DCR for structural components is defined as

$$DCR = \frac{QUD}{QCE} \quad (\text{Eq. 1})$$

Where,

QUD = demand in component or connection/joint (moment, axial force, and shear) determined from the analysis; and

QCE = expected ultimate, un-factored ( $\phi = 1.0$ ) capacity of the component or connection/joint (moment, axial force, and shear).

## VALIDATION OF MODEL

A study "Approximations in Progressive Collapse Modeling", ASCE [11] is done to compare the different models. Structural models that can be utilized for studying progressive collapse are broadly classified as macromodels and micromodels. The emphasis in the former is on generalized strain and/or generalized stress behaviour (for example, curvature and/or bending moment behaviour) as opposed to point wise constitutive response in the latter. Continuum finite-element models are examples of micromodels. Macromodels, on the other hand, utilize a combination of shell, beam-column, and discrete spring finite elements to simulate the overall response of a structure. Four models were studied namely, a detailed micromodel of the full 3D system, termed M1; a model of the full 3D system composed of macroelements for beams, columns and connections and shell elements for the slab, termed M2; a 3D micromodel of a single frame in the system, termed M3; and a macromodel of the frame modeled in M3, termed M4. Model M1 is the most sophisticated, whereas M4 is the least complicated. From this study it is

concluded that a 3D linear elastic model (M2) gives fairly good results and hence can be used as it saves a lot of time. For this purpose ETABS software [10] is used for analysis purpose.

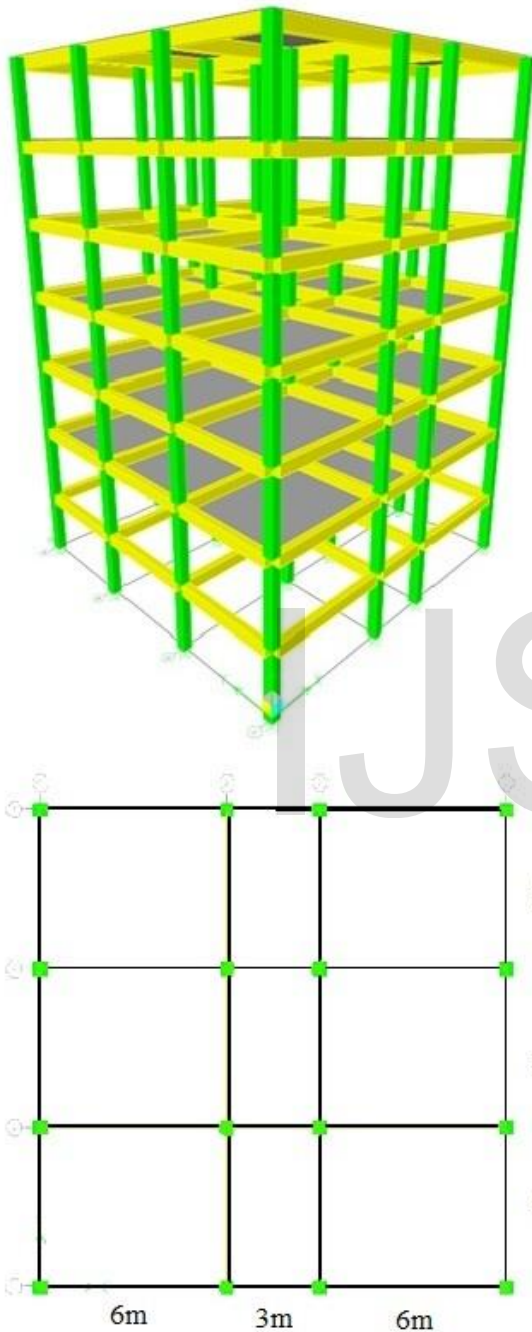


Figure 2: Model building (a) 3-D model, (b) Plan.

### MODEL BUILDING

The model considered for the study is a six storey RC frame structure serving the purpose of Educational building consisting of classrooms. The structure consists of three 5m bay on one direction and two 6m bay in another direction with a gallery of 3m in

between them. Typical floor-to-floor height is 3.5m. Live load of 4 kN/m and superimposed dead load of 2 kN/m are assumed in the analysis.

When performing a static analysis, the vertical load case applied to the structure in the accidental case is as follows:

$$\text{Load} = \text{DIF} (DL + 0.25LL) \quad (\text{Eq. 2})$$

The 3-D ETABS model of the building is shown in Fig. 2, and material property and loads applied data are summarized in Table 1 and 2.

The structure is designed for Zone IV with a damping ratio of 3% is assumed. The progressive collapse design criteria, as well as the element removal procedure, followed the U.S. General Services Administration’s (GSA) “Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects.” [7]. Few changes are made on the procedure given in GSA such as, the DIF is 2 according to the GSA, but research [14] has shown that this will produce an overly conservative design and hence a DIF of 1.5 is assumed.

Beam cross-section: 300mm X 500mm

Column cross-section: 400mm X 400mm

TABLE 1  
Material Properties

PROPERTY	VALUE
Fck	25MPa
Ec	25000MPa
Fy	415MPa
Poisson’s ratio	0.2
Mass/Volume	2.4 Kg/m <sup>3</sup>
Wt/Volume	23.56KN/m <sup>3</sup>

TABLE 2  
Load applied

LOADING	
Dead load (IS 875 Part I)	2KN/m <sup>2</sup>
Live Load ( IS 875 Part II)	4KN/m <sup>2</sup>
Wall loads	25KN/m( Exterior wall) 10kN/m( Interior wall)
Earthquake load (IS 1893 part I)	Zone IV Z=,24 D=3% Soil type-II I=1.5 R=5

Zone IV: Severe earthquake intensity  
R: Response reduction factor  
Soil Type-II: Medium stiff soils

D: Damping ratio  
Z: Zone factor  
I: Importance factor

## ANALYSIS AND RESULTS

In this paper three models are presented. Firstly, 3-D model of the structure with no column removed [case A] (Fig. 2.a), secondly 3-D model of the structure with next to corner column removed [case B] (Fig. 3) and thirdly a 3-D model of the structure with corner

column removed [case C] (Fig. 4), these models are developed based on the guidelines given by the GSA and analysis is done using ETABS software. All the other possible member failures are also considered for checking the progressive collapse, but these three are found to be the critical once and hence only these three are presented in this paper. The maximum moments are found when no column is removed (i.e., case A) and compared with the maximum moment in the same members in case B and case C. The loading condition for case A is according to IS 1893-2002 (i.e. Load = FS (DL + LL)), whereas in case B and case C loading is considered as per Eq. 2. In the loading condition for case B and C, the working Factor of Safety (FS) is not considered as in case A to calculate the design load out of the working load, as these are accidental cases.

The dynamic increase factor (DIF) factor of 1.5 is considered in case B and case C to compensate for the dynamic condition developed when the member is removed abruptly and hence simplifying the analysis by doing static analysis instead of dynamic analysis.

The moments developed in member a, b, c and d (as marked in Fig. 3 and Fig. 4) in all the three cases are compared (table 3). This comparison clearly shows the increase in the amount of moment in the beams. The ultimate capacity of beam is calculate by increasing the characteristic strength of the material. This increase factors are applied to account for strain rate effects and material over-strength and compared to the moment obtained from the above analysis. This comparison shows that only in two beams (a and c) the moment is comparatively higher whose moment ratios are much higher than 2 as shown in table 3. Fig. 7 also shows these beams in red colour. The elastic design of the structure is done in the software to get design to prevent progressive collapse. The reinforcement is altered for the most critical situation and generalised for all the floors. The study shows that there is an increase of only 13% of total steel used in the structure.

Total volume of steel used without considering design for progressive collapse = 3.89 m<sup>3</sup>

Total volume of steel used considering design for progressive collapse = 4.398 m<sup>3</sup>

Percentage increase in the steel = 13%

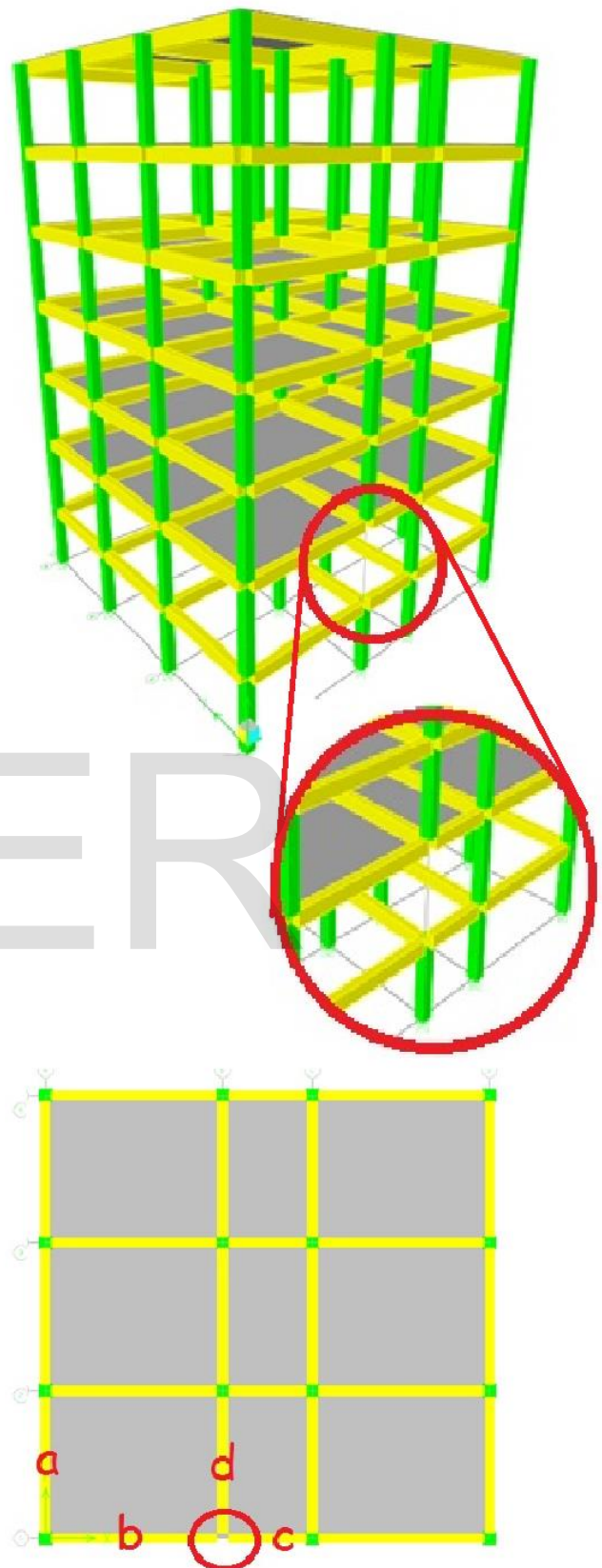
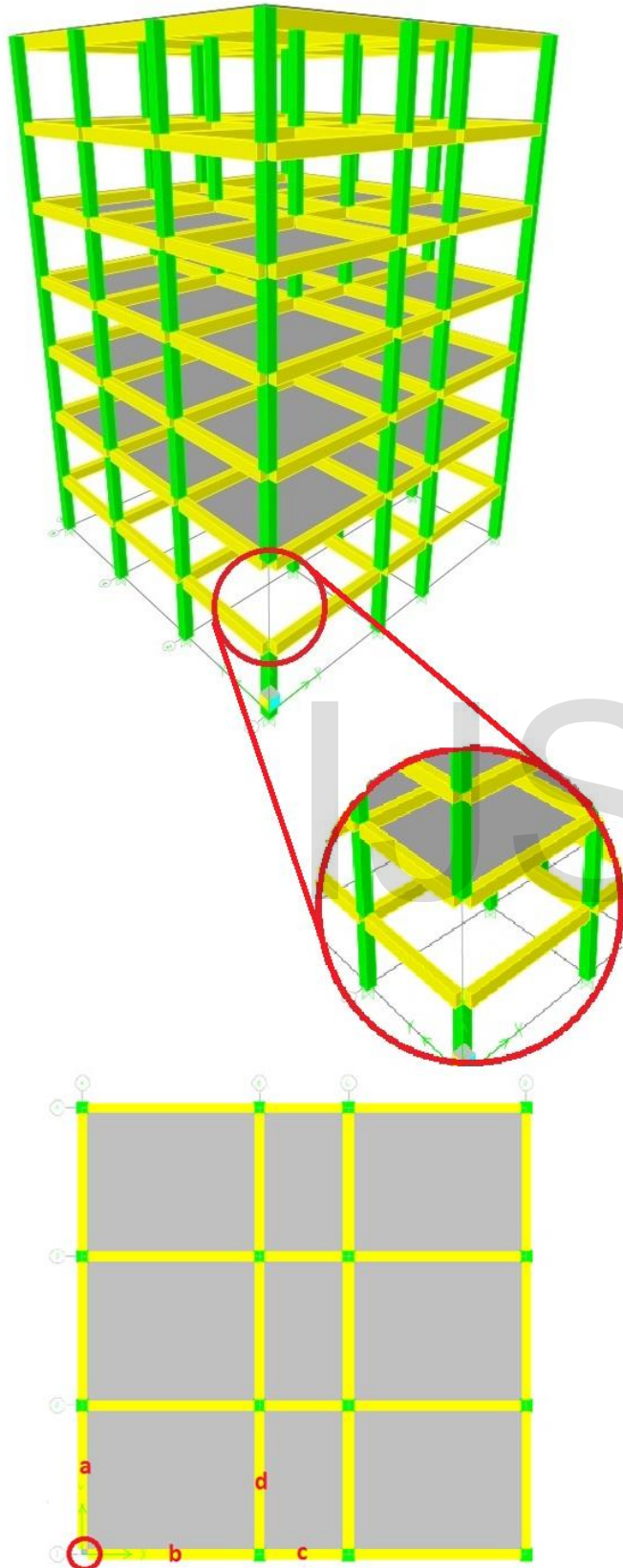


Figure 3: 3-D model and plan of the structure with next to corner column removed (ETABS)



The stress distribution on the slab and bending diagram of beams and columns are shown in Fig. 5 and Fig. 6 for case B and case C respectively. It can be clearly seen that the stresses on the slab are comparatively higher on the slab around the removed column than those on the normal condition. But it is clear that the failure of slab takes place only when the beam fails and since the beam is prevented from failure, the slab will also be safe.

Columns are analysed by calculating the DCR value according to Eq. 1. The DCR value of the columns are shown in Fig. 7(a) and Fig. 7(b) for case B and case C respectively. The section chosen are those containing members with highest DCR values, in rest of the sections the DCR values are within acceptable range.

### CONCLUSION

This study illustrates the inherent ability of seismically designed RC beam-column frames to resist progressive collapse. The analysis shows that a building designed for zone IV is capable of generating an alternate load path to transfer the loads if a vertical member fails, since all the columns are having acceptable DCR values. The high moments generated on the beams are at the beam-column junction and hence design can be done to prevent collapse. The elastic design, to prevent progressive collapse, results in an increase of 13% of total steel used in this structure, which is negligible to the total cost of the structure. This is an economically efficient solution which will prevent collapse. Hence, progressive design can be done in all important structure to prevent loss of the property with small investment. The analysis in this paper prevents the design from being overly conservative by considering a dynamic increase factor of 1.5 and not considering the working factor of safety in the accidental cases. This study will be valuable to engineers involved in the selection of structural systems for projects that require progressive collapse mitigation.

TABLE 3

Maximum Bending Moment and Shear Force on the beam a, b, c and d.

Beam	a		
length (in m)	5		
Column removed	Nil	corner	ratio
failure section (in m)	5 (corner)		
Max. B.M	-138.55	-582.86	4.2
Max.Shear	154.94	341.88	2.2

Beam	b					
length (in m)	6					
Column removed	Nil	next to corner	ratio	Nil	corner	ratio
failure section (in m)	0 (corner)			6 (corner)		
Max. B.M	-165.83	-338.9	2.04	-188.518	-532.95	2.8
Max.Shear	-182.69	-246.96	1.35	192.25	285	1.48

Beam	c			d		
length (in m)	3			5		
Column removed	Nil	Next to corner	ratio	nil	next to corner	ratio
failure section (in m)	3 (corner)			5 (corner)		
Max. B.M	-93.5	-465.5	4.98	-127.06	-274.5	2.1
Max.Shear	78.04	333.5	4.2	132.34	169	1.2

Figure 4: 3-D model and plan of the structure with corner column removed (ETABS)

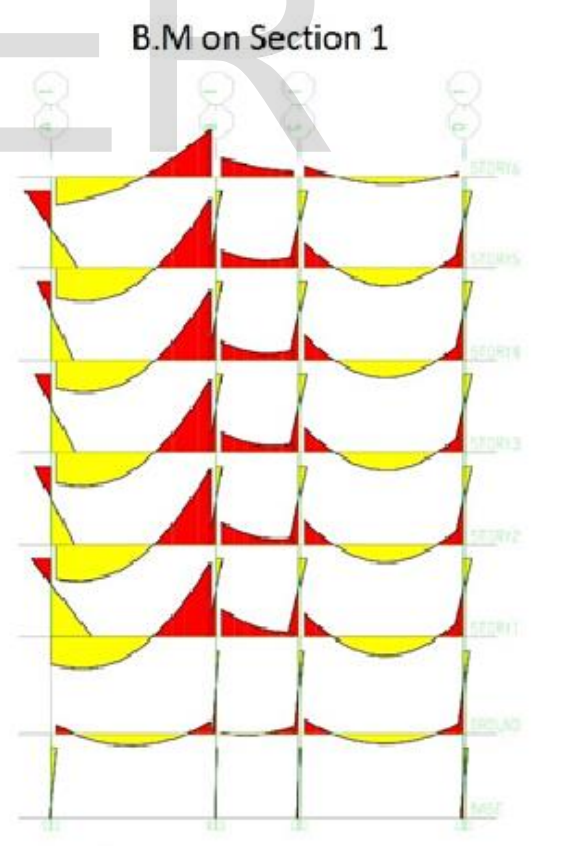
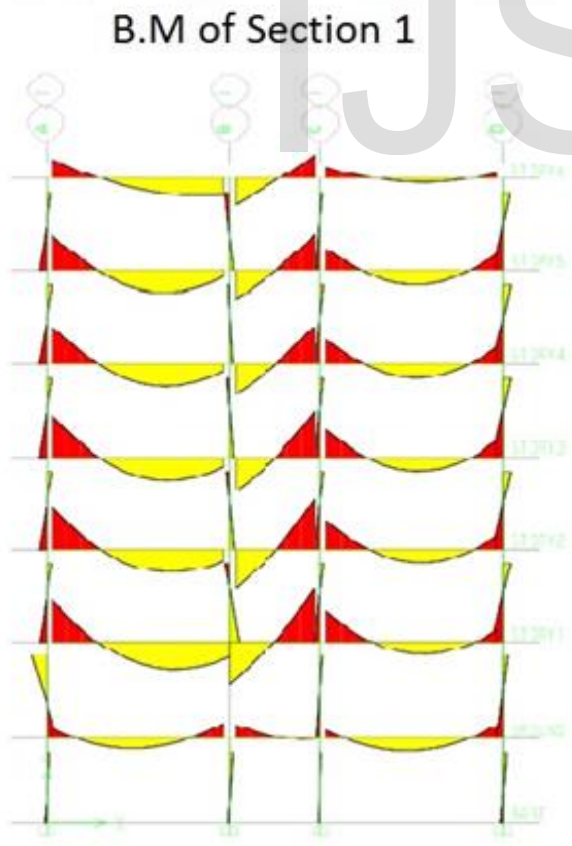
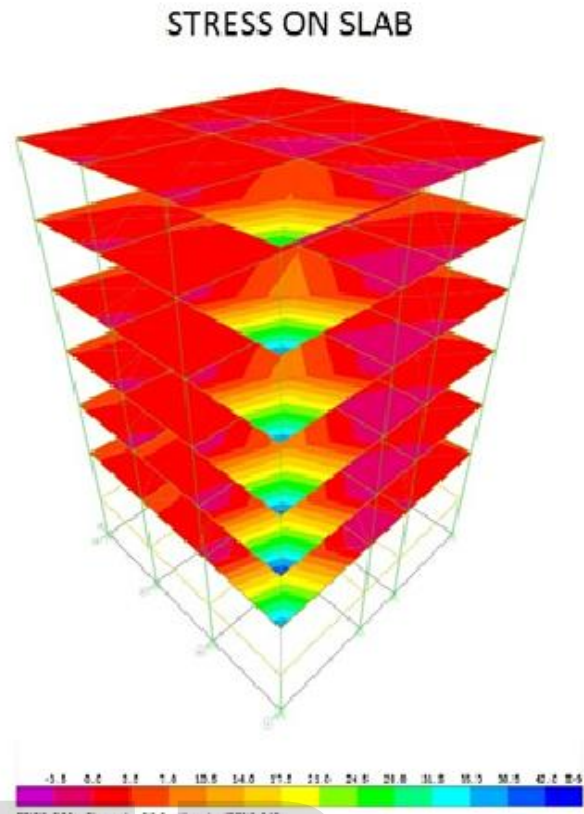
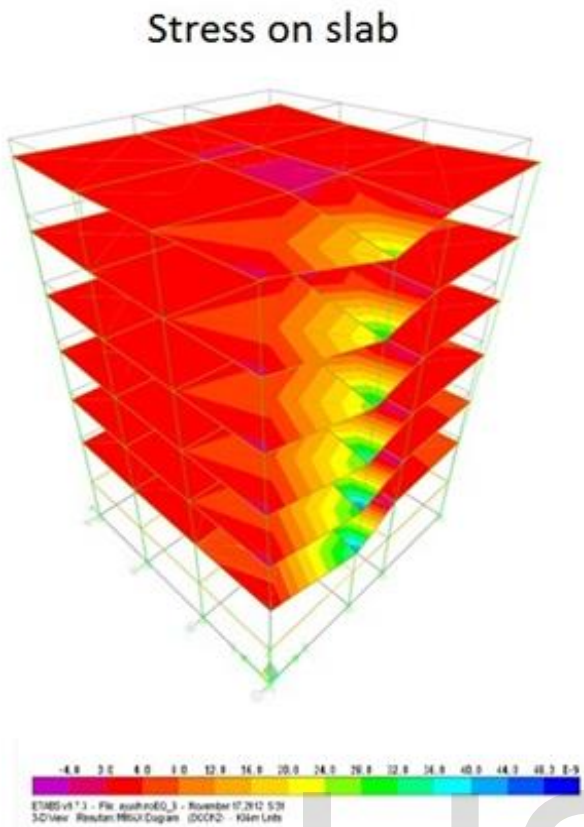


Figure 5: Case B (a) stresses on slab, and (b) bending moment of elevation section 1.

Figure 6: Case C (a) stresses on slab, and (b) bending moment of elevation section 1.

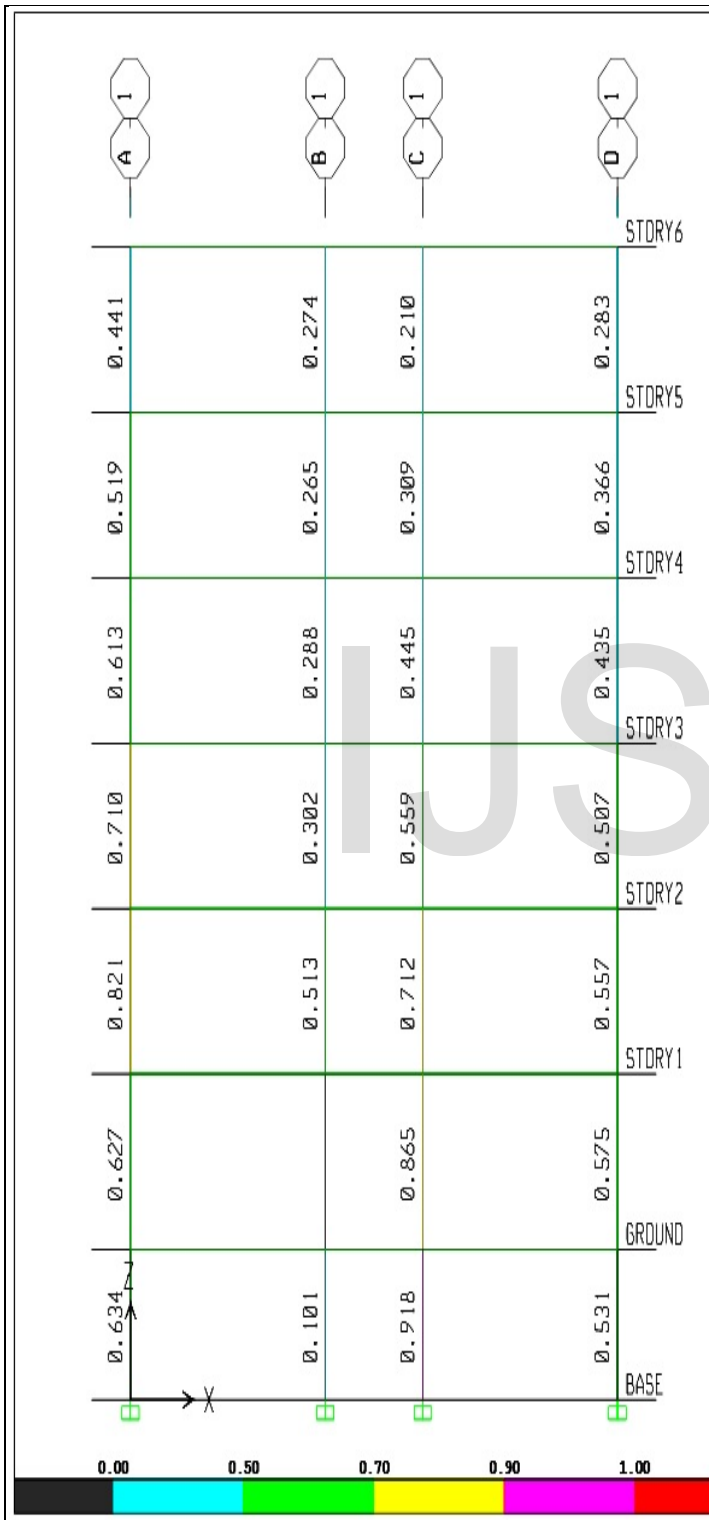


Figure 7(a): DCR values in case B (elevation section 1)



Figure 7(b): DCR values in case C (elevation section A)

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