EFFECT OF RETROFIT STRATEGIES ON MITIGATING PROGRESSIVE COLLAPSE OF MULTISTOREY STEEL FRAME STRUCTURES

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ABSTRACT

EFFECT OF RETROFIT STRATEGIES ON MITIGATING PROGRESSIVE COLLAPSE OF MULTISTORY STEEL FRAME STRUCTURES

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In the past decades, there have been cases where buildings around the world have experienced partial or total collapse under extreme abnormal loading conditions. Although building collapse is a rare event, it may result in significant catastrophes and property loss when it occurs. In particular, the collapse of the World Trade Center in 2001 following the terrorist attacks has triggered increased interest in blast and progressive collapse resistant building design. The upgrading of steel frames building has thus become an important topic for researchers.

In this study, progressive collapse response was investigated using the Alternate Path Method (APM) recommended in the United States' General Service Administration (GSA 2003) and Department of Defense (DoD 2005) guidelines. Nonlinear dynamic analysis for progressive collapse is done as a precise tool for evaluation of the progressive collapse potential of building structures. The studied models were 18 typical floors, with 6 bays in the longitudinal direction, 3 bays in the transverse direction, where six cases of removed column scenarios were conducted.

The objectives of this study are to 1) investigate the effect of three retrofit strategies (by increasing strength and/or stiffness of the beams) on the response of steel frames for model of span 6m (reference model) subjected to GSA and DoD loads by studying three parameters (the chord rotation, tie forces and displacement ductility demand), 2) investigate the effect of variation of bay span on the three performance parameters by studying three models of different spans of 5.0, 7.5 and 9.0 m subjected to GSA load, and 3) propose equations for predicting the effect of retrofit strategy and bay span on the three performance indicators.

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Finally, I dedicate this thesis to my parents, brother and Jody for their invaluable support during my study. Also, special thanks go to my wife for her support and patience during the period of my study and to my daughter Menna, who give me real happiness and blessings in my life. Their presence has encouraged me very much to work on my research and to achieve my goals.

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LIST OF SYMBOLES

GSA General Services Administration.

UFC Unified Facilities Criteria (DOD 2005).

E Material Young's modulus.

$\mathbf{F_y}$ Yield strength of steel.

K Initial stiffness.

$\mathbf{M_p}$ Plastic Moment of the beam.

Z Plastic section modulus of steel.

I Moment of inertia of the beam.

$\boldsymbol{\theta}$ Rotation of the beam at connection (in degress).

$\boldsymbol{\theta_{upgr.(s)}}$ Upgraded chord rotation after increasing the strength only.

$\boldsymbol{\theta_{upgr.}}$ Upgraded chord rotation after increasing the strength and stiffness.

T.F Tie Force in the beam.

$\boldsymbol{\mu_\Delta}$ Displacement ductility demand of the beam.

R_s^θ Reduction factor for chord rotation due to increase in strength.

R_k^θ Reduction factor for chord rotation due to increase in stiffness.

R_s^T Reduction factor for tie force due to increase in strength.

R_k^T Reduction factor for tie force due to increase in stiffness.

R_s^μ Reduction factor for displacement ductility demand due to increase in strength.

R_k^μ Reduction factor for displacement ductility demand due to increase in stiffness.

A_s^θ Factor for effect of variation of bay area on chord rotation due to increase in strength.

A_k^{θ} Factor for effect of variation of bay area on chord rotation due to increase in stiffness.

A_s^{T} Factor for effect of variation of bay area on Tie Force due to increase in strength.

A_k^{T} Factor for effect of variation of bay area on Tie Force due to increase in stiffness.

A_s^{μ} Factor for effect of variation of bay area on displacement ductility demand due to increase in strength.

A_k^{μ} Factor for effect of variation of bay area on displacement ductility demand due to increase in stiffness.

$\boldsymbol{\alpha}$ Upgrading Factor (increase in strength or stiffness).

$\boldsymbol{\alpha_s}$ Strength Factor (increase in strength).

$\boldsymbol{\alpha_k}$ Stiffness Factor (increase in stiffness).

Chapter 1

Introduction

1.1 GENERAL

Nowadays, building structures have been designed to resist normal loads such as those due to self-weight, occupancy or seismic effects, etc. Structures are designed to resist all expected loadings without failure. However, structural failures do occasionally occur due to inadequate design and construction, especially for extreme and abnormal loads. Since the 1968 chain-reaction failure of the Ronan Point Apartment Block in London, as shown in Figure 1.1 (a), triggered by a gas explosion, abnormal loading and progressive collapse have become increasingly recognized as important phenomena to be accounted for in engineering design practice worldwide. Indeed, the complete structural collapse of the twin towers of the World Trade Center (WTC) in New York City on September 11, 2001, as shown in Figure 1.1 (b), has significantly increased the concern about these phenomena.

Progressive collapse results from abnormal loads. These abnormal loads may be grouped as pressure loads (e.g., explosions, detonations, tornado wind pressures), impact (e.g., vehicular collision, aircraft or missile impact, debris, swinging objects during construction or demolition), deformation-related (e.g., softening of steel in fire, foundation subsidence).

In progressive collapse, an initial localized damage or local failure spreads through neighboring elements, possibly resulting in the failure of the entire structural system. The most viable approach to limiting this propagation of localized damage is to maintain the integrity and ductility of the structural system. The commentary in ASCE 7-05 suggests general design guidance for improving the progressive collapse resistance of structures, but it does not provide any specific implementation rules. Recent design procedures to mitigate the potential for progressive collapse in structures can be found in the design guidelines issued by the U.S. General Services Administration (GSA 2003) and the Department of Defense (DoD 2005).

The direct approach, or the Alternate Path Method (APM), is preferred in these design guidelines. In this method, a single column is typically assumed to be suddenly missing, and an analysis is conducted to determine the ability of the structure to bridge across the missing column. The APM is mainly concerned with the vertical deflection phase or the chord rotation of the beams of the building after the sudden removal of a column. The chord rotation is equal to the vertical deflection divided by beam span. As such, it is a threat-independent, design-oriented method for introducing further redundancy into the structure to resist propagation of collapse.

The ductility of steel alone is not sufficient to prevent progressive collapse of steel buildings; but it is the basic requirement even though it cannot guarantee that the steel building will not fail under extreme loading. Failure in steel building occurs due to insufficient strength in the beams to bridge the load from the removed column to the adjacent columns, which leads to the failure of those beams and consequently the whole building. This means that upgrading the beam and increasing its strength and stiffness will prevent a steel frame building from failing. On the other hand, in case of high hazard events where more than one column is lost, upgrading both beams and columns is needed.

Existing buildings that were designed for gravity loads or designed according to earlier codes may have inadequate resistance to progressive collapse. Steel frame structures designed to earlier codes have not behaved well during extreme hazard events due to insufficient carrying capacity. A major challenge for structural engineers is how to retrofit an existing structure to upgrade its capacity and to what level of protection. It is not a normal practice in retrofitting to attempt to make the existing structure comply with the current code provisions. The retrofit objectives for the structure should rather depend on a performance-based criterion to ensure a defined level of damage or to prevent collapse of the building.

The retrofit strategy may involve targeted repair of deficient members, providing systems to increase stiffness and strength or providing redundant load carrying systems by a structural system like mega truss at the top of the building or using bracing systems that redistribute the loads throughout the entire structure. In general, a combination of different strategies may be used in the retrofitting of the structure.

1.2 OBJECTIVE AND SCOPE OF STUDY

The objective of this study is to assess the effectiveness of three retrofit strategies (increasing the strength and/or stiffness of the beams) on the response of steel frame by evaluating the enhancement of three parameters indicators, which are chord rotation, tie forces and displacement ductility demand of the beams after being upgraded. All the beams had been upgraded to safeguard the building against all scenarios of column loss in the ground floor or upper floors. The studied model was 3 bays x 6 bays in plan, 18-story typical steel frame with a span of 6 meters in both directions subjected to two loading combinations of GSA and DoD gravity loads that is damaged by being subjected to six scenarios of a column loss. Also, the effect of variation of bay area on the same parameters (chord rotation, tie force and displacement ductility demand) was investigated by studying models of different spans of 5, 7.5 and 9 meters subjected to GSA loading combination and was used to derive a proposed equation for predicting chord rotation, tie force and displacement ductility demand for different bay areas.

1.3 OUTLINE OF THE THESIS

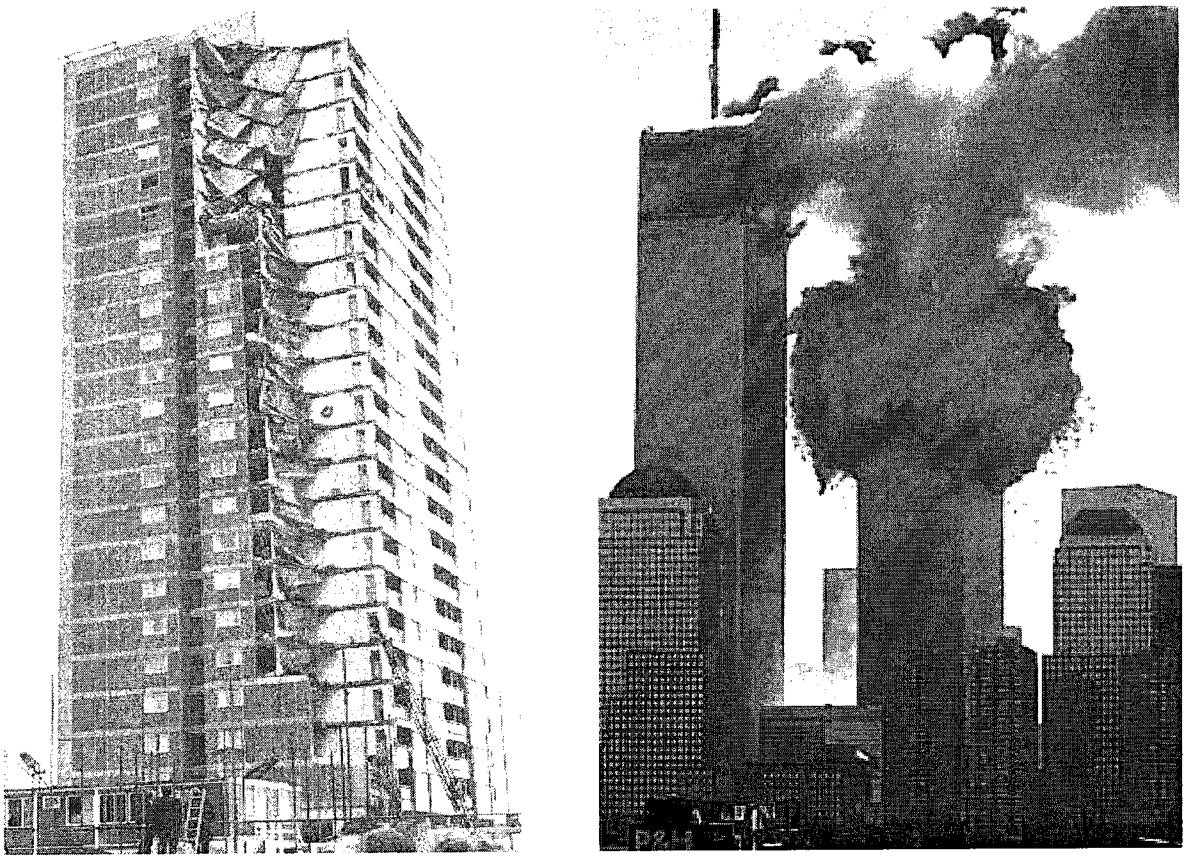
The material of this study is presented in five chapters, and each chapter has its own objective as follows:

Chapter 2 reviews the two most popular guidelines for progressive collapse, which are the General Services Administration guideline (GSA 2003) and Unified Facilities Criteria (DoD 2005). Also, a literature review of the different analyses of progressive collapse, the software used for those analyses, and the robustness of steel frame buildings will be surveyed. Finally, previous research works on the progressive collapse of steel frames will be noted.

Chapter 3 describes the studied models and their properties and assumptions. In addition to the methodology, a flow chart for the analytical work and an overview of the program used are discussed.

Chapter 4 presents the analytical results from using the (ELS®) software program for the reference model and other models of different bay areas before and after upgrading. Also, the proposed curves to predict chord rotation, tie forces and displacement ductility demand on the beams are discussed.

Chapter 5 presents summary of the study and conclusions, in addition to noting future research that can be done.



(a) (b)

Figure 1.1. Two different progressive collapse events.

a) Ronan Point collapse (1968).

b) Progressive collapse of World Trade Center towers (2001).

Chapter 2

Literature Review

2.1 INTRODUCTION

In this chapter, building code requirements for prevention of progressive collapse are discussed in section 2.2. Then, ways of modeling and analyzing progressive collapse are presented in section 2.3. After that, connections in steel frames and the innovative connection that has been developed to mitigate the behavior of steel frames are discussed in section 2.4. Finally, past studies on the progressive collapse of structures are surveyed in Section 2.5.

2.2 CURRENT PROVISIONS IN CODES FOR PREVENTING PROGRESSIVE COLLAPSE IN STRUCTURES

The current philosophy of most of the present building codes is to design structures for loads that may occur during their lifetime. Structures are not usually designed for abnormal events such as explosions due to ignition of gas, vehicle impact, blast effects, etc., which can cause catastrophic failure. Most of the mainstream codes have only general recommendations for mitigating the effect of progressive collapse in structures that are overloaded beyond their design loads. A number of codes, standards, and design guidelines (both national and international) include discussions on the prevention of progressive collapse. The most practical guidelines are those issued by the General Services Administration (GSA) and the Department of Defense (DoD) in the United States, which will be discussed in the next section. Afterwards, the general building codes that give recommendations or guidelines for mitigating the effect of progressive collapse on structures will be presented.

2.2.1 US Government Documents

The US federal government has developed approaches that address the prevention of progressive collapse in building design; these approaches have been developed by the General Services Administration (GSA) and the Department of Defense (DoD). The kind of design guidance provided by these two organizations represents the most important information in the US currently available on this topic. The GSA and DoD requirements apply to new and existing buildings susceptible to progressive collapse. In the following two sections, these two guidelines will be presented in more detail.

2.2.1.1 Overview of the GSA guidelines

The General Services Administration (GSA) guidelines state that redundancy, detailing to provide structural integrity and ductility, and capacity for resisting load reversal need to be considered in the design process to make a structure more robust and thus enhance its resistance to progressive collapse.

The GSA classifies structures as either typical or atypical. Typical structures have a regular plan layout, and atypical structures have irregular planar variations in bay areas or a combination of different systems, such as frames and walls. In this study, the models used are classified as typical structures.

The GSA guidelines propose only the Alternative Path Method (APM) for evaluating the hazard of progressive collapse. The Alternative Path Method (APM) can be conducted by linear-elastic static, nonlinear static, linear-elastic dynamic, and nonlinear dynamic analyses using a suitable software package, which will be discussed in more detail in section 2.3. It stipulates an analytical procedure for removing vertical load-bearing elements to assess the potential for progressive collapse to occur in a structure.

The Alternative Path Method (APM) is used to ensure that structural systems have adequate resistance to progressive collapse. The APM is a threat-independent methodology, which means that it does not consider the type of triggering event, but rather takes into account the building system's response after the triggering event has destroyed critical structural members. If one component fails, alternate paths are available to bear the load and a general collapse does not occur. The methodology is generally applied in the context of a "missing column" scenario to assess the potential for progressive collapse and is used to check if a building can successfully absorb loss of a critical member. The technique can be used for the design of new buildings or for checking the capacity of an existing structure. In the column-removal scenario, the column is removed from the model directly below the joint without affecting the connection between the beams and the removed column, as seen in Figure 2.1. This procedure in a typical structural configuration for exterior column or internal column removal is done on the ground floor only in the GSA guideline.

The main method of analysis in the GSA guideline is elastic static analysis, which is used for low- and medium-rise structures, that is, those with ten or fewer stories above the grade and a typical structural plan. For buildings taller than ten stories above the grade or for atypical structures, the recommended means of analysis is the non-linear method.

To evaluate the result of a linear elastic analysis the demand capacity ratio (DCR) is calculated as = Q_{ud}/Q_{ce} = (acting force determined incompetent / ultimate unfactored capacity of the component). If the DCR exceeds 1.0, the member or connection exceeds the ultimate capacity at that section. If the flexure DCR for a beam exceeds 1.0, then a redistribution of moment will occur and there will be no failure. But if the DCR value for shear exceeds 1.0 at any section of the member, then there will be failure because of brittleness in the shear. Also, if the allowable DCR for flexure is exceeded at both ends and at mid-span, the member fails because of mechanism; and in the next iteration, this member is removed from the analysis. For steel elements, the GSA uses DCRs between 1.0 and 3.0, as shown in Table 2.1. Thus, the design might be either overly conservative or not conservative enough, depending on the DCR value being used.

If dynamic analysis is performed, the applied load combination is equal to the dead load plus 25% of the live load through the whole floor; and the time for removal of the element is less than 0.1 of the period for that element to undergo the effect of sudden removal. On the other hand, in static analysis this load is multiplied by 2 to take into account the dynamic impact of removal of the member, and this load is applied throughout the whole plan on all beams and slabs. Also, in any analytical method the over-strength of the material can be taken by increasing 25% for concrete compression strength and reinforcing steel.

When performing nonlinear analysis, damage criteria must be established to predict the collapse of a structure. Table 2.2 shows the empirical values of damage criteria for typical elements. If the damage criteria are not satisfied, then the structure is susceptible to progressive collapse. The response of the structure can be improved by enhancing the entire building. This enhancement takes the form of increasing steel plates to increase the strength and stiffness of the beams or using FRP that increase the strength only.

The GSA guideline also gives requirements on the maximum allowable collapse area or limit of damage that can occur if one vertical member collapses. The limit of damage for removal of an external element is the smaller of the structural bays directly associated with and above the removed member or 1800 ft^2 of the floor directly above the removed member. For the removal of an internal element, the structural bays directly associated with and above the removed element or 3600 ft^2 of floor area directly above the removed vertical member. Figure 2.2 shows the maximum allowable collapse area if an exterior or interior column fails.

2.2.1.2 Overview of DoD guidelines (UFC 4-023-03)

The Department of Defense (DoD) guideline uses two approaches for design which are a direct method, such as the Alternate Path Method, and an indirect method, such as the tie force method or the local resistance method. The Alternate Path Method (APM) is the same as that used in the GSA guidelines. The new methods introduced in the DoD guidelines are the tie force method and the local resistance method.

Design approach depends on the required level of protection for the facility. The DoD divides buildings into Very Low Level Of Protection (VLLOP), Low Level Of Protection (LLOP), Medium Level Of Protection (MLOP) and High Level Of Protection (HLOP). The tie force design method is used for very low and low levels of protection, which apply to the majority of buildings, while both indirect and direct design are used for medium and high levels of protection. Direct design, which is the Alternative Path Method, is more accurate; and this method must be used for medium and high levels of protection and for low level protection if the column or wall cannot provide the required vertical tie force. The Alternate Path Method is sometimes used after it has been shown that the minimum tie forces required by the indirect method cannot be developed. The designer may then apply the Alternate Path Method to determine whether the structure can bridge the deficiency.

The local resistance method is another way to provide elements with sufficient strength to prevent failure under an extraordinary loading event, such as blast pressure or impulse, and it designs the members to withstand this kind of loading.

The over-strength factor is used in the tie force capacity and alternative load methods. As in the GSA guidelines, this factor is equal to 1.25 for concrete compression strength and reinforcing steel.

2.2.1.2.1 Tie forces (Indirect method)

Indirect design involves the concept of tie forces. In this approach, ties capable of resisting a minimum level of force are provided to enhance a structure's redundancy, continuity and ductility. This system of ties helps keep the structure from collapsing.

All buildings must be effectively tied together at each principal floor level. Each column must be effectively held in position by means of horizontal ties in two directions, approximately at right angles, at each principal floor level supported by that column. Horizontal ties should be arranged in continuous lines wherever practical and distributed throughout each floor and the roof level in two perpendicular directions. Every steel member should act as a horizontal tie, and their connections should be capable of resisting a tensile force. Figure 2.3, taken from UFC 4-023-03 (DoD 2005), illustrates the different types of ties that are typically incorporated to provide structural integrity in a frame.

1- Internal ties

The internal Steel beams must be designed to act as internal ties, and their end connections, to be capable of resisting tie force, as in the following equation:

Internal tie force = $0.5\,(1.2D + 1.6L)\,S_t\,L_l$ but not less than 75 kN (2.1)

Where: D = Dead Load (kN/m^2)

L = Live Load (kN/m^2)

L_l = Span (m)

S_t = Mean transverse spacing of the ties beams adjacent to the ties being checked (m)

2- Peripheral ties

The external steel beams must be designed to act as peripheral ties, and their connections, to be capable of resisting tie force, as in the following equation:

$$\text{Peripheral tie force} = 0.25\ (1.2D + 1.6L)\ S_t\ L_l \text{ but not less than } 37.5 \text{ kN} \quad (2.2)$$

3- Horizontal ties to columns

The horizontal ties are internal steel beams that are connected to external columns or walls. The required tie strength for horizontal ties anchoring the column nearest to the edges of a floor or roof and acting perpendicular to the edge is equal to the greater of the loads calculated in Equation (2.1) or 1% of the maximum factored vertical dead and live load in the column that is being tied, considering all load combinations used in the design.

4- Vertical ties to columns

All columns must be continuous through each beam-to-column connection. All column splices must provide a design tie strength that is equal to the largest factored vertical dead and live load reaction (from all load combinations used in the design) applied to the column at any single floor level located between that column splice and the next column splice down or the base of the column.

2.2.1.2.2 Alternate Path Method (APM) (direct method)

In alternative path analysis, three-dimensional modeling is preferred when analyzing to capture more accurately the behavior of the structure. There are three analytic methods to be used: linear static, nonlinear static, and nonlinear dynamic, which will be discussed later. For HLOP and MLOP, the ALP method is mandatory to prove structural resistance to progressive collapse, while the horizontal and vertical tie force capacity must be satisfactory.

Element removal is controlled by several factors, including location of the threat (external or internal), type of structural system, and building layout. An exterior perimeter column is more susceptible to failure, especially when the detonation of an explosion is from outside the building. On the other hand, internal element removal is required under situations in which there is underground parking or uncontrolled public ground-floor space.

For external column removal, three cases of removed columns are considered for each floor, which are the column at the corner, the column at mid-distance of the short edge, and the column at mid-distance of the long edge of the building, as seen in Figure 2.4. Also, columns should be removed at any location where significant changes in the plan's geometry indicate differences in bay areas and re-entrant corners. For internal column removal there are also three cases, but it is done only on the ground floor, as seen in Figure 2.5.

In the case of nonlinear dynamic analysis, the whole floor is loaded with the following load:

$$\text{Load} = (0.9 \text{ or } 1.2)\ D + (0.5\ L \text{ or } 0.2\ S) + 0.2\ W. \qquad (2.3)$$

But in the case of linear and nonlinear static analyses, there is a magnification factor of load equal to (2) for dead and live loads, only to take into account the sudden removal of an element. This amplified load is applied to the bays immediately above only the removed element at the floor and not to the entire floor as in the GSA guideline.

The limitation of damage in DoD for removal of an element is given for both external and internal elements. For the removal of an exterior primary support element, the maximum damage area is the smallest of 750 ft^2 of the floor area or 15% of the total area of the floor directly above the removed element. For the removal of an interior primary support element, the maximum damage area is the smallest of 1500 ft^2 of the floor area or 30% of the total area of the floor directly above the removed element.

2.2.2 General Building Codes

Many structural design standards in North America and Western Europe have acknowledged for some time the existence and potential consequences of abnormal loads and progressive collapse. Most standards contain a statement of required structural performance for a new construction, to the effect that local damage to the structure shall not have catastrophic consequences. A summary of features from design standards and guidelines in the United States, Canada, and Western Europe that are significant for risk-based general design requirements is presented here.

In Canada, Commentary C on Part 4 of the National Building Code of Canada (NBCC) requires structures to be designed for sufficient structural integrity to withstand all effects that may reasonably be expected to occur during the building's service life, and advises the designer to consider and take measures against severe accidents with probabilities of occurrence of 10-4/yr or more, leaving it to the designer to determine appropriate measures. The figure 10-4/yr serves as a warning to consider the consequences of low probability events and distinguishes the NBCC from most other standards and guidelines that have not adopted a specific threshold. Also, the American Concrete Institute's Building Code Requirements for Structural Concrete 2008 (ACI 318-08) as well as the National Building Code of Canada 2005 (NBCC 2005) rely on structural integrity requirements to prevent progressive collapse of structures. These requirements are based on the assumption that improving redundancy and ductility can help to localize damage so that it will not propagate to other members, and thus the overall stability of the structure can still be satisfied.

ASCE Standard 7, Minimum Design Loads for Buildings and Other Structures, (ASCE 7, 2005): ASCE-7 is the only mainstream standard which addresses the issue of progressive collapse in some detail. Progressive collapse is defined in its commentary as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it". It emphasizes the need to protect the structure against extreme events which can result in progressive collapse and gives two design alternatives to resist progressive collapse: the direct design method and the indirect design method. In the direct design method, the resistance to progressive collapse is considered directly during the design process through (a) the Alternate Path Method (APM), which seeks to provide an alternate load path after a local failure has occurred so that the local damage is arrested and major collapse is prevented, and (b) the Specific Local Resistance Method (SLRM), which seeks to provide sufficient strength to resist failure on the "key" element to prevent the failure of a structural member. The indirect design method implicitly considers the resistance to progressive collapse through provisions of minimum levels of strength, continuity, and ductility. It also provides guidelines for the provision of general structural integrity and stresses the need to provide ductile connections between the structural components that can undergo large deformations and absorb large amounts of energy under the effect of abnormal conditions.

The UK Building Regulations have led with requirements for the avoidance of disproportionate collapse. These requirements, which are refined in material-specific design codes (e.g. BS5950 for structural steel-work), can be described as (i) prescriptive "tying force" provisions, which are deemed sufficient for the avoidance of disproportionate collapse, (ii) "notional member removal" provisions, which need only be considered if the tying force requirements cannot be satisfied, and (iii) "key element" provisions applied to members whose notional removal causes damage exceeding prescribed limits. Finally, Euro code No. 1 states that a structure shall be "designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause."

2.3 METHODS AND TOOLS FOR ANALYSIS OF PROGRESSIVE COLLAPSE

There are different methods for analyzing progressive collapse and different software packages that can be used for progressive collapse analyses. In this section both the analytical methods and the tools used for progressive collapse will be discussed.

2.3.1 Methods for Analysis of Progressive Collapse

Four different analytical methods are presented herein that are used in progressive collapse analysis. They are the linear and nonlinear static analysis methods and the linear and nonlinear dynamic analysis methods

• Linear static analysis is the fastest and easiest to perform, but it does not consider the dynamic effect and any nonlinearity effects due to material and geometric nonlinearity. Also, this analysis is only applicable to analysis of structures with simple and regular configurations.

When analysis by the linear static method is performed, many iterations are done. Each iteration checks that no plastic hinges occurred or members have failed. Formation of plastic hinges occurs at a section where the moment from the analysis exceeds the nominal flexure moment. The analysis is repeated after a hinge is inserted and the nominal moment on the face of the column is applied. After that, if three plastic hinges are formed (two at the ends and one at mid-span), then this member is considered to have failed and is removed from the model, and the redistributed load is carried to the adjacent members.

• Nonlinear static analysis takes into account the effects of material and geometric nonlinearity but does not consider the dynamic effect directly in the analysis. The procedure is relatively simple yet gives sufficient important information about the behavior of a structure. In nonlinear static analysis, only a single iteration is required. It must be verified that the shear capacity is adequate to develop the full design flexure strength. In all cases, the load associated with a failed member has to be distributed to an area equal to or less than the size of the original area. The nominal shear strength is calculated using the material over-strength factor and compared to the design shear strength. If the design shear strength is greater than the nominal shear strength, then the member is considered to have failed. In addition, deformation of the member is checked. If it is found that this member exceeds the allowable maximum values, then it is considered to have failed and is removed and iteration is conducted.

• Linear dynamic analysis includes the dynamic behavior of the structural response, but it does not consider the effects of material and geometric nonlinearity. It may not give good results if the structure exhibits large plastic deformations.

• Nonlinear dynamic analysis gives the most exact results and includes both material and geometric nonlinearity and dynamic effects in addition to the dynamic effect of the removal of a column, but the practical application is rigorous and time consuming. This method is often used as a verification to supplement results obtained from other methods. When a structure undergoes progressive collapse, the response of the structure is affected by dynamic effects. This requires the dynamic behavior of a structure to be taken into account in the progressive collapse analysis. It is also expected that nonlinear structural behavior can significantly affect the progressive collapse behavior of a structure since before reaching collapse a structure and its member components must have exceeded its elastic limits. This method needs an expert engineer and good software package to give reliable results. Also, the accuracy depends on many factors, such as time step, time of removal of the column, and damping ratio of the structure.

It can be concluded that among the available analytical methodologies, nonlinear static analysis and nonlinear dynamic analysis are the two most appropriate methods for evaluation of the progressive collapse behavior of structures. In nonlinear static analysis, dynamic effects in the responses are not considered directly. Despite this limitation, experience has shown that the results obtained by nonlinear static analysis can still provide valuable insights into the behavior of the analyzed structure, and the results tend to be conservative in most cases. The attractiveness of this method is its simplicity compared to the nonlinear dynamic analysis approach. Studies have shown that nonlinear static analysis methods can give good approximations of deformation demands, identify discontinuities in strength, and assess the global stability of structural systems. Nonlinear static analysis has also provided good estimates for the seismic demands of structures. Therefore, the nonlinear static analysis procedure is a valuable alternative method to the more rigorous nonlinear dynamic method for analysis of the progressive collapse behavior of structures.

Using the nonlinear static analysis procedure, a capacity curve of a structure can be generated by pushdown analysis. Pushdown analysis is conducted by applying the gravity load combination in nonlinear static analysis in a way similar to that done in pushover analysis. A capacity curve provides insight as to whether or not a structure has adequate capacity to resist the loading condition. In addition to that, from the capacity curve, the yield deflection that can be used to calculate the ductility of the member can be determined. During progressive collapse, dynamic properties of a structure change after failure of one or more members in the system. Multiple pushdown analyses may be required to capture the progression of the collapse mechanism if the analytical tool employed in the simulation does not specially model and capture the progressive changes in the structural properties and behavior of the system.

2.3.2 Analytical Tools

A number of commercial programs are available for the analysis and design of structures to resist progressive collapse. Most of the commercial programs have the capability of performing nonlinear dynamic analysis, while only a few of these programs can perform dynamic analyses of structures from initial damage up to, and through, collapse. These programs are not efficient in computing structural response for collapse of a frame resulting from member failure and subsequent load redistribution.

Many software packages are available that can be utilized for this purpose, and some even have specific options for the progressive collapse of structures. Researchers have used general finite element software packages for frame structures such as SAP 2000, STAAD Pro, PERFORM 3D, and OpenSees to assess the progressive collapse behavior of structures. The finite element analysis software packages for continuum systems, such as ANSYS and ABAQUS, have also been used. These analytical tools typically assume the analyzed structures remain continuous, meaning that even if a collapse occurs the structure will still maintain its continuity due to the connection between the elements through nodes. Separation of nodes leads to singularity in the stiffness matrix and consequently to error. The collapse mechanism is represented through the behavior of plastic hinges formed due to flexural overstress on members. Using these analytical tools, the effects of member separation due to fracture can be approximated by removing specific failed individual members from the analytical model to assess the capability of other members to withstand progressive collapse. In other words, these software packages can perform the analysis as specified in the provisions and requirements of many current codes and standards discussed earlier.

There are several commercial programs that can be used to analyze the dynamic failure behavior of structures, including LS-DYNA3D, ABAQUS Explicit, and FLEX. Kaewkulchai (2003) stated that LS-DYNA3D program is an explicit nonlinear finite element code specifically designed for use in analyzing the transient, dynamic response of solids and structures. LS-DYNA3D is routinely used by structural engineers to study blast effects on structures in which modeling of walls and slabs by finite elements are of major interest. The capabilities of the software suggest that it may be appropriate for progressive failure analysis of frame structures. However, because it was designed primarily to be used for shock and impact problems along with the use of an explicit solver, the software is not efficient in computing structural response for long time durations such as might be expected for collapse of a frame resulting from member failure and subsequent load redistribution. While FLEX is an explicit, large-deformation, transient analysis, the finite element code is used to compute the response of structures subjected to dynamic loads such as air blast, fragmentation, and ground shock loadings. The software contains a library of constitutive models and finite elements that can be used to analyze a variety of structures through failure. Similar to LS-DYNA3D, FLEX uses a set of finite elements for structural modeling along with the use of an explicit solver.

Although explicit codes can be efficient because they do not require factorization of the system equations with each time step (provided the mass matrix is constant), they can be computationally inefficient for problems of long duration or those in which the mass matrix changes over time. Unlike implicit codes, explicit methods of numerical integration (e.g. the central difference method) are conditionally stable and require a smaller time step size for acceptable results. Therefore, explicit codes may not be efficient for collapse analyses of a frame, as discussed previously (Chopra 2002).

Another finite element code was also developed by Toi and Isobe (1993) to expand the current finite element analysis with the so-called Adaptively Shifted Integration (ASI) technique. Their analytical method takes into account plastic collapse of framed structures using linear Timoshenko or cubic beam element formulations. The basis of this ASI technique is shifting the numerical integration points for the calculation of stiffness matrices immediately after the occurrence of plastic hinges. A later application of this method was also applied for seismic collapse analysis of framed structures. This method can also account for debris loads by considering the contacts of the elements in the analysis.

Modeling a progressive collapse mechanism properly by computer building of a model should begin with the structure fully intact and then specify the structural elements that should be instantaneously removed. However, most programs are incapable of analyzing the change in their geometry and stiffness matrices. A more sophisticated software package was developed using the theory of Applied Element Method (AEM). This software is called Extreme Loading for Structures (ELS) (technical manual 2006), and will be used in this study. The structure is modeled as 3D elements connected to each other by springs to represent the stresses, strains, deformations, and failures of a certain portion of the structure. This analytical tool considers explicitly the effects of element separation and discontinuity as well as debris loads caused by collapsed members and the resulting inertia impact of load effects. Studies have shown that ELS based on the AEM theory can give good estimations for large displacements and deformations of structures undergoing progressive collapse. This software uses an implicit method in numerical integration, which is much better for modeling the structural collapse of buildings than other kinds of software that use an explicit method in numerical integration.

2.4 ROBUSTNESS OF STEEL FRAME

Steel frame is used in building up to 25 floors. It contains three components: beams, columns, and connections. In scenarios involving the loss of one column to study the response of the beams of steel moment resisting frames, it became critical to study more than one column so that each column could carry an additional axial load that it could sustain without failure. Therefore, the connections and beams become more important and need to be upgraded to prevent failure of the structure. This study is concerned with the upgrading of beams only. It is believed that the connections have enough ductility and strength to resist the loss of a column. Figure 2.6 shows a sketch of a steel frame before the loss of a column and its response after losing one column.

To reduce the risk of progressive collapse in the event of loss of structural elements, the following structural traits need to be incorporated into the design. Collectively, they produce "robust" structures capable of limiting the spread of damage due to an initiating event.

• *Redundancy*: The incorporation of redundant load paths in the vertical load carrying system helps to ensure that alternate load paths are available in the event of local failure of structural elements.

• *Ties*: The loss of a major structural element typically results in load redistributions and member deflections. These processes require the transfer of loads throughout the structure (vertically and horizontally) through load paths. The ability of a structure to re-distribute or transfer loads along these load paths is based in large part on the interconnectivity between adjacent members. This is often called "tying a building together" by using an integrated system of ties in three directions along the principal lines of structural framing.

• *Ductility*: In a catastrophic event, members and their connections may have to maintain their strength through large deformations (deflections and rotations) and load redistributions associated with the loss of key structural elements. For steel structures, ductility is achieved by using steels with high toughness, maintaining overall and local structural stability, and creating connections between elements that exceed the strength and toughness of the base material.

2.4.1 Beams and columns in a steel frame

Beams resist loads by two actions, verandial or moment-resisting action and catenary or axial-force resisting action, as shown in Figure 2.7. Beams resist loads by moment in the case of a small deflection (verandial action); but in the case of large deflections, as in the loss of columns scenario that occurs in progressive collapse analysis, the beam starts to exert axial tension force, which is tie force (catenary action), to resist the collapse of the building. Figure 2.8 shows catenary action in a floor system. Beams do not instantaneously switch from regular moment resistance to catenary action. There is a gradual shift in which more and more axial tension is applied as the deflection increases, as seen in Figure 2.9. In order to achieve that, the beams must be capable of resisting the tension force developed, and the connections must be capable of undergoing high inelastic rotation and high rotation capacity and resist the tie force developing in the beams.

Catenary action can be modeled by a finite element software package like SAP2000, which is set to analyze under the P-delta with large displacement. The catenary action can mitigate failure and protect structures from progressive collapse. Most progressive collapse analyses ignore catenary action as being more conservative, which is not an ideal solution. In this study, the tie force (tension force) exerted in beams is calculated by the software that has been developed for high deflections.

Column axial load-carrying integrity is significantly increased by using tube steel or built-up box columns filled with high-slump lean concrete to effectively reduce the slenderness ratio $K.l/r$ of the column and thereby preclude column buckling and instability.

Each column splice should be capable of resisting a tensile force equal to the 1.2 Dead Load plus 0.5 Live Load reaction applied to the column at a specified number of floor levels located immediately below that column splice or a percentage of the column capacity.

2.4.2 Connections in a steel frame

In steel frames, there are three types of connections: pin connections, rigid connections, and semi-rigid connections. Pin connections do not transfer moment between the columns and the beams; there is also relative rotation between them. But rigid and semi-rigid connections transfer moment between the columns and the beams. The difference between semi-rigid and rigid connections is that semi-rigid connections make for relative rotation between columns and beams, while rigid connections do not. Where rigid connections are used to mitigate progressive collapse, pinned or semi-rigid connections are not used.

It is very common practice to use rigid or pinned connections between steel members for analytical purposes. However, experiments have shown that a real steel connection is neither rigid nor pinned (Kameshki 2003). Furthermore, experiments have also shown that when a moment is applied to a ductile connection, the relationship between the moment and the beam column rotation is nonlinear (Kameshki and Saka 2003), as seen in Figure 2.10.

For retrofitting steel moment frames, the SidePlate™ retrofit system is used (Crawford 2002). This retrofitting scheme physically separates the face of the column flange and the end of the beam, which in turn mitigates premature fracture of the connection. The SidePlate™ system uses parallel full-depth side plates to directly connect the beam and the column, reducing the stress transfer through panel zones. Additional information about this system and other types of upgrades can be obtained from FEMA 351. An illustration of the

SidePlate™ system is given in Figure 2.11. SidePlate™ connection technology inherently eliminates the need for costly traditional internal diaphragm plates in tube steel or box columns at the beam flange elevations. The use of tube steel columns or built-up box columns with the SidePlate™ moment connection system is an approved option.

The unique trademark geometry, simple design configuration, increased connection stiffness, and proven construction methods that collectively characterize the attributes of the SidePlate™ connection system are ideally suited to satisfy a wide variety and technically diverse set of design and construction applications for steel frame structures. The System Pre-Qualification of the SidePlate™ moment connection provides rigorous justification for any practicable combination of rolled-shape beam and column sizes.

SidePlates use of full-depth side plates ensures that all significant energy dissipation/connection deformation occurs for ductility outside the column and connection welds and plates. Rotational performance completely avoids dependence on a column web's weak panel zone participation, thus escaping vulnerability to column kinking and to sudden pre-mature rupture along the rolled fillet between the flange and web. SidePlate™'s inherent increased lateral stiffness on global frame performance is provided by the full-depth side plates. Collectively, these attributes increase the amount of ductility available for connection deformations and thus enhance the performance of the connection under deformations exceeding its elastic limit.

SidePlate™ combines redundant simplified load paths with fast fabrication. Actual load transfer (i.e., load distribution), from beam to side plates and brace to side plates, and from side plates to column, is accomplished by the use of plates and fillet welds loaded in a predictable manner. The ability to identify realistic and redundant load transfer mechanisms provides a clear understanding of the function of each element of the connection, leading to a rational design procedure and reliable performance. The result is an integrated cost-efficient set of ductile steel frame connection solutions that perform with reliable and repeatable ductility, while providing virtually unlimited design expression for both moment frame systems and braced dual systems.

2.5 PREVIOUS RESEARCH ON PROGRESSIVE COLLAPSE

Marjanishvili et al. (2006) modeled a nine-storey moment resisting frame and analyzed this model when an edge column is removed, as shown in Figure 2.12, by performing linear and nonlinear static and dynamic analyses using SAP2000. They found that linear static analysis with a magnification factor for a load of 2 gave a similar response to a linear dynamic analysis, which means that magnification of a load by 2 as stated in the guidelines can be used to take into account the dynamic effect in linear static analysis. They also recommend not to rely on nonlinear static analysis procedure alone, but to use nonlinear dynamic analysis in addition to it.

Ruth et al. (2006) worked on many analytical models and found that a factor of 1.5 or better represents the dynamic effect, especially for steel moment frames, when performing static analysis. On the other hand, the condition stipulated by the DoD and GSA is more conservative, with a factor of 2 to be used only in situations with linear elastic material.

In a recent investigation, Kim et al. (2008) studied the progressive collapse-resisting capacity of steel moment resisting frames using the alternate path methods recommended in the GSA and DoD guidelines. The linear static and non-linear dynamic analysis procedures were carried out for comparison. It was observed that the nonlinear dynamic analysis provided larger structural responses, and the results varied more significantly. However, the linear procedure provided a more conservative decision for progressive collapse potential of the model structures. They studied the response of steel moment resisting frames using alternative load paths with different damage scenarios when a corner, a first edge, and an internal edge column are removed, each done separately. Applying static, nonlinear static and dynamic analyses, they found that nonlinear dynamic is the most precise and that it can be carried out using commercially available software packages. The dynamic analysis results varied more significantly, depending on the variables, such as applied load, location of column removal, or the number of stories in the building. Also, they found that the potential for progressive collapse was highest when a corner column was suddenly removed, but the potential for progressive collapse decreases as the number of stories increases.

Fu (2009) assessed the response of a twenty-storey building subjected to sudden loss of a column for different structural systems and different scenarios of column removal. One of his conclusions is that under the same general conditions, removal of a column at a higher level will induce larger vertical displacement than removal of a column at ground level. Also, the dynamic response of the structure is related mainly to the affected loading area after the column's removal.

Lee et al. (2008) studied the progressive collapse resistance for a welded steel moment frame based on a nonlinear finite element. They found that the beam span-to-depth ratio is the most influential factor governing the catenary action of double-span beams in welded steel moment frames under a column-missing scenario and using a parametric nonlinear finite element.

Khandelwal et al. (2008) investigated seismically designed steel braced frames using validated computational simulation macro-models. Two types of braced systems were considered: concentrically braced frames and eccentrically braced frames in a typical ten-storey building. They found that the eccentrically braced frames are less vulnerable to progressive collapse than the concentrically braced frames.

Kim and Park (2008) introduced a way to design a building to satisfy the given failure criteria using the plastic design method, and they studied three-, six- and nine- storey steel moment resisting frames. They calculated the beam's plastic moment using the virtual work method in the plastic design procedure from equilibrium of the internal and external works triggered by the sudden removal of a column; for illustration, as seen in Figure 2.13.

$$\mathbf{P} \cdot \boldsymbol{\delta} = \sum \mathbf{M}_{\mathbf{pi}} \cdot \boldsymbol{\theta}_{\mathbf{i}} \qquad (2.1)$$

Then the required plastic moment to prevent progressive collapse was calculated:

$$M_p = \frac{P.L}{N} \qquad (2.2)$$

where P = vertical load P acting on the removed column, L = span of the bay , and N = number of plastic hinges.

The applied load from the GSA guidelines is (D.L + 0.25 L.L) and ignores the catenary action of the beams that occurs due to large deformation, which gives more conservative solutions. They also found that the deflection for nine stories is smaller than that for three stories due to the contribution of more beams in resisting the deflection caused by the removal of columns in nine stories, as seen in Figure 2.14. The straight dashed line represents the maximum limit of deflection to consider for the beam to fail, which is 21 cm. As the plastic moment of inertia increases, the maximum deflection decreases and satisfies the limit of deflection. In addition, to prevent strong beam/weak column, the column's dimension must increase in proportion to the beam.

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1 \qquad (2.3)$$

An experimental and analytical progressive collapse research study by Song and Sezen (2009) was conducted on the Ohio Student Union building, which was scheduled for demolition. It was a steel frame building on which linear static and nonlinear dynamic analyses were done for removal of four columns, as shown in Figure 2.15. The researchers found that the building became most susceptible to progressive collapse after the last column was removed. Some columns exceeded the limits of the GSA guidelines when linear static analysis was conducted, and the DCR exceeded 2 after removing four columns, while the DCR for beams did not exceed 2. That means that columns are more critical and subject to progressive collapse than beams in the case of losing four columns. Also, it was shown that nonlinear dynamic analysis results in smaller displacements than linear static analysis.

Powell (2005) used simple energy balance to explain the essential nonlinear behavior of a frame structure following the sudden removal of a column. He illustrated the behavior of the frame structure under loss of columns, as shown in Figure 2.16. The structure does not collapse, provided it has a sufficient combination of strength and ductility. If the strength and/or ductility are insufficient, collapse will occur. In addition, he concluded that the ductility demand is sensitive to the structure's strength. From Figure 2.17 it can be seen that increasing the strength by 5% leads to decreasing the ductility demand by 50%.

Table 2.1. Acceptance criteria for linear procedures in steel frame components

Component	DCR
Steel Beams	3
Steel columns (For 0< P/P_{cl} < 0.5)	2
Steel columns (For 0.5< P/P_{cl})	1
Columns Panel Zone – Shear	2

Table 2.2. Acceptance criteria in nonlinear analysis in GSA 2003 guideline

Component	Ductility (μ)	Rotation (θ)
Steel Beams	20	12
Steel columns (tension controls)	20	12

Table 2.3. Deformation Limits for steel structures in UFC-4-023-03

	Alternative path for LLOP		**Alternative path for MLOP and HLOP**	
Component	**Ductility (μ)**	**Rotation (θ)**	**Ductility (μ)**	**Rotation (θ)**
Beams—Seismic Section	20	12	10	6
Beam—Compact Section	5	--	3	--
Beam—non-Compact Section	1.2	--	1	--
Plates	40	12	20	6
Columns and Beam-columns	3	--	2	--
Steel frame Connections, Fully Restrained	--	--	--	--
Welded Beam Flange or Coverplated(all types)	--	2	--	1.5
Reduced Beam Section	--	2.6	--	2
Steel frame Connections, partially Restrained	--	--	--	--

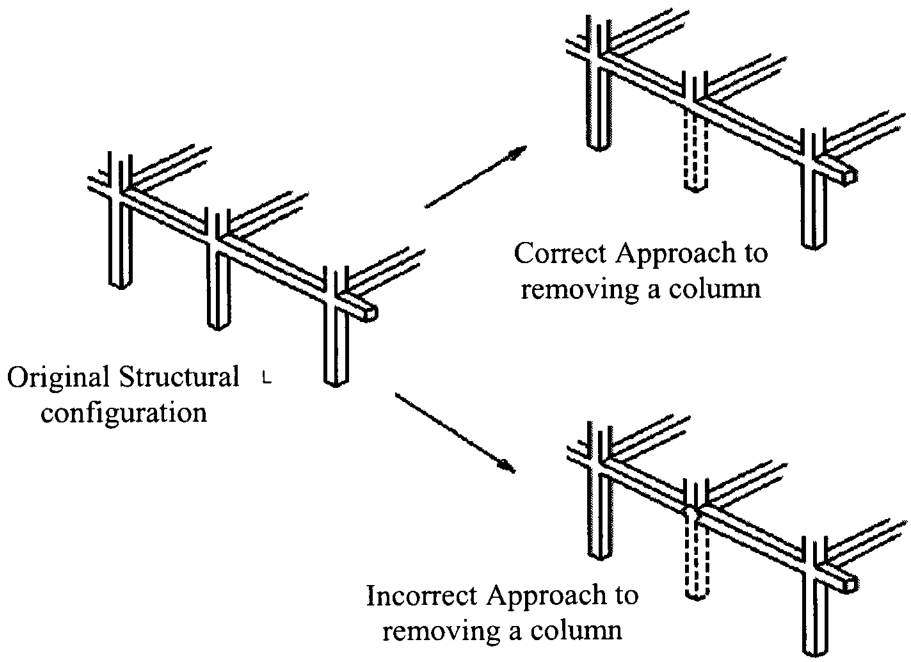


Figure 2.1. Sketch of the correct and incorrect approach for removing a column (GSA 2003).

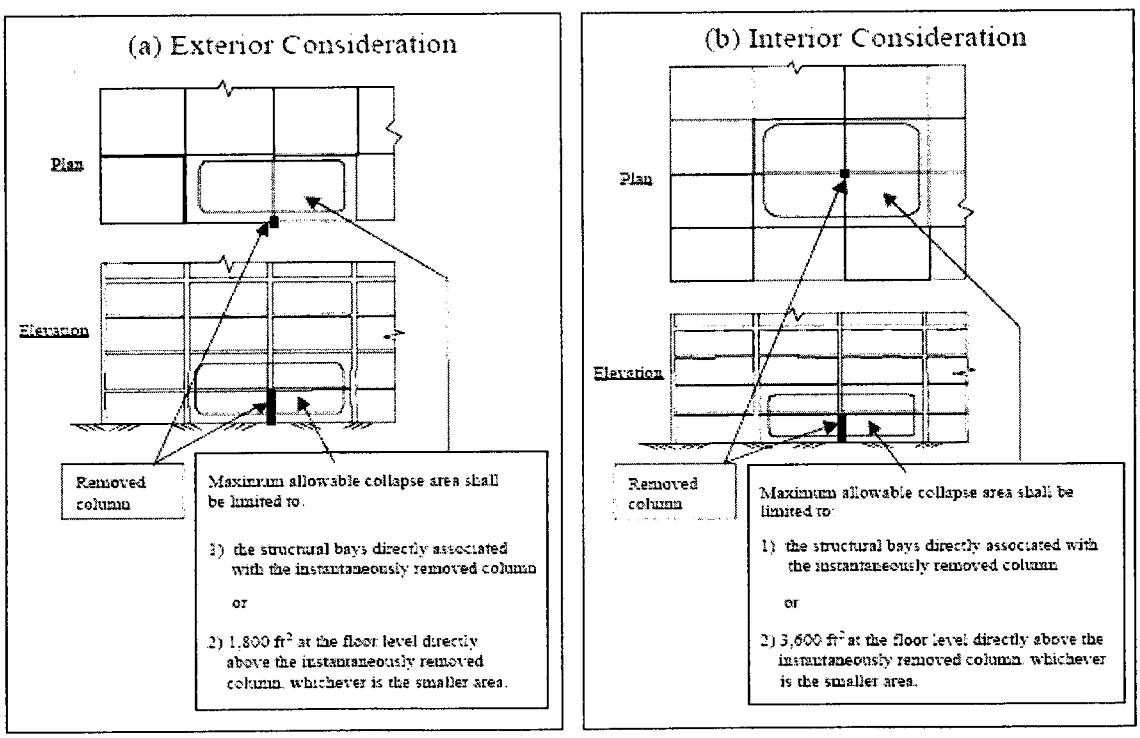


Figure 2.2. Example of the maximum allowable collapse area if an exterior or interior column fails (GSA 2003).

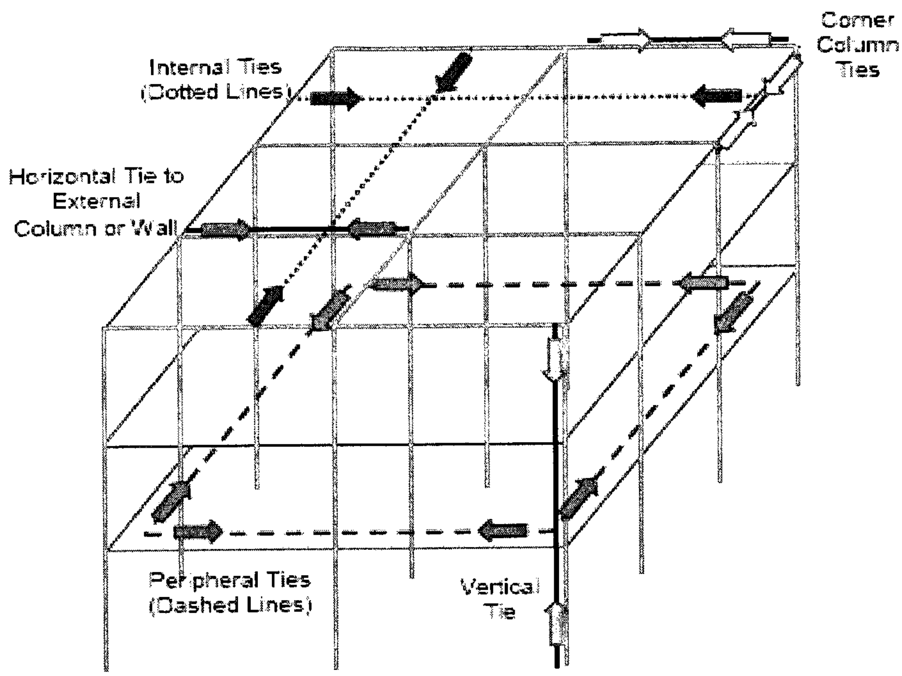


Figure 2.3. Schematic of Tie Forces in a Frame Structure (DOD 2005).

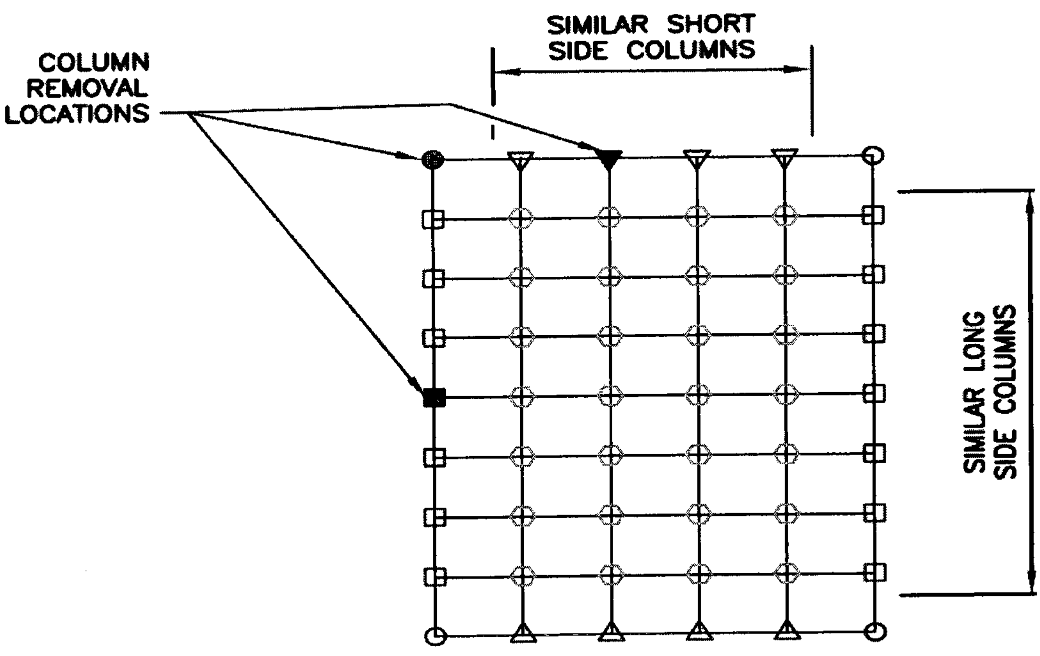


Figure 2.4. Plan view of the Locations of External Column Removal (DoD2005).

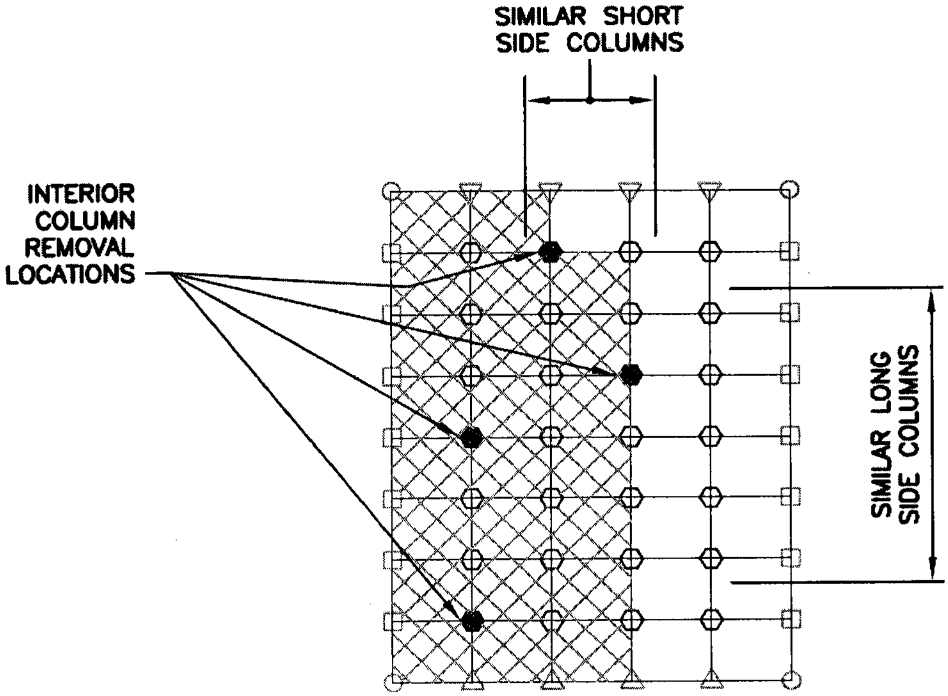


Figure 2.5. Plan view of the Locations of Internal Column Removal (DoD2005).

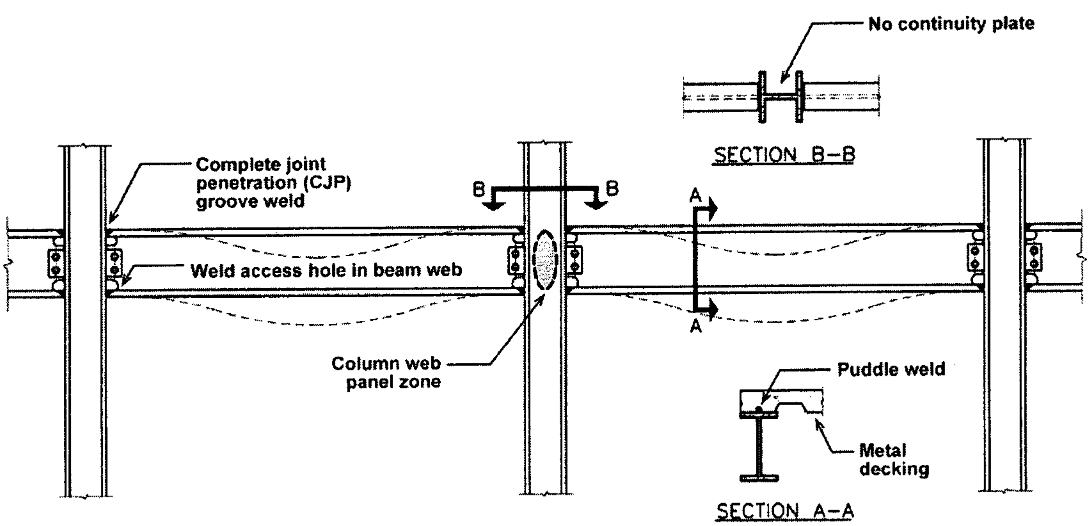


Figure 2.6.a. A sketch depicting a steel frame beam-to-column-to-beam "traditional" moment connection scheme prior to removal of primary column support (GSA2003).

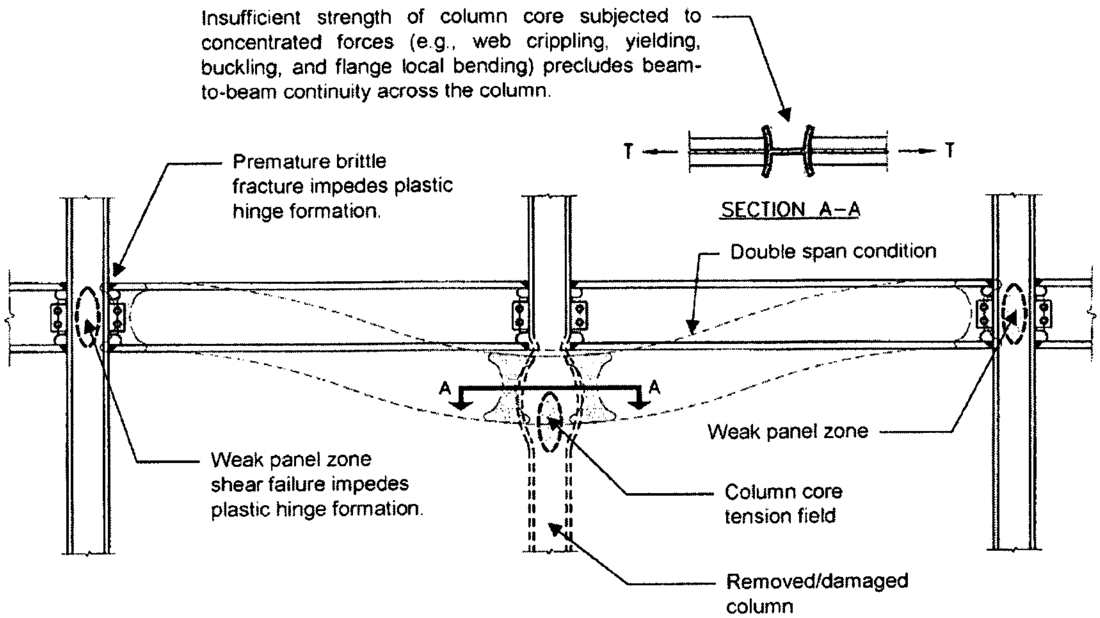


Figure 2.6.b. Response of the framing scheme shown in Figure 2.6.a, after the loss of primary column support, shows the inability to protect against progressive collapse (GSA2003).

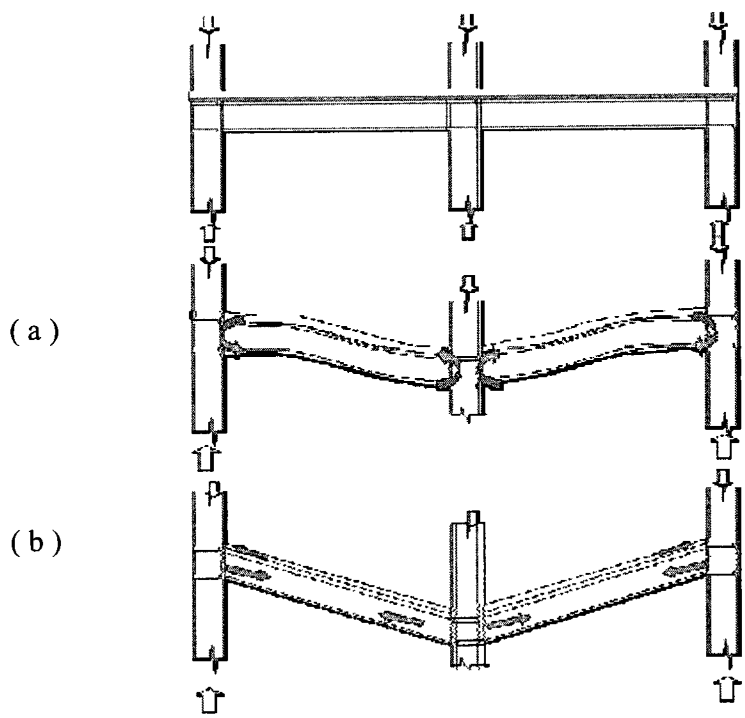


Figure 2.7. Two actions in a beam: a) Verandial action; b) Catenary action.

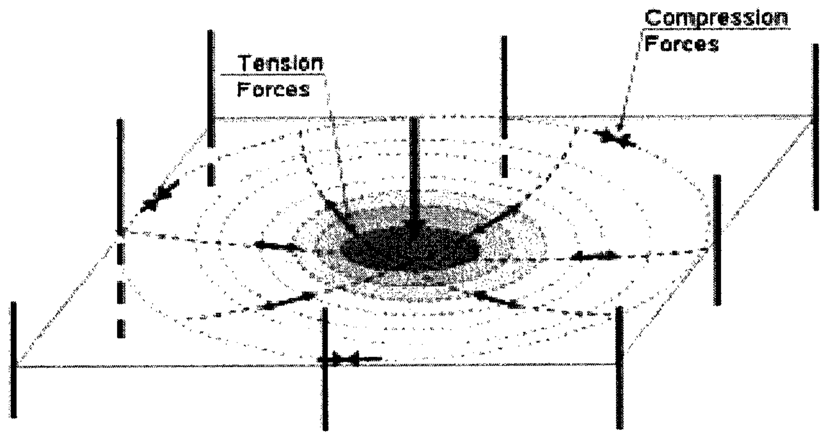


Figure 2.8. Three dimensional view of catenary action in floor system.

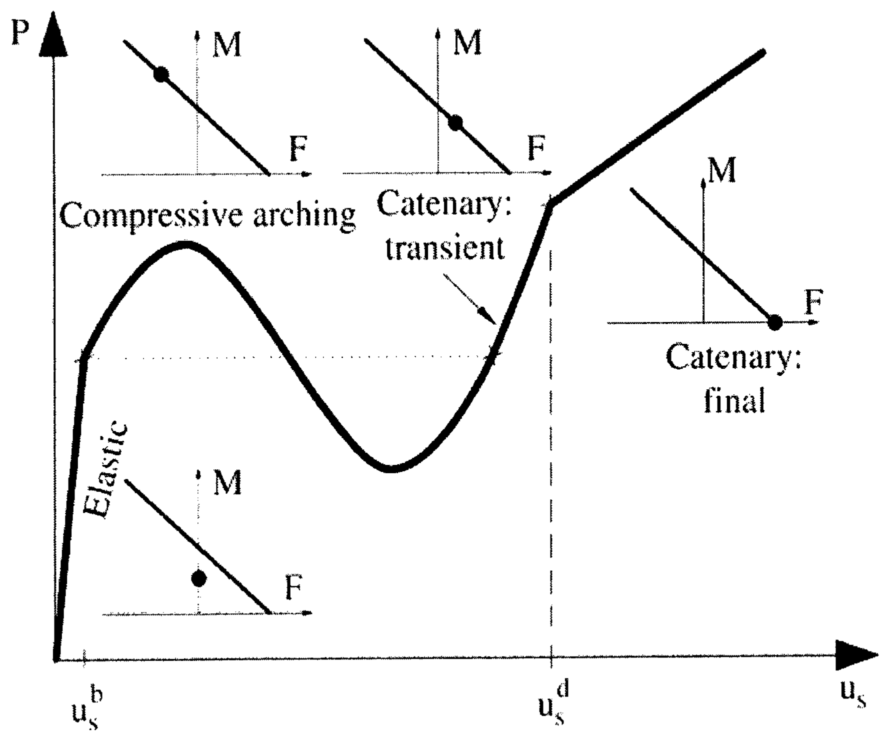


Figure 2.9. Steel frame progression from flexural response to tensile catenary action (Izzuddin et al. 2007).

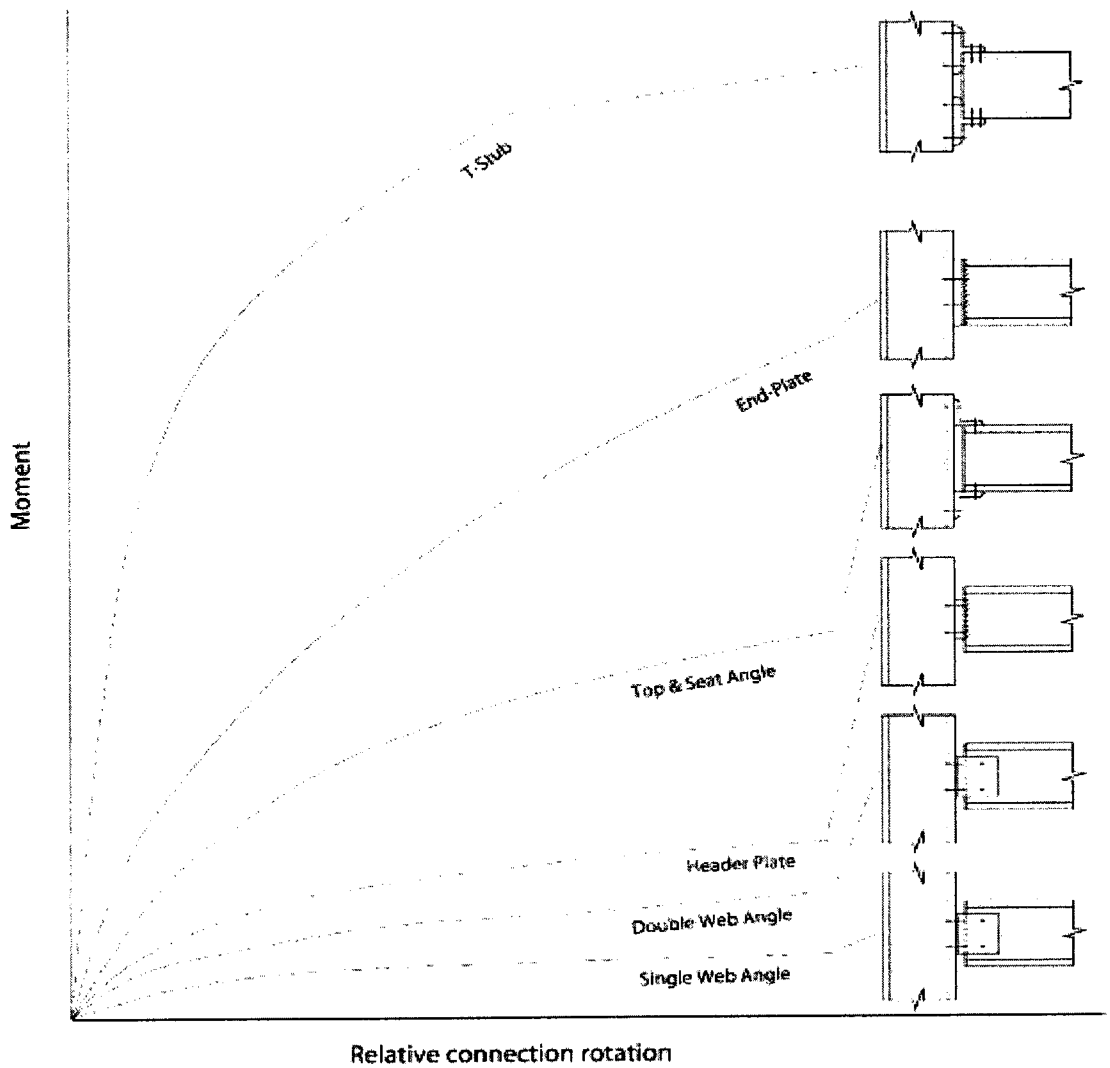
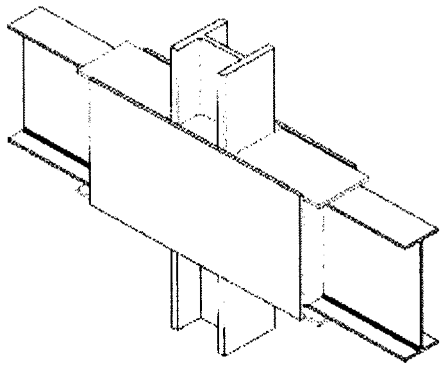
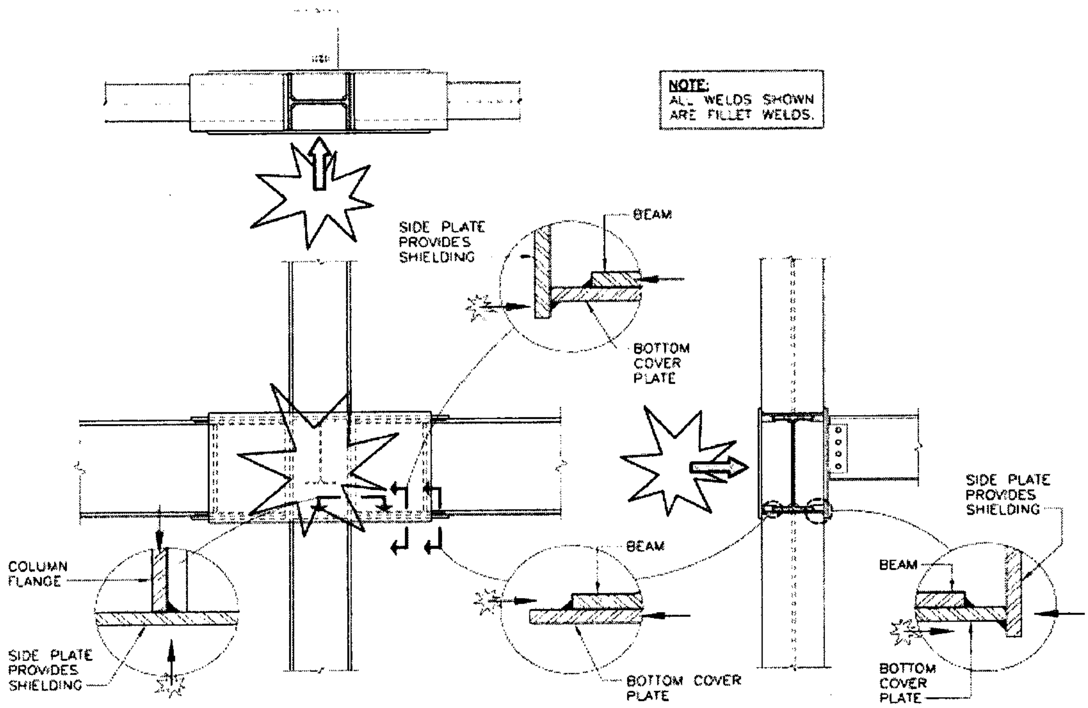


Figure 2.10. Connection Moment-Rotation Curve (Kameshki and Saka 2003).



(a)



(b)

Figure 2.11. Sideplate™ steel connection for mitigation of progressive collapse and blast protection: a) Isometric view, b) Elevation, plan and side view (Crawford 2002).

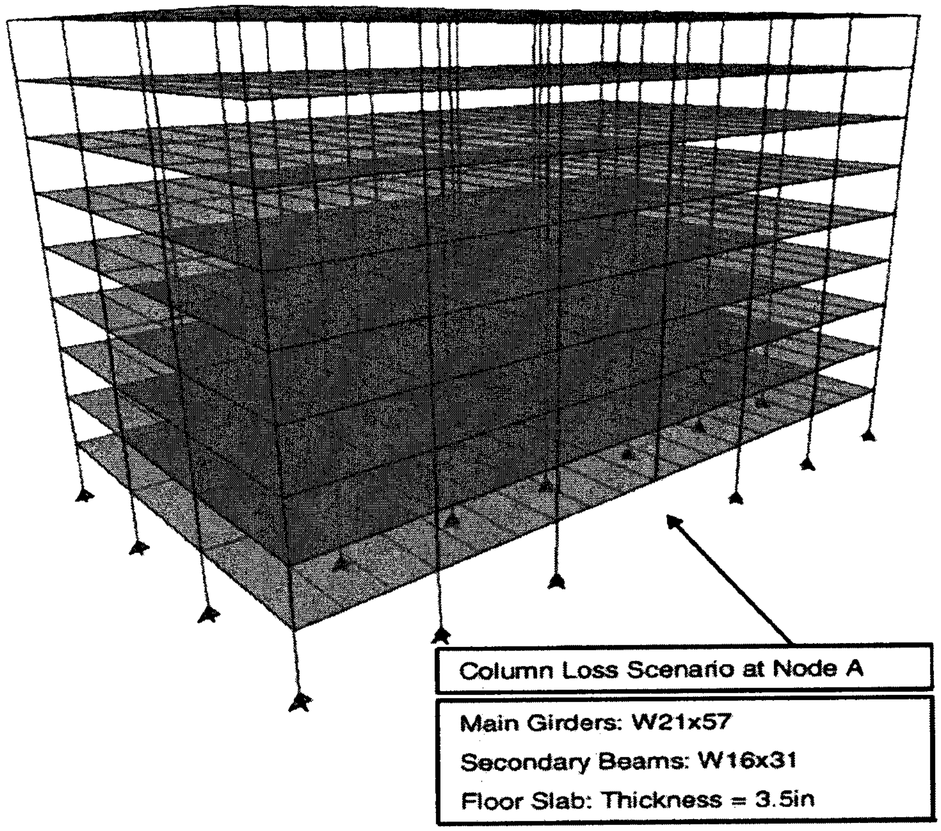


Figure 2.12. Three-dimensional model created in SAP 2000 by (Marjanishvili et al. 2006).

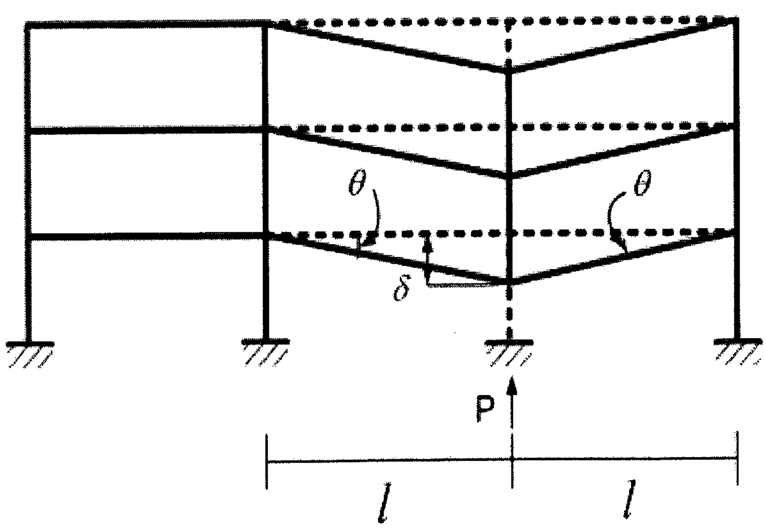
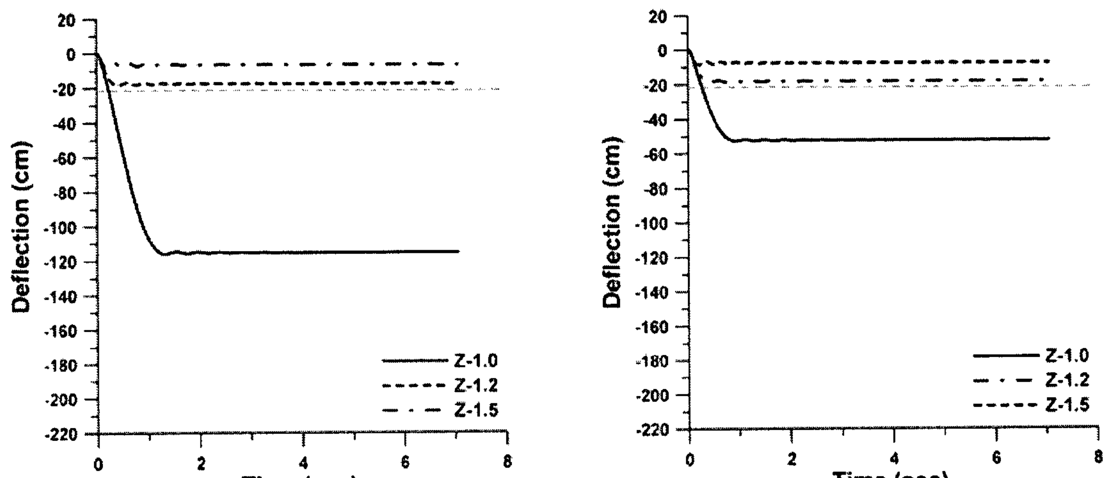


Figure 2.13. Illustration of virtual work method on model of three-storey building (Kim et al. 2008).



(a) 3-story structure (b) 9-story structure

Figure 2.14. Time history of vertical deflection at the lost column-beam connection of model structures with various beam plastic moments of inertia (Kim et al. 2008).

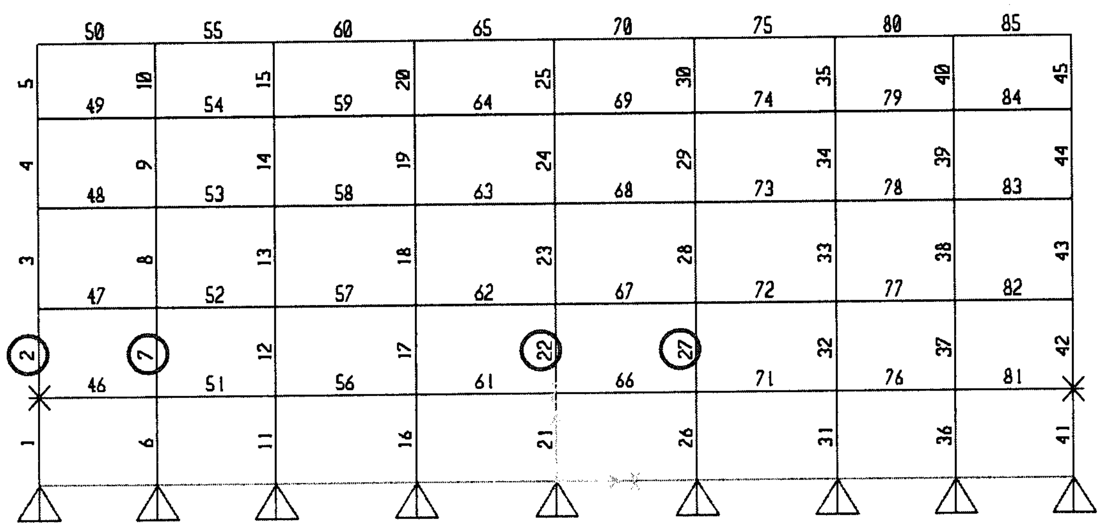


Figure 2.15. The 2D frame model (circled columns are removed) studied by Song and Sezen (2009).

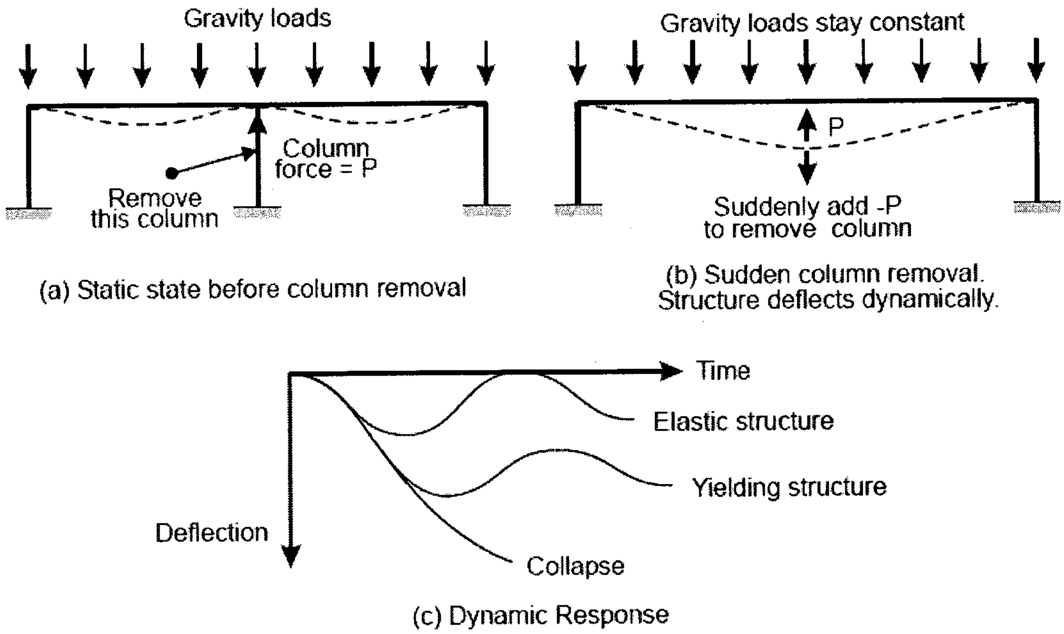


Figure 2.16. Simple structure to illustrate behavior (Powell 2005).

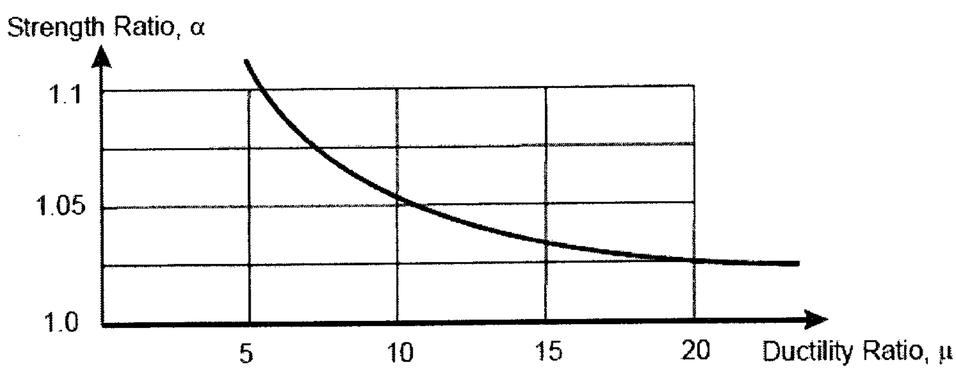


Figure 2.17. Effect of strength on elastic perfectly plastic structures (Powell 2005).

Chapter 3
Modeling of Multistorey Steel Structure and Different Retrofit Techniques

3.1 INTRODUCTION

Most steel frame existing buildings were designed without taking into account extreme loads like blast or gas explosions that can lead to the loss of key elements of the structure, especially on the ground floor, usually at the location of the garage, and consequently leading to the progressive collapse of the entire building.

These steel frame buildings need to be retrofitted to prevent the building from collapsing or to mitigate the response of the steel frame by decreasing vertical deflection under the lost column.

This chapter discusses the models studied, the assumptions made and the different retrofit strategy models that will be studied. Also included are the performance indicators that are used to investigate the behavior of the steel building frame.

3.2 PROPERTIES OF THE STUDIED MODELS

In this study, a three-dimensional model of an 18-story high-rise steel moment resisting frame building, with six bays in the longitudinal direction and three in the transverse direction was constructed using the Extreme Loading for Structures (ELS®) program. The building has the same plan throughout the entire height. The sizes of the columns were kept constant for every three stories along the height; whereas two sizes for the beams where designed and kept constant for the whole height, namely, external beams and internal beams. An initial span for the bays (i.e. in the two directions) was taken as 6.0 m, which is designated as the reference model. This span was later varied to be 5.0m, 7.5m, and 9.0m in order to evaluate the effect of bay size. The building was designed according to CSA-S16-01 (Design of steel structure) for gravity loading condition. Figures 3.1 and 3.2 show the elevation and plan of the studied buildings, respectively, with their respective column and beam sizes.

3.3 DESIGN PARAMETERS

3.3.1 Material properties

The material properties of the steel used in the studied model are as follows:

1) In the model, a bilinear stress-strain relationship of the steel members was taken, with Fy = 350 MPa, and strain hardening of 1% as shown in Figure 3.3.
2) Modulus of elasticity and Poisson's ration for steel were taken as 200 GPa and 0.2, respectively.
3) In the model, the inherent damping due to friction or yielding of steel elements was taken into account as stated in the technical manual of ELS®, whereas the external damping was neglected.

3.3.2 Design loads

The building was designed according to Canadian Code CISC 95 (CSA-S16-01). The frame columns and beams were designed to carry a slab thickness of 200mm. The floors were subjected to a live load of 2.4 kPa, representing the load of an office building, and a superimposed dead load of 2 kPa was taken to account for the equivalent load from interior partition, mechanical and plumbing loads.

3.4 STUDIED SCENARIOS OF PROGRESSIVE COLLAPSE AND LOAD COMBINATION

In this study, six column removal scenarios at ground level are studied, as shown in Table 3.1. One column is removed at a time, and a nonlinear dynamic analysis is performed using the ELS® software program. In each analysis, there are two stages: a static stage and a dynamic stage. The static stage is for importing the load combination on the model before removing any column. The dynamic stage is for tracing the response of the model after the column is removed as determined at that stage, as shown in Figure 3.4.

In the conducted nonlinear dynamic analyses, two load combinations to represent the gravity load are used in the case of studying the model with a bay span of 6.0 meters (reference model). The first load combination is (1.0 D.L+0.25 L.L), which follows the GSA guidelines, while the second is (1.2 D.L + 0.5 L.L), according to the DoD guidelines. These two load combinations were applied in each column removal scenario. While, in the case of evaluating the effect of bay size with bay spans of 5.0 m, 7.5m and 9.0 m, the GSA loading only was applied.

3.5 ASSUMPTIONS IN MODELING

In the analytical models, the following assumptions were made:

(1) Loads from concrete slabs are applied on the beams according to area method without representing the slabs in the model.

(2) There is no composite action between the steel beams and the concrete slab.

(3) Connection between the beam and the column are maintains continuity.

(4) Support conditions at the foundation are considered to be fixed.

(5) Increase of yield strength arising from the high rate of strain due to sudden removal of the column is neglected.

3.6 DIFFERENT RETROFIT STRATEGIES USED IN THIS STUDY

Three different retrofit strategies are applied to the existing model. They are: increasing strength only, increasing stiffness only and increasing both strength and stiffness of the beams. Figure 3.3 shows the three retrofit strategies studied. Retrofitting the beams was done for all beams in each floor along the height of the building were the result of the existing to safeguard the building against all cases of column loss scenarios.

In the current analyses, the effect of these three retrofit strategies has been investigated up to a level of 4 times that of the existing beam. In this study, an upgrading factor, α, that represents the increase in strength, α_s, or stiffness, α_k , or both, $\alpha_{s,k}$, of the retrofitted beam is introduced. The evaluation of the retrofitted beams was evaluated at upgrading levels of 1.1, 1.25, 1.5, 2 and 4, which correspond respectively to increases in strength or stiffness, or both, of 10, 25, 50, 100 and 300% from the existing model.

In the next sections the way to perform each retrofit strategy on the analytical model and the corresponding physical occurrences will be described.

3.6.1 Rehabilitation of the frame by increasing the strength of the beam only

In this technique, the increase of strength is done by changing the yield strength (F_y), which leads to a proportional increase in the strength or capacity of the section, where the capacity of the section is ($M_p=Z_x * F_y$). This strategy corresponds to using a Fiber-Reinforced Polymer composite (FRP) to strengthen an existing beam and is expected to contribute to the strength of the beam, without significant contribution to its stiffness.

3.6.2 Rehabilitation of the frame by increasing the stiffness of the beam only

In this technique, the increase of stiffness is done by increasing both Young's modulus (E) and shear modulus (G) in proportion, where increasing Young's modulus of the steel section will increase stiffness in proportion (K = EI/L). This strategy corresponds to strengthening a beam by using intermittent steel plates that will result in an increase in the stiffness without altering the strength of the beam

3.6.3 Rehabilitation of the frame by increasing both strength and stiffness of the beam only

In this retrofitting strategy an existing beam is strengthened by using additional continuous steel plates to increase both the strength and the stiffness of the beam. This is done by increasing the steel plates at the flanges so that both the strength and the stiffness will be increased in proportion.

This technique can be done easily by increasing the thickness of the flanges to increase the strength and the stiffness proportionally. Table 3.2 shows the increase in flange thickness that increases both strength and stiffness simultaneously with increases of 10%, 25%, 50%, 100% and 300% from the original case.

3.7 PERFORMANCE INDICATORS USED TO EVALUATE THE STUDIED MODELS

For each case, the effect of three retrofitting strategies on three indicator parameters is evaluated. These parameters are the chord rotation, the tie forces, and the displacement ductility demand of the beams surrounding the removed column. As in this study is concerned with the local zone around the removed column which is the most affected zone.

3.7.1 Chord rotation of the beams

Chord rotation can be calculated by dividing the vertical deflection under the removed column over the span of the bay as shown in Figure 3.5.a. The DoD guidelines state that the maximum chord rotation for beams is 12 degrees for Very Low Level Of Protection (VLLOP) and Low Level Of Protection (LLOP), and 6 degrees for Medium Level Of Protection (MLOP) and High Level Of Protection (HLOP). In the ELS program it is easy to visualize the failure of the model after performing nonlinear dynamic analysis. This failure can occur in parts of or throughout the entire building.

3.7.2 Tie force in the beams

Beams resist loads in two ways: moment resisting at the ends of the beams and tie force in the beams. Tie force in a beam is the tension force exerted in the beam in cases of high deflection, as shown in figure 3.5.b. The DoD guidelines mention the minimum tension force that both external and internal beams must resist in order for a tie to be created if a column is lost.

3.7.3 Displacement ductility demand in the beams

The displacement ductility demand of the beams is equal to the vertical displacement under the removed column that results from nonlinear dynamic analysis divided by vertical deflection at yield. Vertical deflection at yield can be calculated by performing pushdown analysis (nonlinear static analysis). The linear portion in the curve between vertical deflection under removed column and the increment of vertical applied loads represents the deflection at yield, as shown in Figure 3.6.

3.8 STEPS OF THE ANALYTICAL WORK

For the reference model of a span of 6m for each bay, two load combinations were applied to the beams, the GSA and DoD load combinations. The internal beams were loaded by doubling the loads of the external beams. Then, nonlinear dynamic analyses were conducted to calculate the three parameters (chord rotation, tie force and displacement ductility demand) of the beams for the original building before retrofitting. After that, there was the application of the three retrofit strategies, which increase strength and/or stiffness of the beams on the original (existing) building; and the three parameters were calculated again after retrofitting by an upgrade factor of 1.1, 1.25, 1.5, 2 and 4. Finally, the reduction factor for each parameter and retrofit method was conducted and the proposed equation was derived. The proposed equation can be used to predict these parameter indicators after upgrading. A flow chart for the reference model of the nonlinear dynamic analysis to evaluate the effect of three retrofit strategies on three performance indicators (θ, TF, and μ_Δ) is shown in figure 3.7.

Also, effect of variations in bay area were investigated by conducting three models with spans of 5m, 7.5m and 9m, taking the span of 6m as the reference model and applying only the GSA load combination. Then, nonlinear dynamic analyses were conducted for the existing building and after retrofitting. Finally, factors for variations of bay area for each parameter were conducted and proposed formulae were derived to take into account the various bay areas. A flow chart of nonlinear dynamic analysis for the various bay areas is shown in figure 3.8.

3.9 PROGRAM DISCRIPTION

The program used is the Extreme Loading Structure (ELS ®) software program, which is based on theApplied Element Method (AEM). This section will provide an overview of the Applied Element Method (AEM) and the program.

3.9.1 Applied Element Method Overview

The AEM models the structure as an assembly of small elements, which are made by dividing the structure virtually. Any two elements shown are assumed to be connected by one normal and two shears springs located at contact points that are distributed around the elements' edges, as shown in Figure 3.9. Each group of springs completely represents stresses and deformations of a certain volume.

The Applied Element Method (AEM) is capable of predicting the discrete behaviour of the structure to a high degree of accuracy. It has been proved to be a method that can track structural collapse behavior, passing through all the stages of the application of loads, elastic stage, crack initiation, element separation, and collision with the ground and with adjacent structures. AEM has the relative advantage over the Finite Element Method (FEM) inasmuch as the elements are capable of separation and can thus simulate the real collapse of the structure, whereas the FEM does not have this capability because of the continuity between elements; therefore, no separation that could lead to a singularity in its geometric matrix can occur.

In AEM, each element has 3-D physical coordinates and shape. Hence, elements are a group of 3D elements which can be separated and/or collided. In the connectivity of FEM, an element cannot be separated and each element must be connected to the other element through nodes, but in AEM there is no need for nodes because the elements are connected by springs. Both FEM and AEM are accurate in elastic and nonlinear analyses of small displacements. The difference is that AEM gives good results in large displacements and collapse cases, but FEM it is poor in large displacements and cannot create a collapse case because of the impossibility of separating the elements. It cannot simulate a real collapse because the solid elements are compatible in deformation along with their intersection nodes and cannot be separated. Figure 3.10 shows the domains of analysis of both AEM and FEM.

3.9.2 Extreme Loading for Structures (ELS®) program software

Extreme Loading for Structures (ELS®) is a program that works with the Applied Element Method (AEM). AEM is capable of predicting to a high degree of accuracy the discrete behavior of the structure. The ELS® nonlinear solver analyzes structural behavior during elastic and inelastic modes, including the automatic detection and generation of plastic hinges, buckling, cracks, and collapse. The resulting debris and its impact with the structural elements are automatically analyzed.

The structural members (beams and columns) are discretized into small rigid elements that are connected by contact points. Each contact point has three springs, one normal and two shears that represent the nonlinear material properties of steel. The springs are generated between elements that pass through the steel section only, while elements that do not pass through steel section are considered as a vacuum without any material properties, as shown in Figure 3.11.

The stiffness of each spring depends on the area it serves, as shown in Figure 3.9.b. Each rigid element contains 6 degrees of freedom (3 rotations and 3 translations). The stiffness matrix components corresponding to each degree of freedom are determined by assuming a unit displacement in the studied direction and by determining forces at the centroid of each element. The stiffness matrix of each rigid element is calculated by summing up all the stiffnesses produced by all the springs of that element.

The stiffness matrix of the rigid element combined with its associated contact points is constructed by summing up all the stiffnesses produced by its components (rigid elements and springs). Finally, the assembly of all the rigid elements' stiffnesses in the structure results in the global stiffness matrix of the entire structure.

Figure 3.12 shows the different stages that the elements pass through during loading. First, elements are in the initial stage; then the springs start to be under tension. The elements will be separated when they reach the separation strain which is defined in the program input and recommended to be 0.1. After separation the initial springs are demolished; and in case these elements come into contact with each other, they again produce another type of springs called contact springs. These contact springs are generated when debris comes into contact with other structural elements in the building.

3.10 VERIFICATION PROBLEMS FOR THE SOFTWARE PROGRAM

For verifying this program, two models where analyzed to check the results. The first model was a 2D model that was created by macro-modeling. The other model was a 3D model that was modeled by the finite element program SAP2000.

The first verification model has a cross-section as shown in Figure 3.13. The applied load is dead and live load of 3.64 KN/m^2 and 4.789 KN/m^2. The area method was used to determine the load transfer to the beams along line 6. Figure 3.14 shows the model constructed in the ELS program. The macro model and the ELS program model give excellent matching results for the deflection due to the removal of a column along axis (C): 239 mm from the macro model and 233 mm from the ELS program model.

The second verification model was a nine-story steel moment frame structure with six bays in the longitudinal direction and three in the transverse direction. A scenario of removing a column in the edge-long direction was conducted as shown in Figure 3.15. This building was modeled using ELS software, as can be seen in Figure 3.16. The vertical deflection by conducting nonlinear dynamic analysis was 281 mm, while the vertical deflection obtained by modeling the same building using ELS was 278mm, which is very close.

Therefore, beside the proven capabilities of the software due to its commercialization, the conducted verification gave more confidence in the input parameters of modeling steel frame structures and consequently their computed response

Table 3.1. Removed columns and there location in plan

Case No.	Removed column	Column location
1	Internal Column (IC)	D-3
2	Corner Column (CC)	A-1
3	Edge Long Column (ELC)	D-4
4	Edge Short Column (ESC)	A-3
5	First Internal Column (FIC)	B-3
6	First Edge long column (FELC)	B-4

Table 3.2. Increasing both strength and stiffness in proportion by increasing flange thickness with the corresponding plastic moment and moment of inertia

Factor of increase both (Upgrading factor)	Beam type	Flange Thickness increase (mm)	Plastic Moment (kN.m)	Moment of Inertia ($\times 10^6$) (mm^4)
1 (Original Case)	External	ZERO	130.8	34
	Internal	ZERO	225.6	60.5
1.1 (increase 10 %)	External	1.2	144	37.7
	Internal	1.6	247.8	66.5
1.25 (increase 25 %)	External	2.9	162.42	42.77
	Internal	4	280.8	75.97
1.5 (increase 50%)	External	5.8	193.98	51.47
	Internal	7.8	333.1	90.8
2 (increase 100%)	External	11.4	254	68.37
	Internal	15	432.3	119.1
4 (increase 300%)	External	33.5	496	136.6
	Internal	45	846.5	241.6

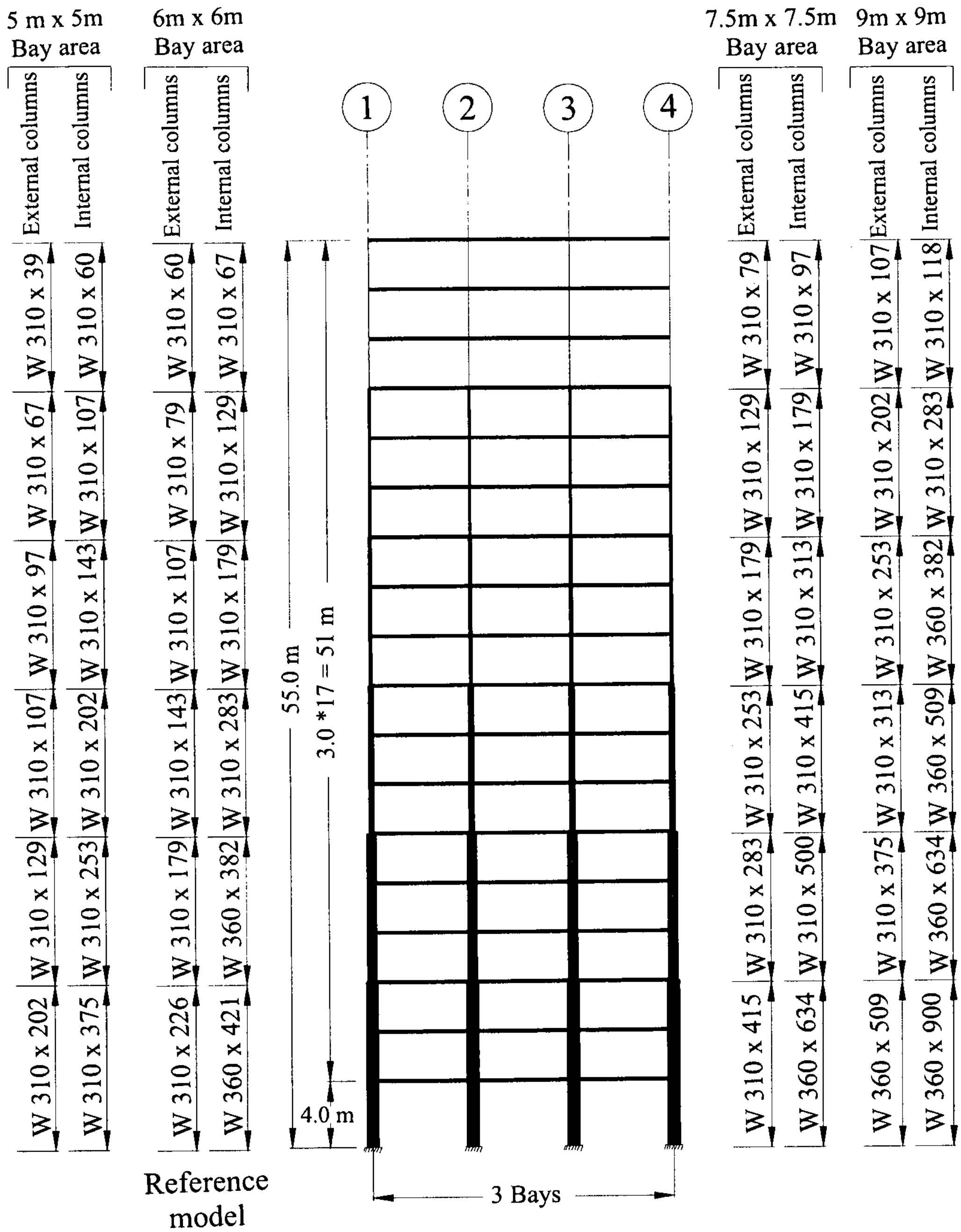


Figure 3.1. Elevation for the studied models and their column size for gravity frames.

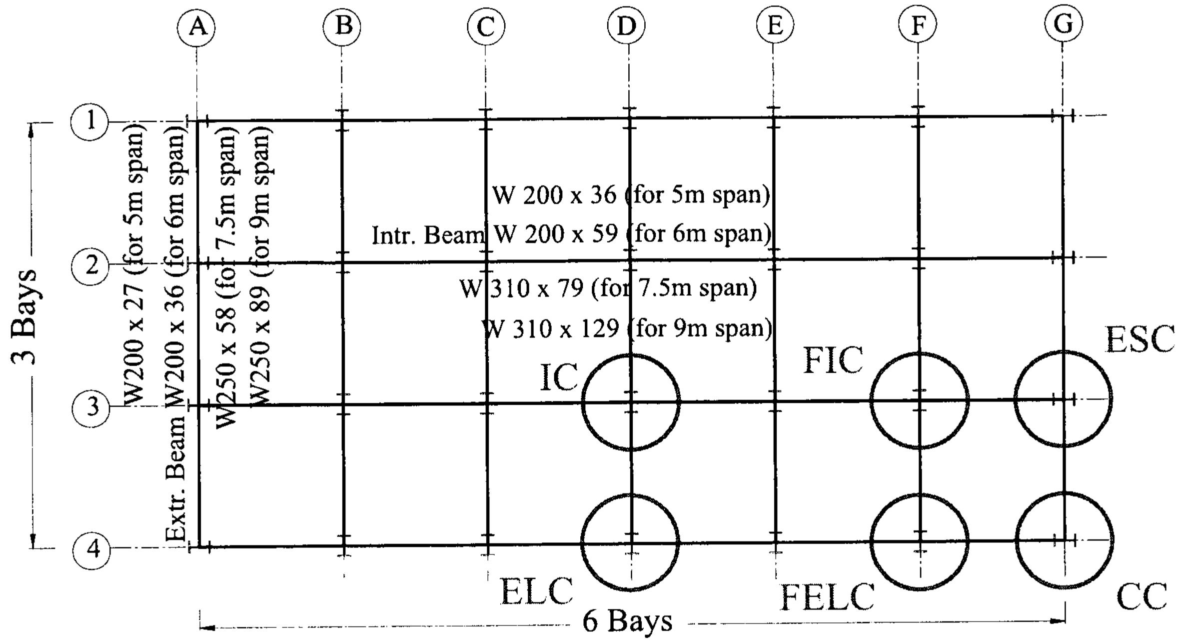


Figure 3.2. Plan of the studied models, beam size and location of the removed column for the gravity frames.

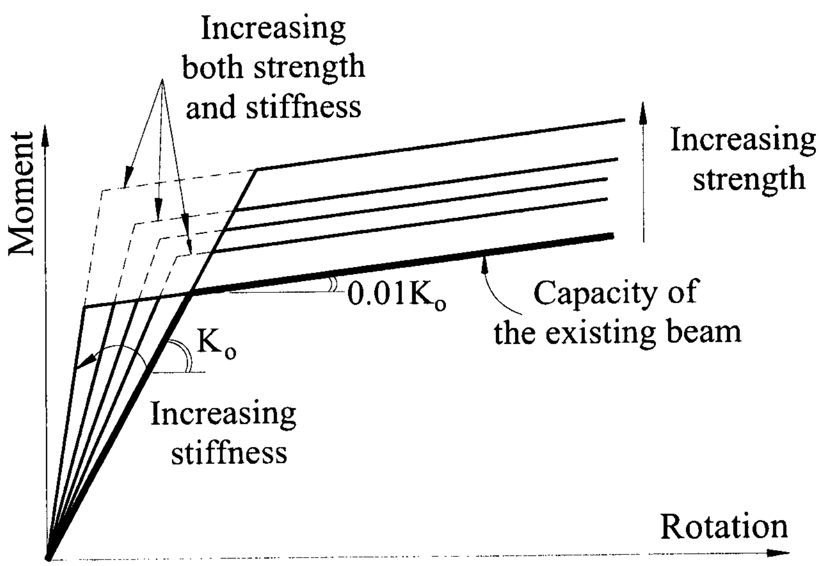


Figure 3.3. Methods of upgrading increase strength and/or stiffness of the beams.

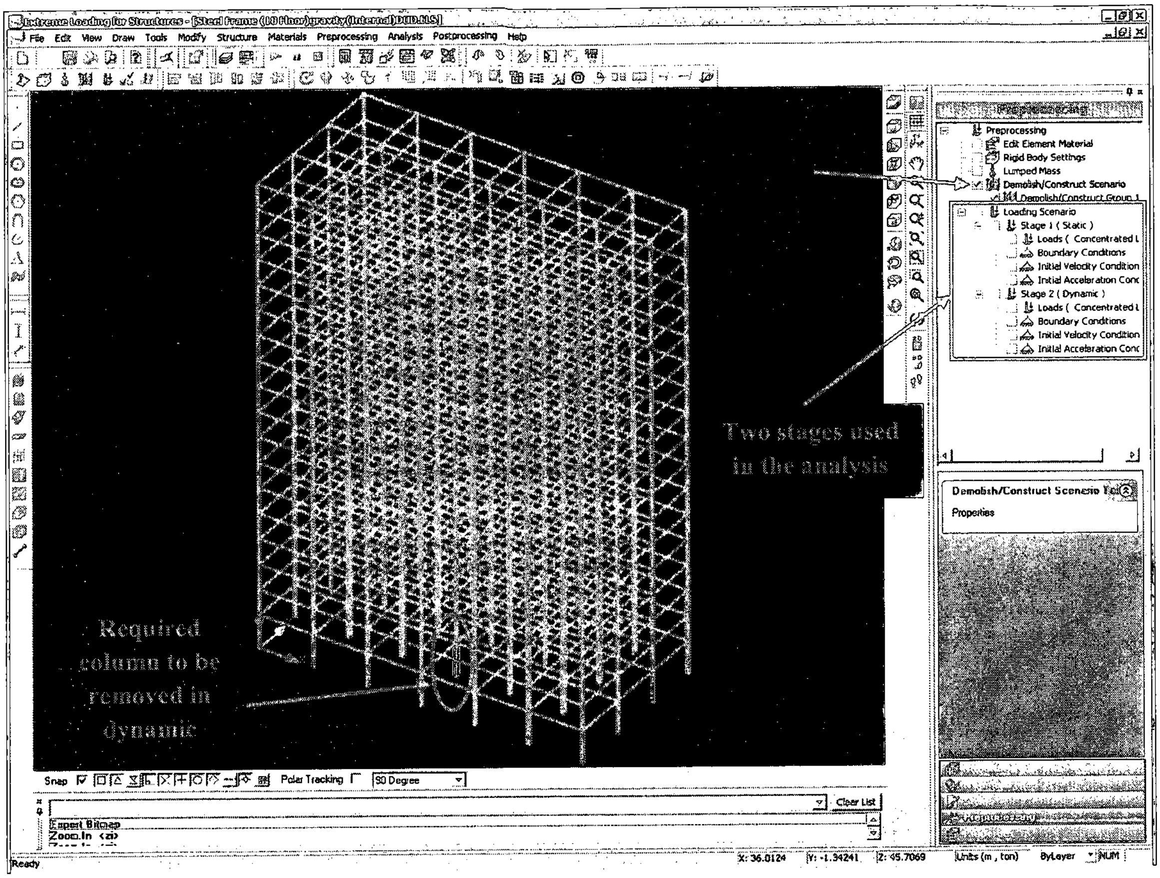
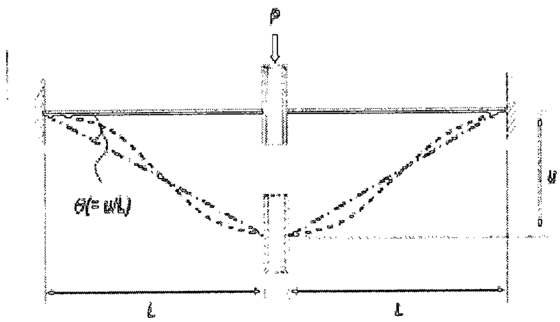
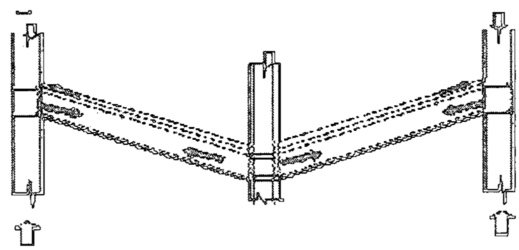


Figure 3.4. Studied model input in ELS® software.



a) Chord rotation in beams



b) Tie force in beams

Figure 3.5. Chord rotation and tie force in beams.

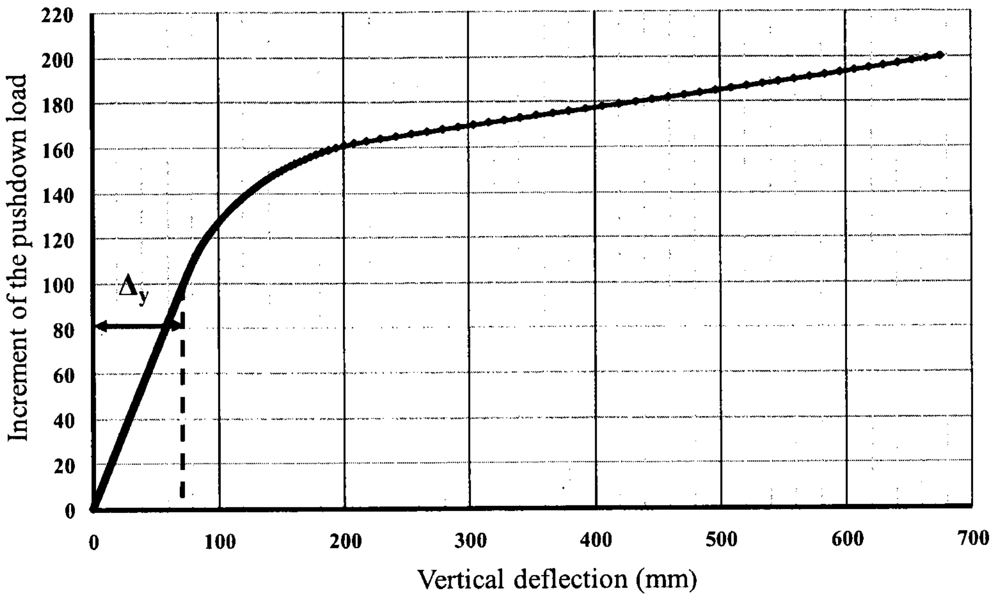


Fig. 3.6. Result curve from pushdown analysis to determine deflection at yield.

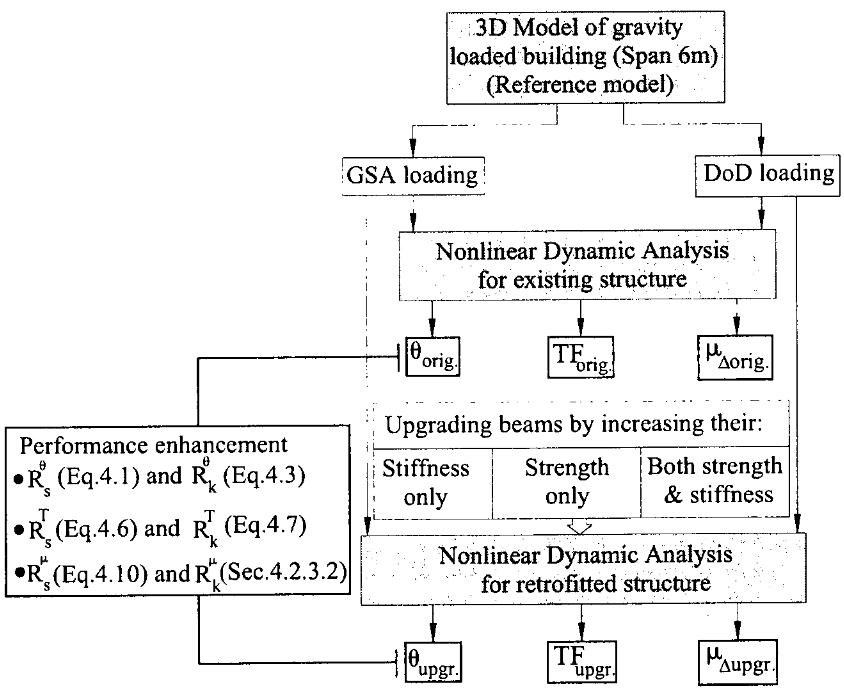


Figure 3.7. A flow chart for the reference model of the nonlinear dynamic analysis to evaluate the effect of retrofit strategies on three performance indicators (θ, TF, and μ_Δ).

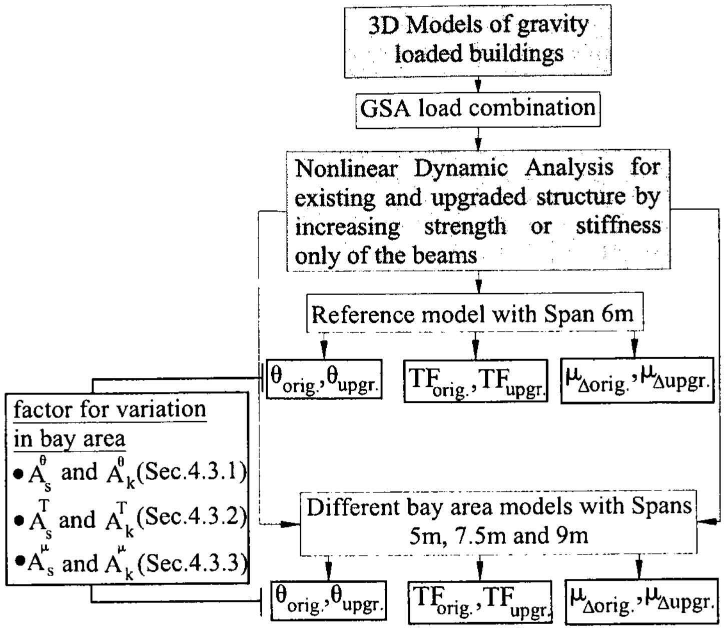
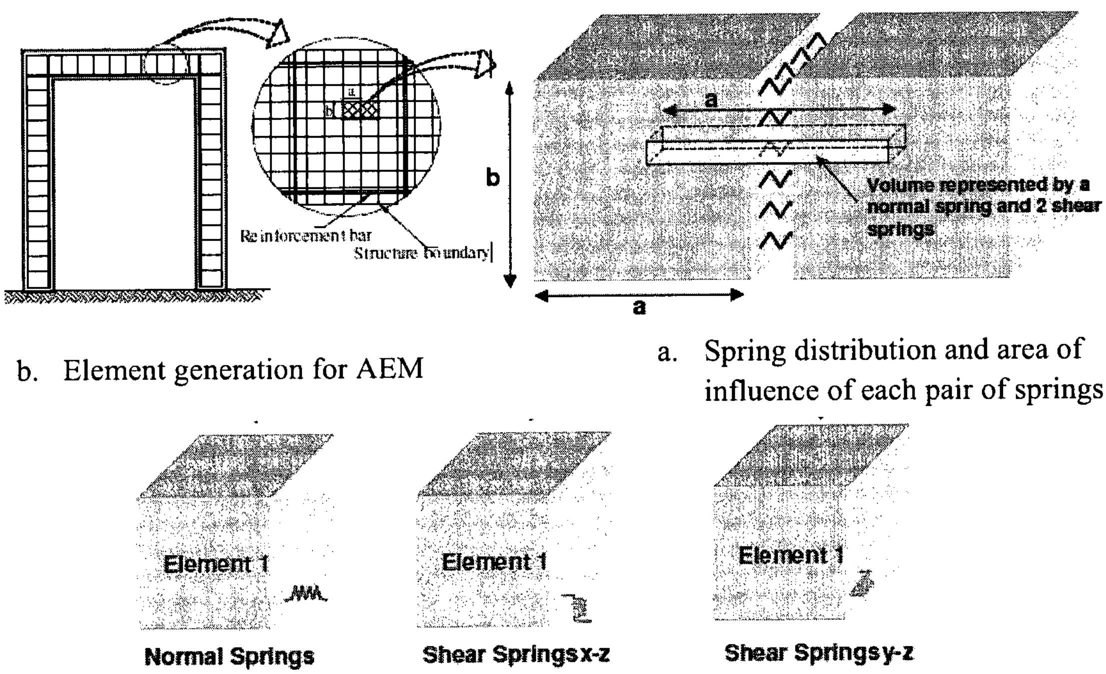


Figure 3.8. A flow chart of nonlinear dynamic analysis for various bay areas to evaluate the effect of three retrofit strategies on three performance indicators (θ, TF, and μ_{Δ}).



b. Element generation for AEM

a. Spring distribution and area of influence of each pair of springs

c. Three springs each contact point

Figure 3.9. Elements and springs generated for AEM (ELS® technical manual 2006).

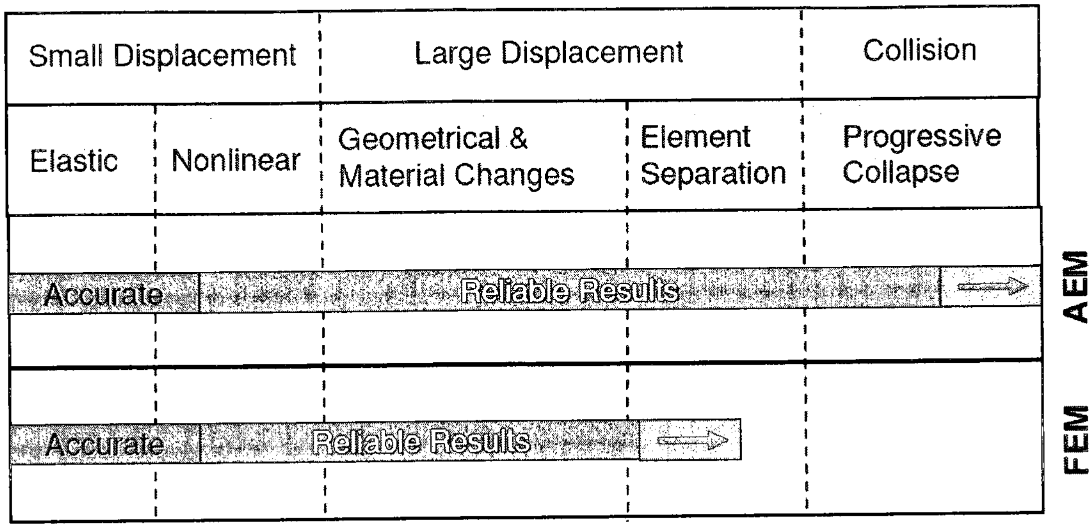


Figure 3.10. Domain of analysis of both AEM and FEM (ELS® technical manual 2006).

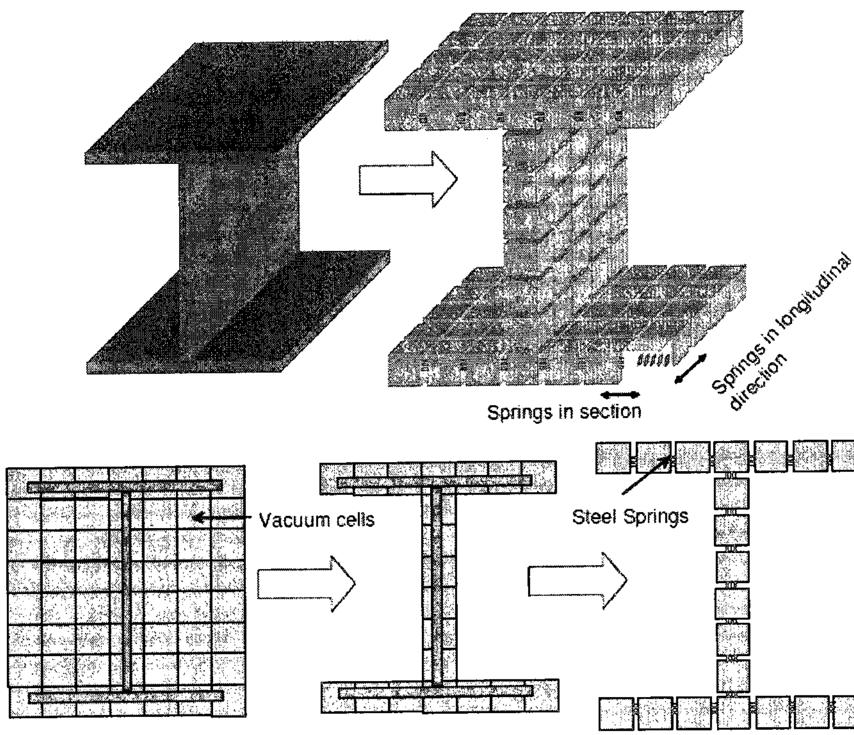
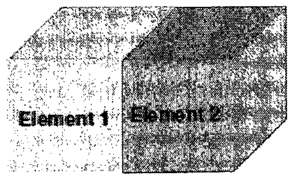
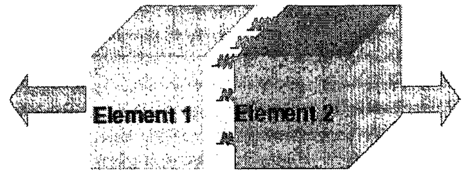


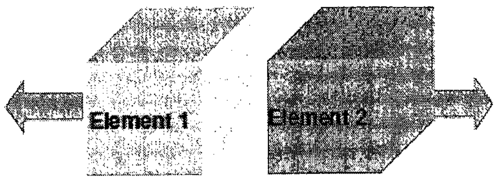
Figure 3.11. Modeling of steel members (ELS® technical manual 2006).



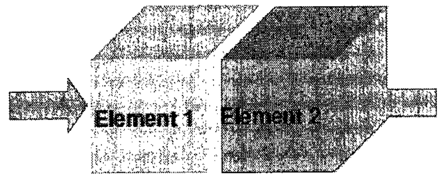
5) Initial position



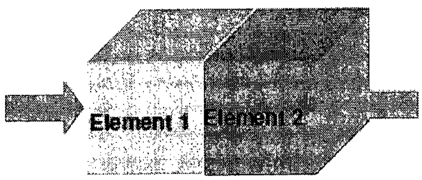
4) Matrix springs under tension



3) Separation strain reached

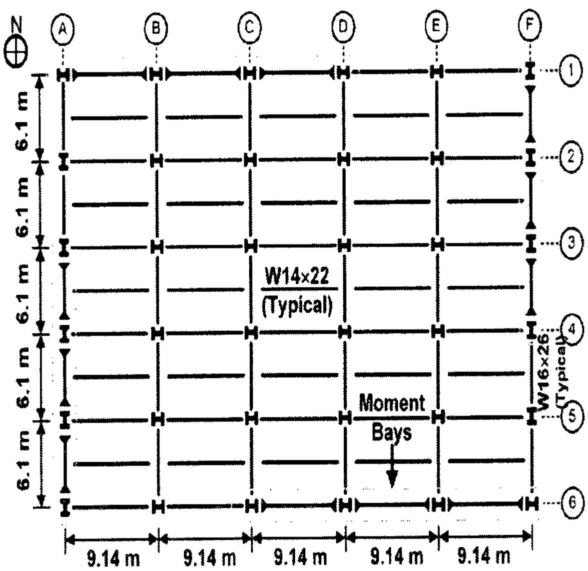


2) Element forced towards each other (according to loading condition)

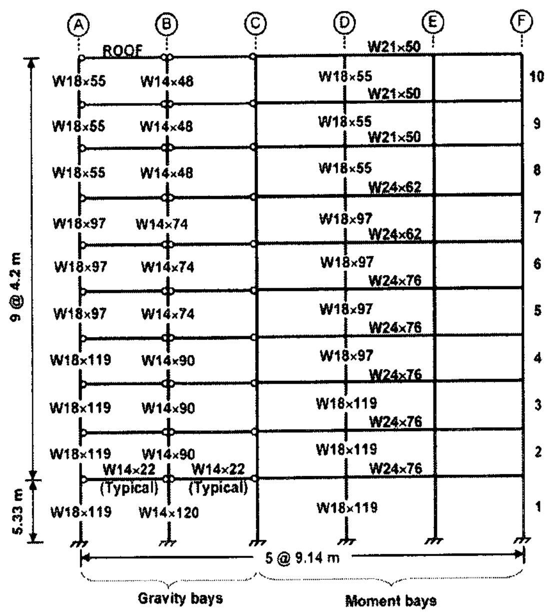


1) Contact springs activated

Figure 3.12. Elements separation and re-contact again (ELS® technical manual 2006).



Plan



Elevation

Figure 3.13. 2D Model 1 made by Khandelwal et al. (2008) used for verification.

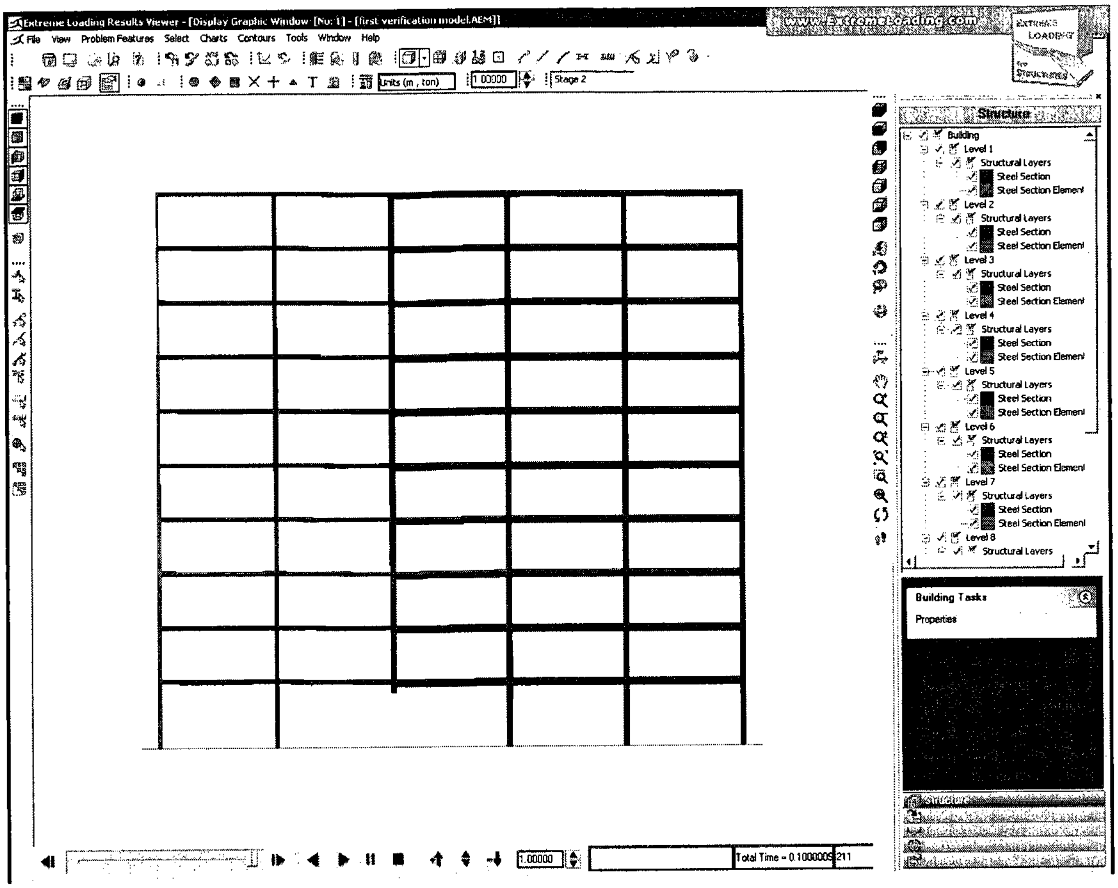


Figure 3.14. Verification Model 1 in ELS.

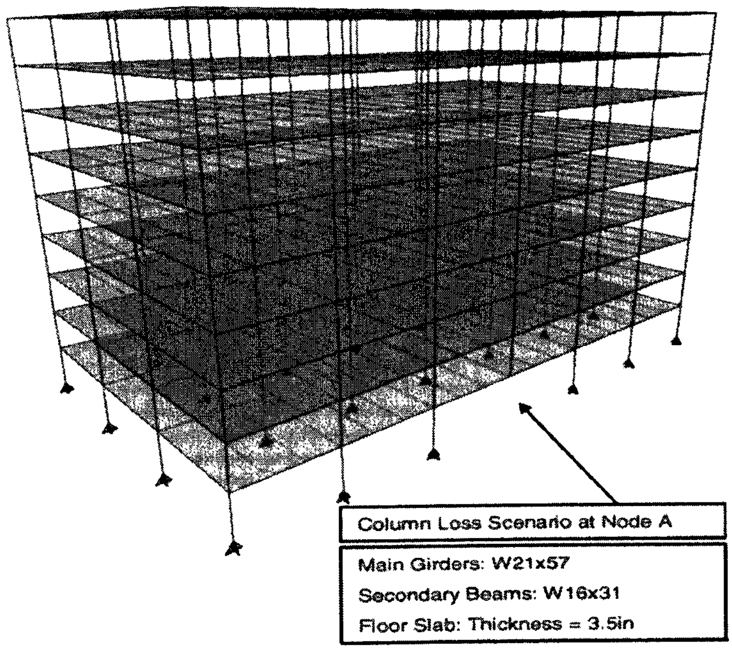


Figure 3.15. Three-dimensional Model 2 in SAP 2000 by Marjanishvili et al (2006) used for verification.

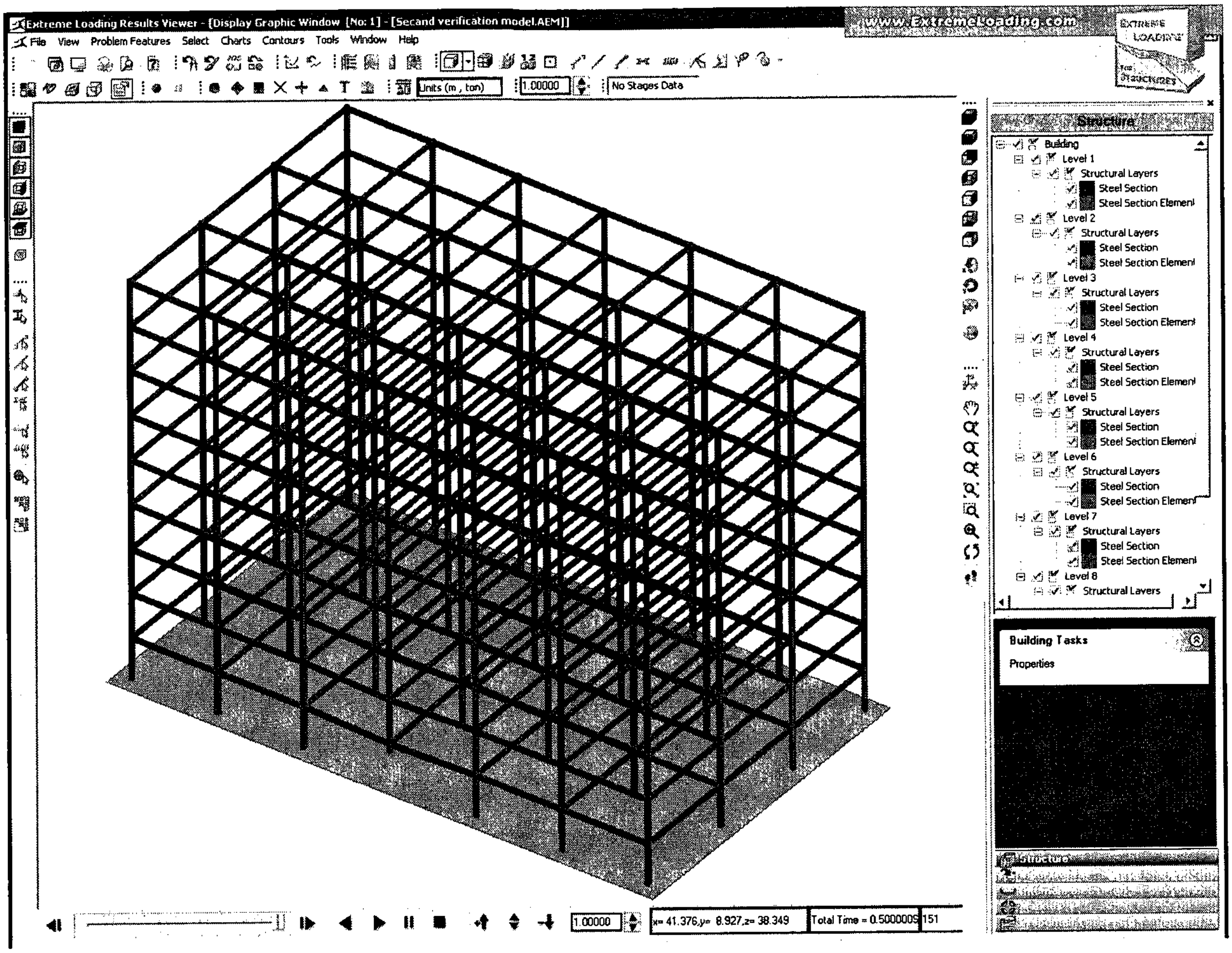


Figure 3.16. Verification Model 2 in ELS.

Chapter 4

Analytical Results

4.1. INTRODUCTION

In this chapter, the analytical results of all models will be scrutinized. As well, the proposed equations will be stated. In section 4.2 the analytical results for the reference model will be discussed. Then the results for variations in bay areas will be disscused.

4.2. ANALYTICAL RESULTS OF THE REFERENCE MODEL

In this section results of the reference model of a span of 6m in both directions are investigated by applying both the GSA and DoD loading combinations. Analysis of these models is investigated according to three parameters: chord rotation, tie force and displacement ductility demand in the beams. This parameters are mesured for the beams under the removed column where the local damage occurs.

4.2.1. Effect of retrofit strategies on chord rotation (θ)

As mentioned in section (3.7.1), chord rotation is equal to the deflection under the removed column divided by the span; therefore, the chord rotation can be calculated from the deflection under the removed column. Figure 4.1 shows the output vertical deflection from the ELS software. The maximum rotation according to the DoD guidelines is 12 degrees for VLLOP and LLOP, which correspond to a vertical deflection of 127.5 cm. For MLOP and HLOP, the maximum rotation is 6 degrees, which correspond to a vertical deflection of 63 cm for span of 6m.

4.2.1.1. Before Upgrading

For an existing building, under GSA factored loading (D.L+ 0.25 L.L) none of the six scenarios of column removal failed. The worst case was removal of the Edge Short

Column (ESC) which gives the highest deflection of 1070 mm, while the least was the removal of the First Edge Long Column (FELC) with a deflection of 640 mm, as shown in Table 4.1.

Also it was found that the removal of First Internal Column (FIC) and (FELC) give smaller deflection than those of the corresponding deflection in removal of Internal Column (IC) and Edge Long Column (ELC), respectively. This could be attributed to the orientation of the four columns adjacent to the removed one; i.e. in case of removal of (IC) it had two columns in their strong axis and two columns in their weak axis, while removal of (FIC) had three columns oriented in strong axis and one on its weak axis as shown in Fig. 4.2. Similarly, it is found that removal of (FELC) give have smaller deflection than that in case of removal of (ELC). This can be attributed to the orientation of (ELC) had one column oriented on its strong axis and two columns on their weak axis, while removal of (FELC) had two columns oriented on their strong axis and one on its weak axis.

Also, the deflection of removal of (ESC) is found to be the largest deflection and rotation and that could be due to that three beams projected from the removed column are connected to the adjacent three columns through their weak axes and connected to small number of bays. On other hand, scenario of removal of (ELC) shows smaller deflection than that scenario of (ESC) because it has one column oriented on its strong axis and has higher number of bays in its direction, as shown in Figure 4.2.

4.2.1.2. After Upgrading

In this section, the effect of upgrading the beams by increasing their strength and/or stiffness is investigated. Two reduction factors R_s^θ and R_k^θ are introduced and defined as the reduction factor of chord rotation after increasing the strength and stiffness factors, respectively, and are equal to the percentage of the ratio of upgraded chord rotation $\theta_{upgr.}$ to the chord rotation $\theta_{orig.}$ of the existing structure, as seen in Figure 4.3.

Figures 4.4 to 4.9 show the reduction factors in chord rotation (θ) for all scenarios of removing columns after increasing strength and/or stiffness for removing columns ESC, CC, IC, FIC, ELC, and FELC, respectively, after applying both the load combinations of GSA and DoD. Also, two proposed equations for the reduction factors R_s^{θ} and R_k^{θ} are plotted in red by curve fitting in these Figures. From Figures 4.4(a) through 4.9(a), it can be seen in all these curves of applying GSA loading that increasing the strength to a strength factor of 2 (α_s=2) has a great effect on the reduction in chord rotation R_s^{θ}; after that, slight effects on the level of reduction in chord rotation R_s^{θ} were seen (reduction less than 10% to α_s=4). While, this is not the case for the reduction factor R_k^{θ} due to increase in stiffness factor αk which decreases approximately linearly. It can also be seen that increasing the strength of the beams has more effect on reducing the chord rotation compared to increasing the stiffness of the beams, especially for an upgrading factor of less than 2 (α_s< 2). The latter observation is valid for all six scenarios of column removal. From the analysis, it was found that for upgrading the beams by an upgrading factor of 2 (α=2), which corresponds to an increase in either strength or stiffness by 100% from the existing model, the reduction factors of chord rotation after the increase of strength only and stiffness only for all six scenarios were around 35% and 65%, respectively, which means that the retrofit strategy of increasing strength only is more effective than that of increasing stiffness only.

For case of increasing both stiffness and strength, the analysis showed that the reduction factor in chord rotation $R^{\theta}_{s,k}$ at a different upgrading factor, $\alpha_{s,k}$, was simply the product of both reduction factors R^{θ}_{s} and R^{θ}_{k}.

The original model subjected to the DoD load combination collapsed, as shown in Figures 4.4(b) to 4.9(b); increasing stiffness did not prevent the failure because the beams did not have sufficient capacity to resist the loads. Where building considered to be collapsed in case of total or partial local collapse that exceed the limitation stated in guidelines. Therefore, the effective retrofit strategy in this case is to increase strength only. Thus, the reduction factor in chord rotation R^{θ}_{k} in the case of increasing the stiffness of the beams is associated with an increase in strength by 1.25 of the original structure subjected to DoD loads, as shown in Figures 4.4(b) to 4.9(b). In the same manner, R^{θ}_{s} is calculated with respect to the model after increasing the strength of beams by 1.25 of the original model. Also, Table 4.1 shows the deflection and chord rotation of the beams after upgrading by a strength factor of 1.25 for all the scenarios of column removal.

In this research, two equations for the reduction in chord rotation due to increasing stiffness R^{θ}_{k} and strength R^{θ}_{s} for different levels of upgrading factor α are proposed. Equation 4.1 gives the values of R^{θ}_{s} as a function of α_s, and Equation 3 gives the values of R^{θ}_{k} as a function of α_k. The coefficients "a" and "b" in both equations are given for different cases of column removal in Tables 4.2 and 4.3 for loading using GSA and DoD, respectively. The proposed equations for calculating the reduction factors R^{θ}_{s} and R^{θ}_{k} conducted by curve fitting are as follows:

$$R_s^{\theta} = \frac{100.\alpha_s}{a.\alpha_s + b} \qquad (4.1)$$

where $$\theta_{upgr.,s} = R_s^{\theta} \, . \, \theta_{orig.} \qquad (4.2)$$

$$R_k^{\theta} = \frac{1000}{a.\alpha_k + b} \qquad (4.3)$$

where $$\theta_{upgr.,k} = R_k^{\theta} \, . \, \theta_{orig.} \qquad (4.4)$$

Using the above equations, the chord rotation after upgrading can be estimated. It was also concluded that for retrofitting the beams by increasing both stiffness and strength the chord rotation after upgrading $\theta_{up.,s,k}$ can be predicted by the following equation:

$$\theta_{upgr.s,k} = R_k^{\theta} \, . \, R_s^{\theta} \, . \, \theta_{orig.} \qquad (4.5)$$

where R_k^{θ} and R_s^{θ} can be obtained from Equations 4.1 and 4.3 and their corresponding coefficients in Tables 4.2 and 4.3.

4.2.2. Effect of retrofit strategies on tie force

Tie force in beams, which is an axial tension force exerted in a beam under high deflection due to the catenary action of the beam evaluated by the ELS software, as shown in Figure 4.10, is compared to that calculated by the DoD guidelines for internal and external tie force, as follows:

1- Internal tie force = 0.5(1.2 x D.L+1.6L.L) x S x L= 0.5[(12 x 0.9)+(1.6 x 2.4)] x 6 x 6= 264 kN.

2- External tie force= 0.25(1.2 x D.L+1.6L.L) x S x L= 0.5[(12 x 0.9)+(1.6 x 2.4)] x 6 x 6= 137 kN.

4.2.2.1. Before Upgrading

In GSA loading, it was found that the highest tie force is from the scenario of the removal of an internal column with 1150 kN, which is more than four times what is stated in the DoD guidelines. Also, the first internal column removal is smaller than that of the internal removal where tension force of 625 kN in the beams is exerted. All edge removal scenarios exert values around 400 kN, which is almost three times the amount stated in the DoD guidelines. However, the scenario of removing the edge long column exerts a higher tie force compared to other edge scenarios. The values of the tie forces in the beams of the original model are shown in Table 4.4.

With DoD loading, the original model failed, as mentioned in section 6.1; and the first model did not fail only after an increase in the strength by a factor of 1.25. The removal of the internal column also creates the highest tension force in the beams: 1340 kN, which is more than five times the design tie force in the DoD guidelines. The second highest tension force is in first internal column removal, with 720 kN. Finally, in the the edge removed scenarios, it is about 460 kN, which is more than three times the designed value, as shown in Table 4.4.

That mean internal columns scenarios exert high tie force compared to the edge column scenarios; and that is due to the higher loads, which lead to a relaying of the tie force in a beam after exerting full capacity of the moment to bridge the loads to prevent it from collapse. It was found that the tie force exerted in all scenarios was more than three times that of the designed value in the DoD guidelines. Results of similations in the current research agree with Liu et al. (2005) found, the tie force in the beams of a 7-story model was very high compared to BS 5950 [BSI, 2000].

4.2.2.2. After Upgrading

Similar to the reduction factors defined for the chord rotation, two reduction factors, R_s^T and R_k^T , are introduced and defined as the reduction factors of tie forces after increasing strength only and increasing stiffness only, respectively, and are equal to the percentage of the ratio of the tie force of upgraded beams $TF_{upgr.}$ to the tie force of the original beams $TF_{orig.}$. Whereas, for the DoD, these ratios are defined as the percentage of the ratio of the Tie Force of upgraded beams TF_{upgr} to the tie force of the beams after increasing strength by 1.25 times (α_s=1.25). This was due to the collapse of the original model; thus, it is of no value for the tie forces.

Figures 4.12 to 4.17 show the reduction factors in ties force (TF) for all scenarios of removing columns after increasing strength and/or stiffness for removing columns IC, FIC, ELC, FELC, ESC, and CC, respectively, after applying both the load combinations of GSA and DoD with two proposed equations for the reduction factors R_s^T and R_k^T .

From Figures 4.12(a) to 4.17 (a), by applying GSA loading, it was found that upgrading the beams by increasing their strength only up to a strength factor α_s=2 leads to a significant reduction in the tie forces, whereas an additional increase in the strength factor beyond α_s=2 does not enhance the reduction in the tie forces. On the other hand, increasing the stiffness of the beams up to a stiffness factor of α_k=2 has a linear trend on the reduction factor for tie force; and similar to the case of increasing strength, increasing stiffness beyond α_k=2 has an insignificant effect on enhancing the reduction in the tie forces. Figures 4.12(b) to 4.17(b) show a similar trend in the reduction factors in ties forces of the beams when the building is loaded with DoD loading.

After conducting the nonlinear dynamic analysis on the building using the three retrofit strategies and the six scenarios of column removal when subjected to the two cases of loading (GSA and DoD), two equations for estimating the reduction factors in tie force due to increased stiffness R_k^T and strength R_s^T for different levels of upgrading factor α are proposed. Equation 4.6 gives the values of R_s^T as a function of α_s, and Equation 8 gives the values of R_k^T as a function of α_k. The coefficients "a", "b" and "c" in both equations are given for different cases of column removal in Tables 4.5 and 4.6 for loading using GSA and DoD, respectively. The proposed equations for calculating the reduction factors R_s^T and R_k^T conducted by curve fitting are as follows:

$$R_s^T = \frac{100.\alpha_s}{a.\alpha_s^2 + b.\alpha_s + c} \qquad (4.6)$$

$$R_k^T = \frac{1000}{a.\alpha_k^2 + b.\alpha_k + c} \qquad (4.7)$$

It was concluded that for the case of retrofitting the beams by increasing both stiffness and strength, the Tie Force in a beam after upgrading $TF_{up.,s,k}$ can be predicted by the following equation:

$$TF_{upgr.,s} = R_s^T \, . \, TF_{orig.} \qquad (4.8)$$

$$TF_{upgr.,s,k} = R_s^T \, . \, R_k^T \, . \, TF_{orig.} \qquad (4.9)$$

where R_k^T and R_s^T can be obtained from Equations 4.6 and 4.7 along with their corresponding coefficients in Tables 4.5 and 4.6 for the GSA and DoD load combination, respectively.

4.2.3 Effect of retrofit strategies on displacement ductility demand

As mentioned earlier in section (3.7.3), displacement ductility demand is defined as the ratio of the deflection under the removed column for each case to the yield deflection (Δ_y) of the adjacent beams. Yield deflection can be calculated by a pushdown analysis that can determine the linear portion in the force deflection curve. Pushdown analysis is constructed by nonlinear static analysis without performing dynamic analysis.

4.2.3.1 Before Upgrading

The GSA and DoD guidelines limit the maximum displacement ductility demand in the beams to a value of 20. In all scenarios of column removal under GSA and DoD loading for the studied building, the maximum displacement ductility demand reached was 10, which is half of that stated by GSA and DoD. The highest ductility demand occurs from the scenario of removing the Edge Short Column (ESC), while the least value arises from the First Edge Long Column (FELC). This trend is similar to that of chord rotation and deflection. Table 4.7 shows the displacement ductility demand μ_Δ, of the beams adjacent to the removed columns of the existing building under the GSA and DoD loadings.

4.2.3.2. After Upgrading

Similar to the reduction factors defined previously, two reduction factors R_s^μ and R_k^μ for the cases of increasing strength only and stiffness only, respectively, are introduced and defined as the percentage of the ratio of the ductility demand of upgraded beams, μ_{upgr}, to the ductility demand of the original beams, $\mu_{orig.}$. On the other hand, for DoD loading, these ratios are defined as the percentage of the ratio of the ductility demand of upgraded beams μ_{upgr} to the ductility demand of the beams after increasing strength by 1.25 times (α_s=1.25), that is, due to the collapse of the existing building if it is not retrofitted (i.e. at α_s=1.0).

Figures 4.18 to 4.23 show the reduction factors in displacement ductility demand (μ_Δ) for all scenarios involving the removal of columns after increasing strength and/or stiffness for removing columns ESC, CC, IC, FIC, ELC, and FELC, respectivly, and the proposed equations for R_s^μ for GSA and DoD loading conducted by curve fitting. It was observed that upon increasing the strength only of the beams, the displacement ductility demand decreases; and this is attributed to the decrease in maximum deflection along with increase in yield deflection, which leads to a decrease in the displacement ductility demand. On the other hand, increasing the stiffness only of the beams results in a reduction in both the maximum deflection and yield deflection of almost the same rate. This resulted in a fluctuation of the values of the displacement ductility demand within a range of ±15% of its original values. Thus, strengthening the beams by increasing their stiffness has no significant effect on their displacement ductility demand. This means that increasing both strength and stiffness will lead to a ductility behaviour similar to that of increasing the strength only.

Since R_k^μ does not change significantly, its values are taken to be constant and equal to 100%. Equation 4.10 is proposed to calculate the values of R_s^μ for different levels of increased strength α_s, where the coefficients "a" and "b" are shown in Table 4.8. The coefficients had almost the same values for the different scenarios of column removal under either loading criterion, i.e. GSA or DoD.

$$R_s^\mu = \frac{100.\alpha_s}{a.\alpha_s + b} \tag{4.10}$$

$$\mu_{upgr.,s} = R_s^\mu \ . \ \mu_{orig.} \tag{4.11}$$

Using these reduction factors, the displacement ductility demand in the beam after upgrading can be estimated according to Equation 4.11. It was also concluded that for the case of retrofitting the beams by increasing both stiffness and strength, displacement ductility demand in a beam after upgrading $\mu_{up.,s,k}$ can be considered approximately equal to $\mu_{up.,s}$, as the reduction factor for retrofitting by increasing stiffness only R_k^{μ} is about 100%.

It is worth mentioning that the values of the coefficients in Tables 4.2, 4.3, 4.5, 4.6, and 4.8 for a coefficient of determination, R^2, ranged from 0.9 to 1.0.

4.3. EFFECT OF VARIATION OF BAY AREA (BAY SPAN)

In this section, the effect of variation of bay span on the values of the chord rotation, tie force, and displacement ductility demand (for the cases of building before and after upgrade) were studied by considering three other different spans 5.0m, 7.5m and 9.0m.

This section investigates the effect of variation of bay area on the response of the studied 18-storey building by the results to those of the model bay area 6m x6m (reference model). Three parameters:- chord rotation, tie force and displacement ductility demand in beams - will be investigated. Three other models were studied with different bay areas of 5m x 5m, 7.5m x 7.5m and 9m x 9m and by applying the GSA load combination.

For different scenarios of removed columns in bay areas 7.5m x7.5m and 9m x 9m, all the scenarios failed for the existing building before upgrading and had to be upgraded by a strength factor of 1.1.

4.3.1. Effect of variation of bay area on chord rotation (θ)

It was found that the most critical case for models of the spans 5m, 6m (reference model) and 7.5m was the scenario of removing the Edge Short Column. While for the model of span of 9m the most critical case was removing of the Corner Column. From the analysis of models with different spans, it can be concluded that the perimeter column loss scenario is more critical than the interior column loss scenarios. Moreover, as span increases significantly, the removal of the Corner Column scenario will become the most critical.

Similar observations related to the reference model were found in models of spans 5.0 m, 7.5 m and 9.0 m. Vertical deflection in the cases of the removal of (IC) and (ELC) is more than that of (FIC) and (FELC), respectively. Also, the vertical deflection in removing (ESC) is more than that of removal of (ELC).

Effect of variation in bay area on chord rotation A_s^θ and A_k^θ were investigated. A_s^θ or is equal to the ratio of chord rotation of two different span models at the same upgrading level. Figures 4.24 to 4.26 show the factor for effect of variation in bay area on chord rotation in models of span 5m, 7.5m and 9m, respectively, as compared to bay span of 6.0 m for all column loss scenarios when beams are retrofitted by increasing their strength or stiffness only. From the figure it can be seen that the average value for the six scenarios of column removal fluctuates around 91%, 112% and 122% for buildings with spans of 5.0 m, 7.5 m and 9.0m, respectively. These values for A_s^θ and A_k^θ were found to be close to the square root of the ratio of spans, as shown in Equation 4.13. This equation is valid for the three cases of retrofit by increasing strength and/or stiffness only for all column loss scenarios. Using this factor, one can predict the upgraded chord rotaton for different bay areas using the following equation:

$$\theta_{upgr.,s,k} = A_{s,k}^\theta \bullet R_s^\theta \bullet R_K^\theta \bullet \theta_{orig.} \quad (4.12)$$

$$A_{s,k}^\theta = A_K^\theta = A_s^\theta = \frac{\theta_{orig.,1}}{\theta_{orig.,2}} = \frac{\theta_{upgr.,1}}{\theta_{upgr.,2}} = \left[\frac{L_1}{L_2}\right]^{0.5} \quad (4.13)$$

Where: L_1 ,L_2 are two different bay span,where L_2 were taken as span of reference model (6.0m) for Figures (4.24-4.26).

Table 4.9 shows the ratio of values of the displacement ductility demand for different spans A_s^{θ} as obtained from the analysis and as estimated by equation 4.13. The table shows that the difference between the estimated and obtained values is insignificant.

4.3.2. Effect of variation of bay area on Tie Forces (TF)

Similar observations to the reference model were found in models of spans of 5.0 m, 7.5 m and 9.0 m. The removal of (IC) and (ELC) exert a higher tie force than (FIC) and (FELC), respectively. Also, the tie force exerted in the scenario of the removal of the Edge Long Column is higher than that in scenario of the removal of the Edge Short Column.

Effect of variation in bay areas on ties force A_s^T and A_k^T were investigated. A_s^T or A_k^T is equal to the ratio of tie forces of two different span models at the same upgrading level. Figures 4.27 to 4.29 show the factor for effect of variation in bay area on tie force in models of span 5m, 7.5m and 9m, respectively, as compared to bay span of 6.0 m for all column loss scenarios when beams are retrofitted by increasing their strength or stiffness only. From the figure it can be seen that the average value for the six scenarios of column removal fluctuates around 57%, 195% and 337% for buildings with spans of 5.0 m, 7.5 m and 9.0m, respectively. These values for A_s^T and A_k^T were found to be close to the ratio of spans cubed, as shown in Equation 4.15. This equation is valid for the three cases of retrofit by increasing strength and/or stiffness only for all column loss scenarios. Therefore, using this factor, it is possible to predict the upgraded tie force for different bay areas by the following equation:

$$TF_{upgr.,s,k} = A_{s,k}^T \bullet R_s^T \bullet R_K^T \bullet TF_{orig.} \qquad (4.14)$$

$$A^{T}_{s,k} = A^{T}_{K} = A^{T}_{s} = \frac{TF_{orig.,1}}{TF_{orig.,2}} = \frac{TF_{upgr.,1}}{TF_{upgr.,2}} = \left[\frac{L_1}{L_2}\right]^3 \qquad (4.15)$$

Where: L_1 ,L_2 are two different bay span,where L_2 were taken as span of reference model (6.0m) for Figures (4.27-4.29).

Table 4.10 shows the ratio of values of the displacement ductility demand for different spans A^{T}_{s} as obtained from the analysis and as estimated by equation 4.15. The table shows that the difference between the estimated and obtained values is insignificant.

4.3.3. Effect of variation of bay area on displacement ductility demand (μ_Δ)

The highest ductility demand was found to be in the case of removing the Edge Short Column for spans 5.0 m, 6.0 m and 7.5m, while for the span of 9.0 m the removal of the Corner Column was found to be the highest among the scenarios. Also, it was found that the effect of the location of the removed column on the level of displacement ductility demand follows similar trend as that observed for chord rotations.

Effect of variation in bay areas on displacement ductility demand A^{μ}_{s} and A^{μ}_{k} were investigated. A^{μ}_{s} or A^{μ}_{k} is equal to the ratio of displacement ductility demand of two different span models at the same upgrading level. Figures 4.30 to 4.32 show the factor for effect of variation in bay area on displacement ductility demand in models of spans of 5.0 m, 7.5 m and 9.0 m, respectively, as compared to bay span of 6.0 m for all column loss scenarios when beams are retrofitted by increasing strength or stiffness only. From the figure it can be seen that the average value for the six scenarios of column removal fluctuates around 83%, 125% and 150% for buildings with spans of 5.0 m, 7.5 m and 9.0m, respectively. These values for A^{μ}_{s} and A^{μ}_{k} were found to be close to the ratio of spans, as shown in Equation 4.17. This equation is valid for the three cases of retrofit by increasing strength and/or stiffness only for all column loss scenarios. Using this factor can predict the upgraded displacement ductility demand for different bay areas by the following equation:

$$\mu_{upgr.,s,k} = A^{\mu}_{s,k} \bullet R^{\mu}_{s} \bullet R^{\mu}_{K} \bullet \mu_{orig.} \tag{4.16}$$

$$A^{\mu}_{s,k} = A^{\mu}_{K} = A^{\mu}_{s} = \frac{\mu_{orig.,1}}{\mu_{orig.,2}} = \frac{\mu_{upgr.,1}}{\mu_{upgr.,2}} = \left[\frac{L_1}{L_2}\right] \tag{4.17}$$

Where: L_1 ,L_2 are two different bay span,where L_2 were taken as span of reference model (6.0m) for Figures (4.30-4.32).

Table 4.11 shows the ratio of values of the displacement ductility demand for different spans A^{μ}_{s} as obtained from the analysis and as estimated by equation 4.17. The table shows that the difference between the estimated and obtained values is insignificant.

Table 4.1. Maximum deflection and corresponding (chord rotation) for all column removal scenarios for the existing building under GSA loading and for upgraded building by strength factor of 1.25 under DoD loading

Removed column	GSA 2003	DoD 2005
Edge Short Column	1070 mm (10.1°)	1168 mm (11.0°)
Corner Column	930 mm (8.8°)	1020 mm (9.6°)
Internal Column	876 mm (8.3°)	973 mm (9.2°)
First Internal Column	819 mm (7.77°)	921 mm (8.8°)
Edge Long Column	737 mm (7°)	822 mm (7.8°)
First Edge long column	643 mm (6.1°)	728 mm (6.9°)

Table 4.2. Values of "a" and "b" coefficients for estimating the reduction factors R_s^θ and R_k^θ when subjected to GSA loading

Removed column	R_s^θ		R_k^θ	
	100 α_s /[a x α_s + b]		**1000/[a x α_k + b]**	
	a	**b**	**a**	**b**
Edge Short Column	6.10	-5.10	7.86	2.14
Corner Column	5.10	-4.10	6.35	3.65
Internal Column	4.30	-3.30	4.90	5.10
First Internal Column	4.85	-3.85	6.00	4.00
Edge Long Column	4.44	-3.44	6.40	3.60
First Edge Long Column	4.10	-3.10	6.82	3.18

Table 4.3. Values of "a" and "b" coefficients for estimating the reduction factors R_s^θ and R_k^θ when subjected to DOD loading

Removed column	R_s^θ		R_k^θ	
	100 α_s /(a x α_s +b)		**1000/(a x α_k +b)**	
	a	**b**	**a**	**b**
Edge Short Column	5.25	-5.31	7.65	2.6
Corner Column	4.52	-4.4	8.06	0.928
Internal Column	4.28	-4.1	8.3	0.53
First Internal Column	4.28	-4.1	7.87	1.5
Edge Long Column	3.81	-3.51	8.3	0.78
First Edge Long Column	3.6	-3.25	7.85	2.22

Table 4.4. Tie Forces (kN) in beams for all column removal scenarios for GSA loading of the existing building under GSA loading and for upgraded building by strength factor of 1.25 under DoD loading

Removed column	GSA 2003	DoD 2005
Internal Column	1150	1340
First Internal Column	625	720
Edge Long Column	500	640
First Edge long Column	390	490
Edge Short Column	400	450
Corner Column	410	460

Table 4.5. Values of "a", "b" and "c" coefficients for estimating the reduction factors R_s^T and R_k^T when subjected to GSA loading

Removed column	R_s^T			R_k^T		
	100 α_s /(a x α_s^2 +b x α_s + c)			**1000/(a x α_k^2 +b x α_k + c)**		
	a	**b**	**c**	**a**	**b**	**c**
Internal Column	-1.38	10.6	-8.22	11.3	-1.3	0
First Internal Column	-0.01	4.91	5.1	-0.61	5.23	-3.62
Edge Long Column	-1.13	8.42	-6.29	-0.15	9.45	0.7
First Edge Long Column	-1.1	7.9	-5.8	-1.1	12.5	-1.4
Edge Short Column	-1.32	9.27	-6.95	-0.4	5.7	4.7
Corner Column	-1.3	9.8	-7.5	11.2	-1.2	0

Table 4.6. Values of "a", "b" and "c" coefficients for estimating the reduction factors R_s^T and R_k^T when subjected to DOD loading

Removed column	R_s^T			R_k^T		
	100 α_s /(a x α_s^2 +b x α_s + c)			**1000/(a x α_k^2 +b x α_k + c)**		
	a	**b**	**c**	**a**	**b**	**c**
Internal Column	-1.13	8.66	-7.8	-2.3	15.7	-4.7
First Internal Column	-0.76	5.8	-4.8	-2.4	16	-4.8
Edge Long Column	-1.1	8.32	-7.45	-2.19	15.4	-4
First Edge Long Column	-0.92	7.0	-6.05	-2.3	15.1	-2.5
Edge Short Column	-1.2	8.5	-7.5	-2.27	15.44	-3.96
Corner Column	-1.1	8.6	-7.7	-2.2	15.6	-4.6

Table 4.7. Displacement ductility demand, μ_{Δ}, of the beams for all scenarios for GSA Loading of the model before upgraded and DoD loadings after upgrading by strength factor of 1.25

Removed column	GSA 2003	DoD 2005
Edge Short Column	9.78	8.90
Corner Column	8.45	7.85
Internal Column	7.96	7.78
First Internal Column	7.45	7.10
Edge long Column	6.70	6.30
First Edge long Column	6.12	5.60

Table 4.8. Values of "a" and "b" coefficients for estimating the reduction factors for ductility demand in beams due to increasing strength, R_s^{μ}, when subjected to GSA and DoD loading

Loading case	Increase strength only $R_s^{\mu}=100\alpha_s/(a\mathrm{x}\alpha_s+b)$	
	a	**b**
GSA Loading	8	-7
DoD Loading	7.7	-8.38

Table 4.9. Ratios of chord rotations values (average of six scenarios of column removal) for different spans A_s^{θ} as obtained from the analysis, and as obtained from Equation 4.13, as well as the (percentage of error)

	Span 5.0m	Span 6.0m	Span 7.5m	Span 9.0m
Span 5.0m	1, 1	1.17, 1.10 (-6.3%)	1.33, 1.23 (-7.7%)	1.40, 1.34 (-4.2%)
Span 6.0m	0.85, 0.91 (6.8%)	1, 1	1.13, 1.12 (-1.3%)	1.20, 1.22 (2%)
Span 7.5m	0.76, 0.82 (7.4%)	0.89, 0.90 (0.8%)	1, 1	1.06,1.10 (3.3%)
Span 9.0m	0.72, 0.75 (4%)	0.84, 0.82 (-2.4%)	0.95, 0.91 (-3.5%)	1, 1

Table 4.10. Ratios of tie forces values (average of six scenarios of column removal) for different spans A_s^T as obtained from the analysis, and as obtained from Equation 4.15, as well as the (percentage of error)

	Span 5.0m	Span 6.0m	Span 7.5m	Span 9.0m
Span 5.0m	1, 1	1.84, 1.73 (-6%)	3.53, 3.37 (-4.3%)	6.10, 5.83 (-4.4%)
Span 6.0m	0.55, 0.58 (5.9%)	1, 1	1.92, 1.95 1.7%	3.31, 3.37 (1.8%)
Span 7.5m	0.29, 0.30 (3.8%)	0.53, 0.51 (-2.6%)	1, 1	1.74, 1.73 (-0.5%)
Span 9.0m	0.17, 0.17	0.30, 0.30	0.58, 0.58	1, 1

Table 4.11. Ratios of displacement ductility demand values (average of six scenarios of column removal) for different spans A_s^{μ} as obtained from the analysis, and as obtained from Equation 4.17, as well as the (percentage of error)

	Span 5.0m	Span 6.0m	Span 7.5m	Span 9.0m
Span 5.0m	1, 1	1.15, 1.20 (4%)	1.45, 1.50 (3%)	1.66, 1.80 (8.6%)
Span 6.0m	0.87, 0.83 (-3.9%)	1, 1	1.26, 1.25 (-0.9%)	1.44, 1.50 (4.3%)
Span 7.5m	0.69, 0.67 (-3.2%)	0.79, 0.80 (0.8%)	1, 1	1.14, 1.20 (5%)
Span 9.0m	0.60, 0.56 (-8%)	0.7, 0.67 (-4.2%)	0.88, 0.83 (-5%)	1, 1

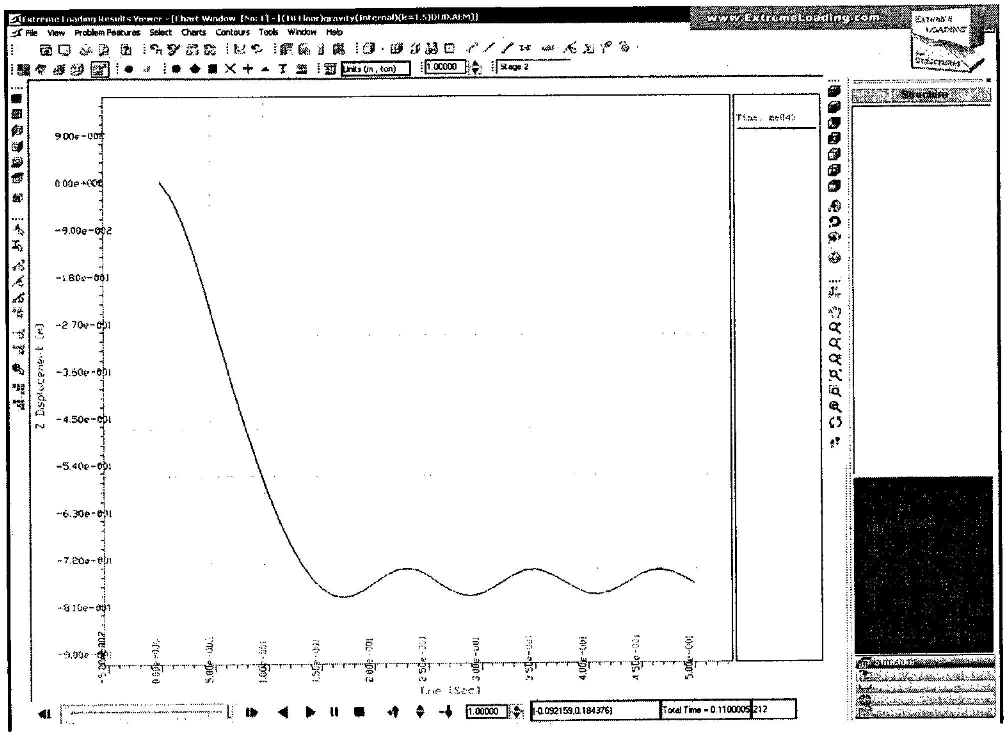


Figure 4.1. Deflection at point of the removed internal column from the output file of ELS software.

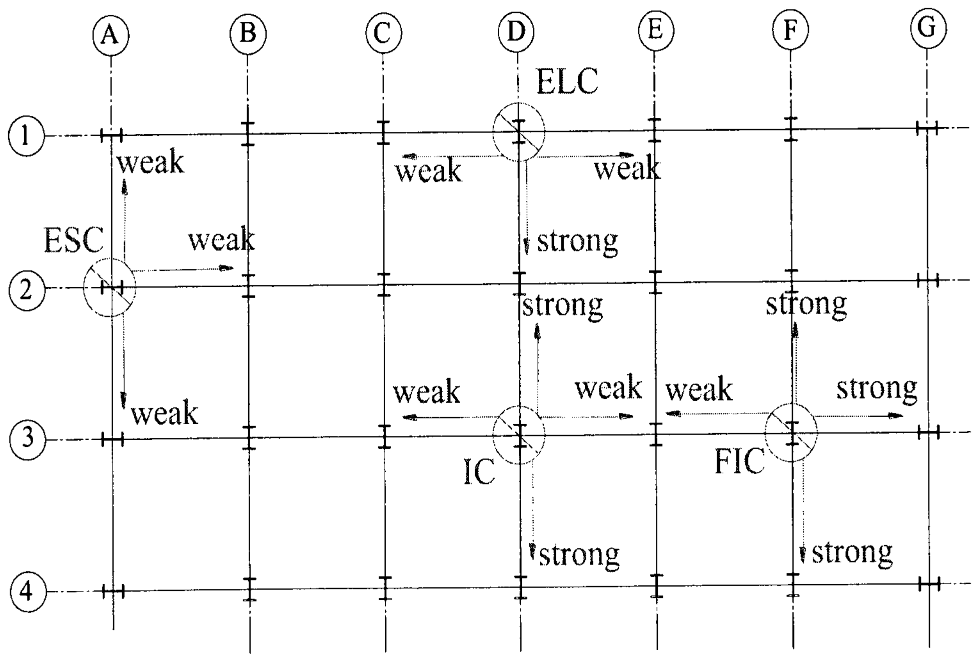
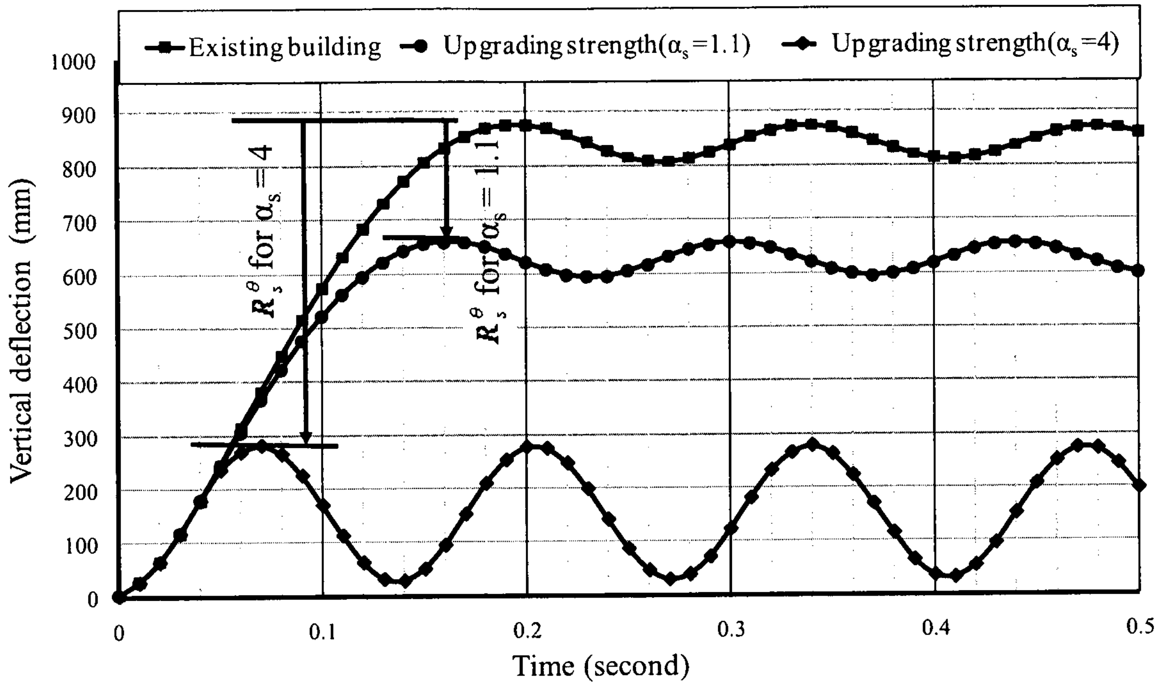


Figure 4.2. Illustration of strong and weak connections for cases of removal of Edge Short (ESC), Edge Long (ELC), Internal (IC) and First Internal (FIC) Columns.



$$R_s^\theta = \frac{\text{maximum deflection of upgraded building}}{\text{maximum deflection of existing building}} * 100$$

Figure 4.3. Time history response of the vertical deflection under the removed Internal Column of the existing building before upgrading and when upgraded by strength factor of 1.1 and 4 showing the reduction factor for chord rotation (R_s^θ).

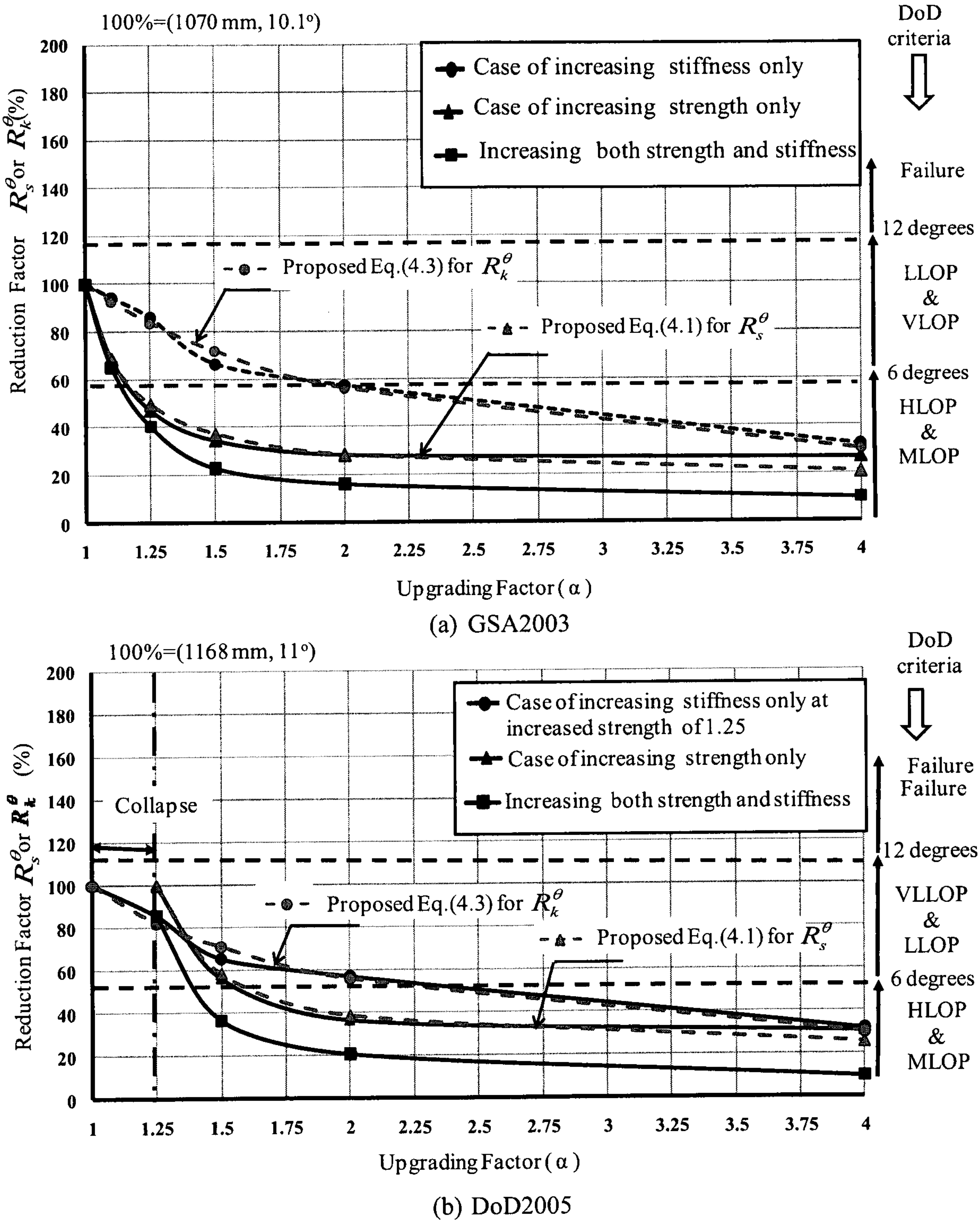
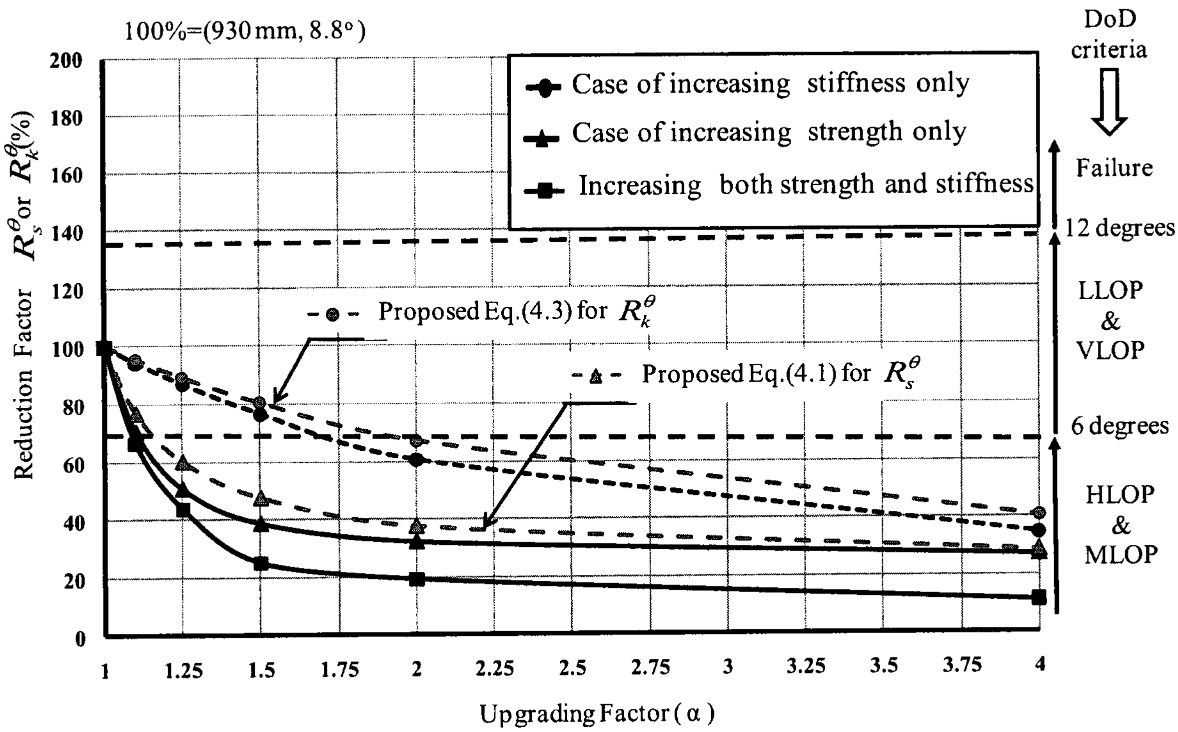
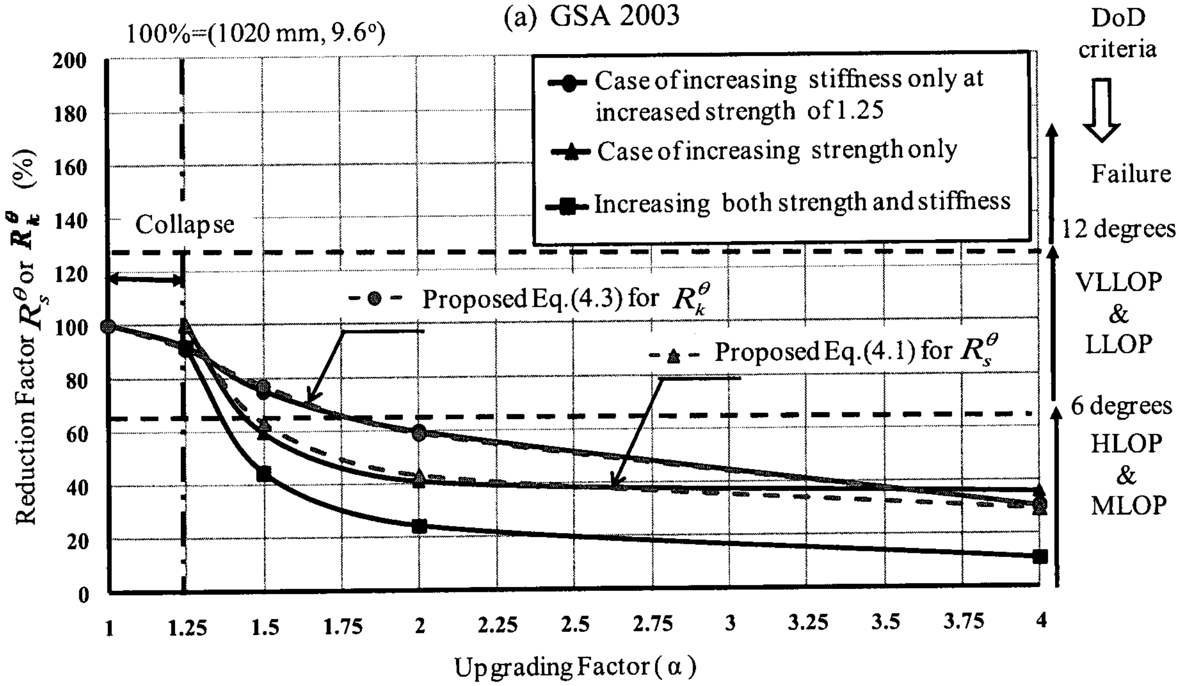


Figure 4.4. Reduction factors in chord rotation (θ) for case of removing the *Edge Short Column (ESC)* after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: a) GSA 2003; b) DoD 2005.

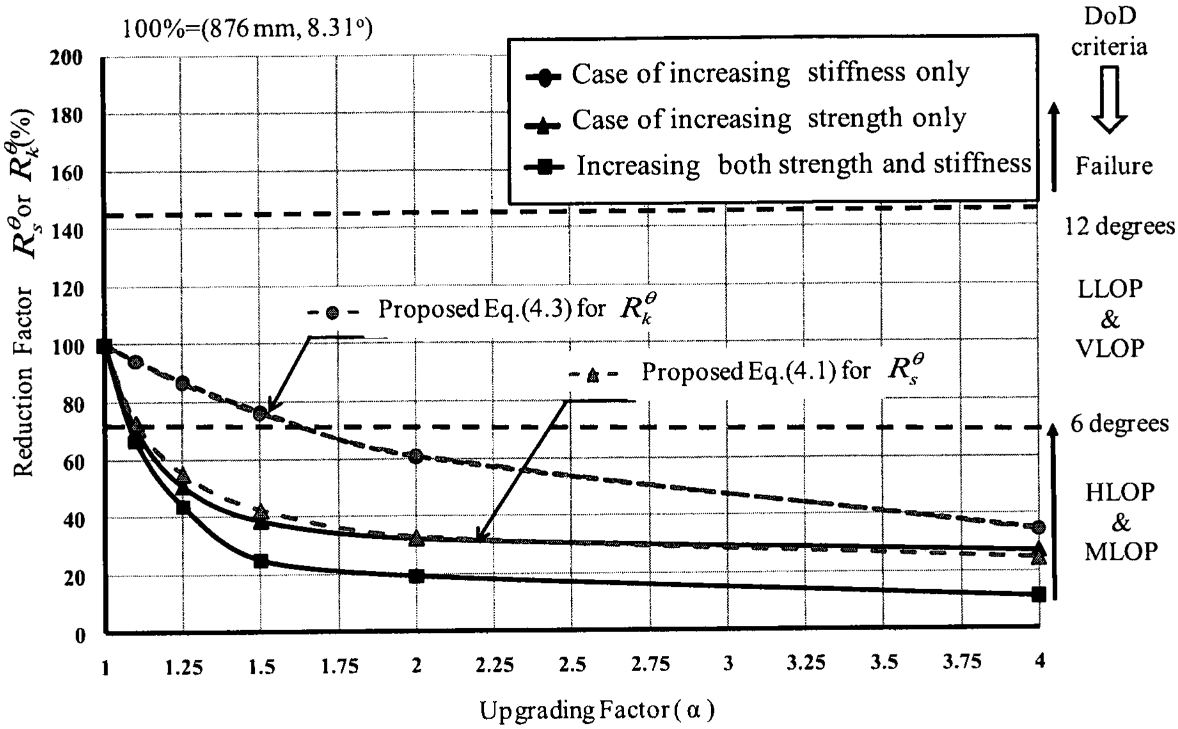


(a) GSA 2003

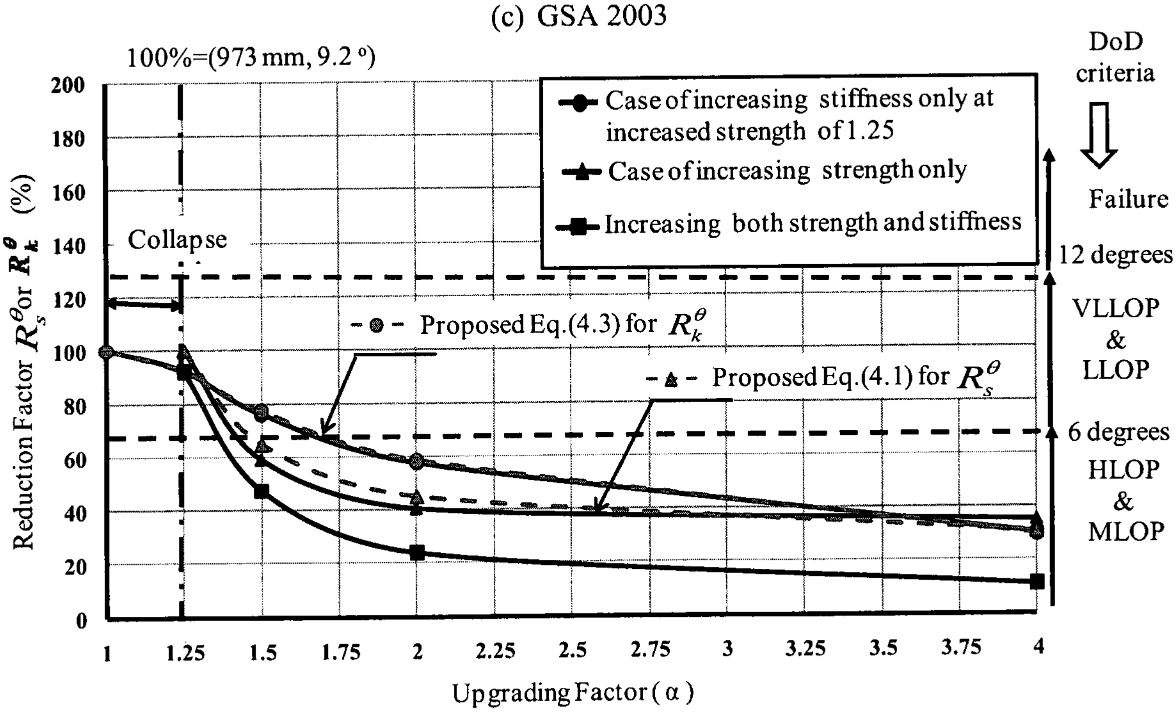


(b) DoD 2005.

Figure 4.5. Reduction factors in chord rotation (θ) for case of removing the *Corner Column (CC)* after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: a) GSA 2003; b) DoD 2005.

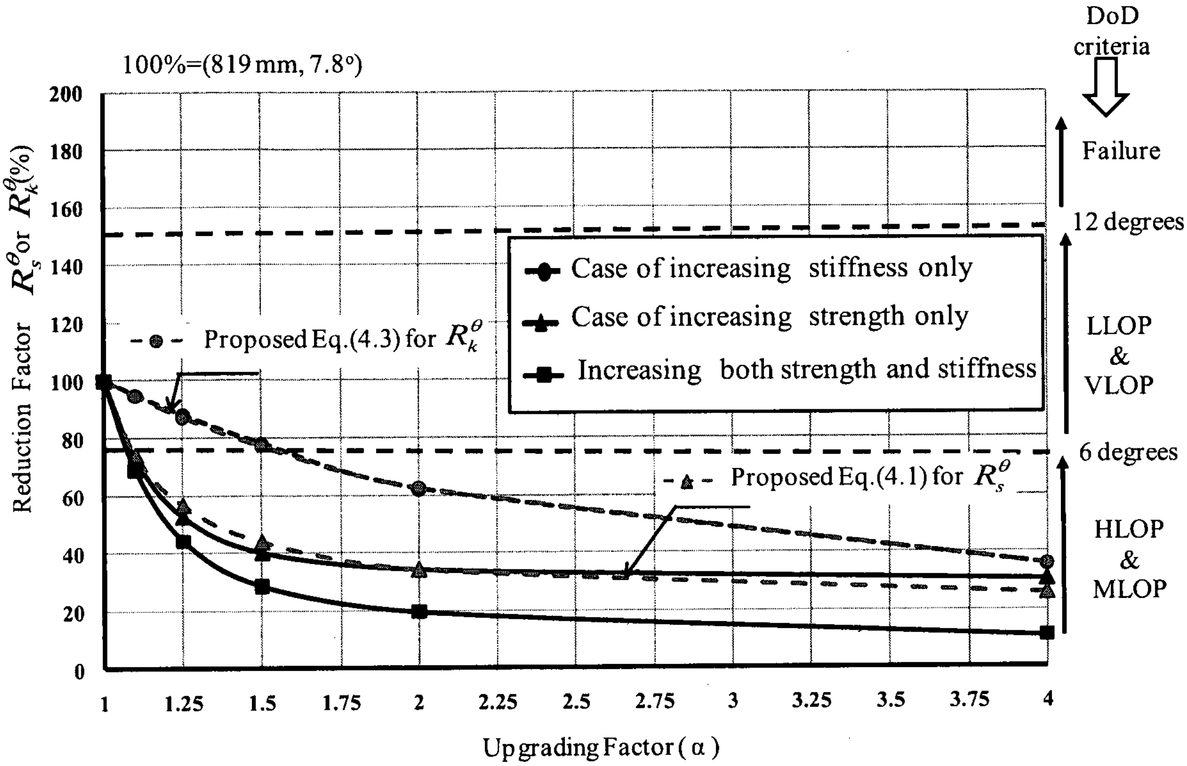


(c) GSA 2003

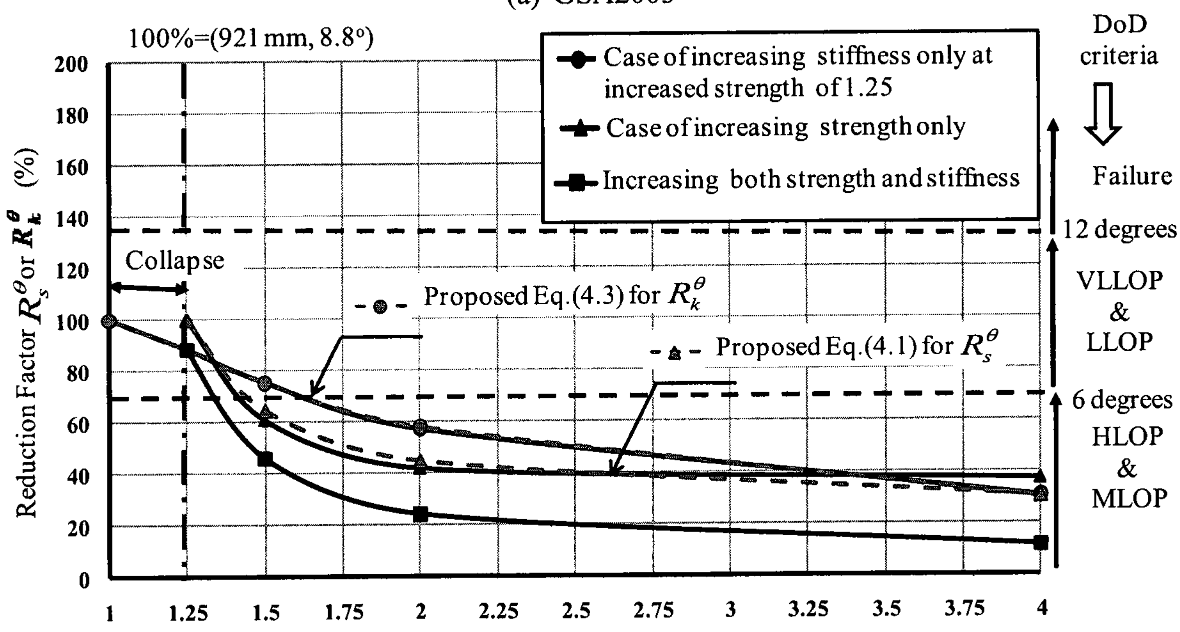


(c) DoD 2005.

Figure 4.6. Reduction factors in chord rotation (θ) for case of removing the *Internal Column (IC)* after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: a) GSA 2003; b) DoD 2005.

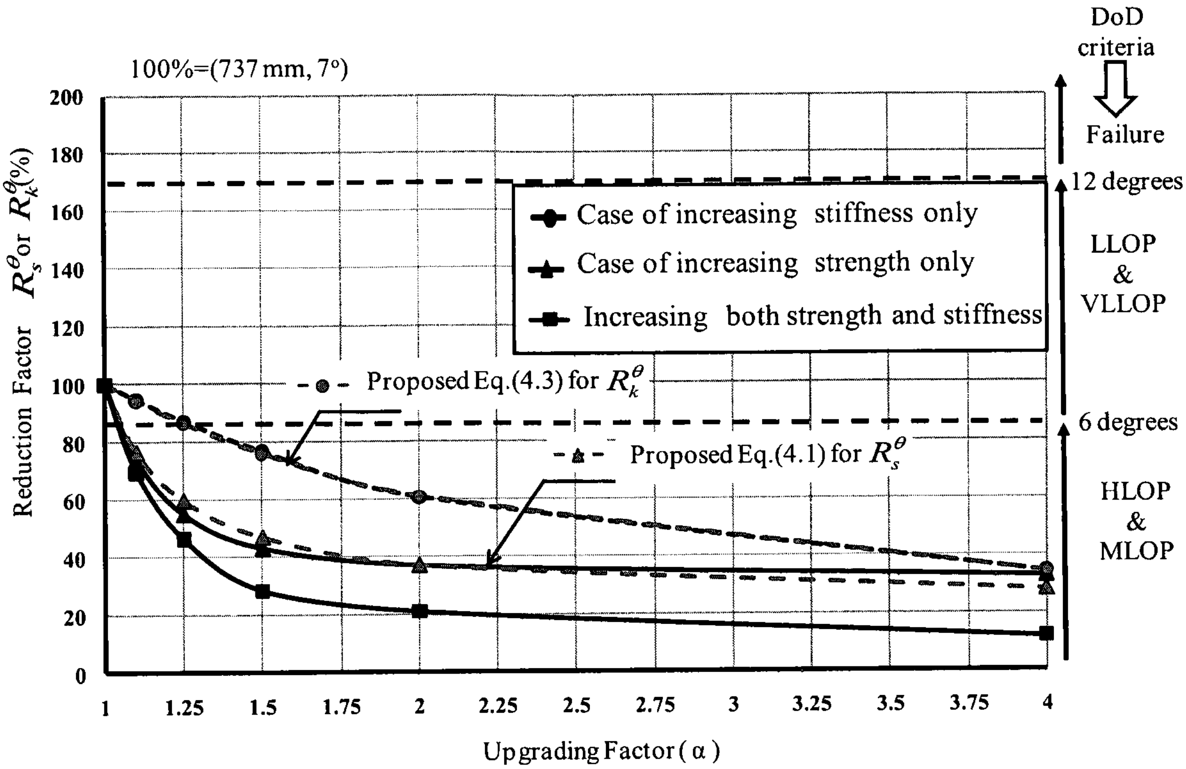


(a) GSA2003

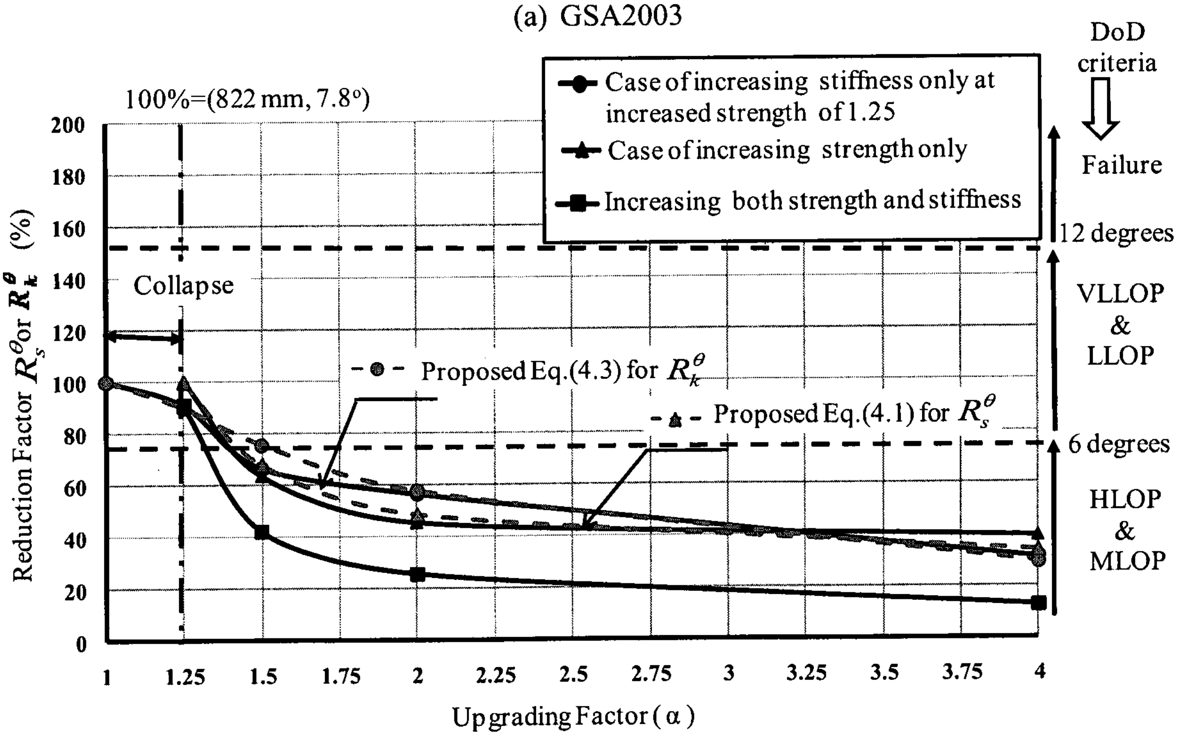


(b) DoD 2005

Figure 4.7. Reduction factors in chord rotation (θ) for case of removing the *First Internal Column (FIC)* after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: a) GSA 2003; b)DoD 2005.



(a) GSA2003



(b) DoD2005

Figure 4.8. Reduction factors in chord rotation (θ) for case of removing the *Edge Long Column (ELC)* after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: a) GSA 2003; b) DoD 2005.

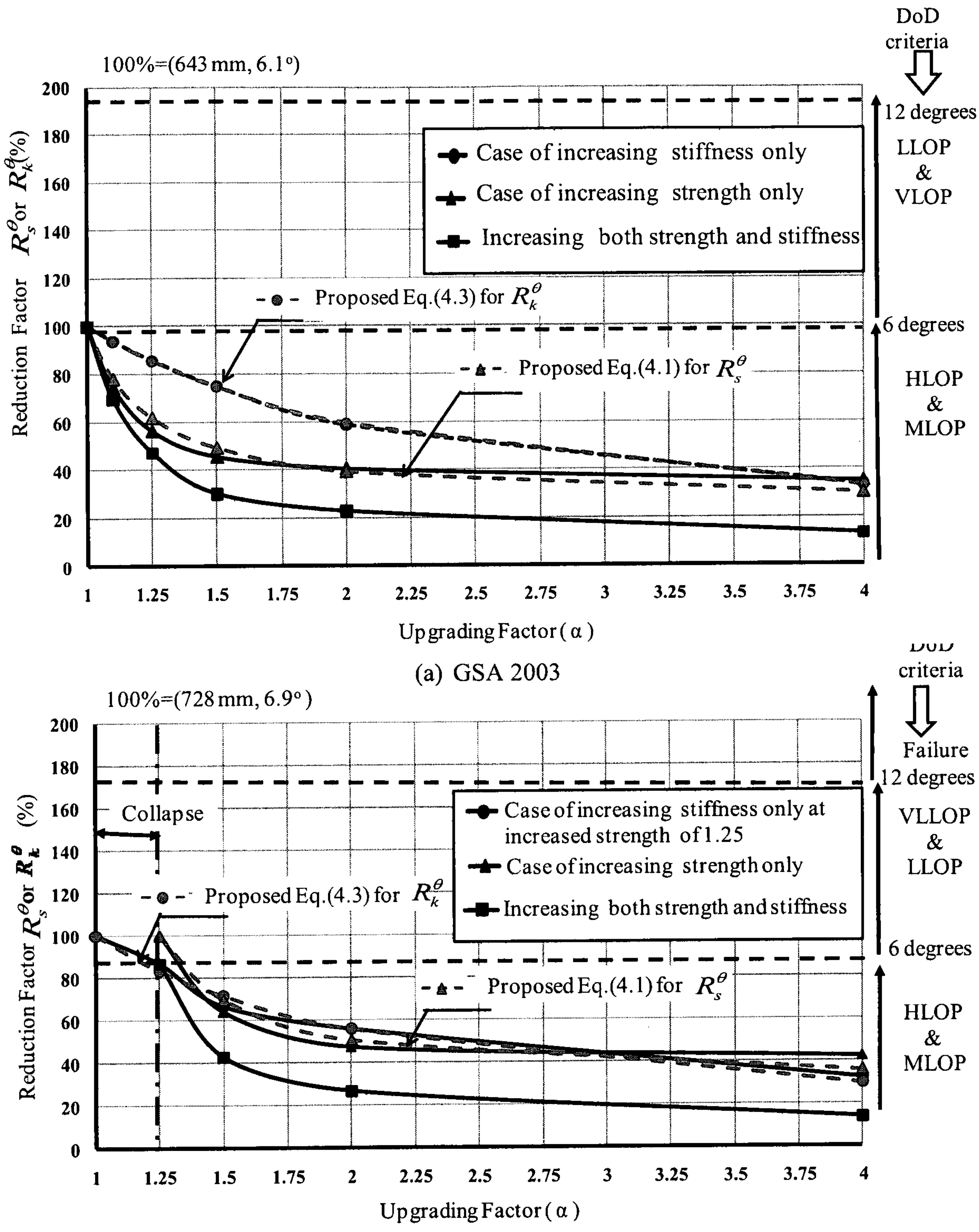


Figure 4.9. Reduction factors in chord rotation (θ) for case of removing the *First Edge Long Column (FELC)* after increasing strength and/or stiffness only and the proposed equations for R_s^θ & R_k^θ for loading according to: a) GSA 2003; b) DoD 2005.

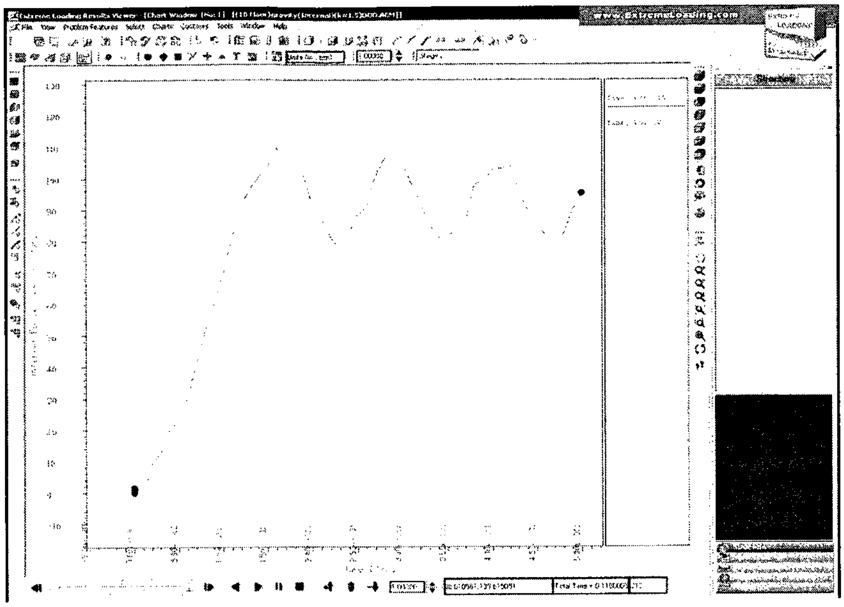


Figure 4.10. Tie force in beams from the output file of ELS software.

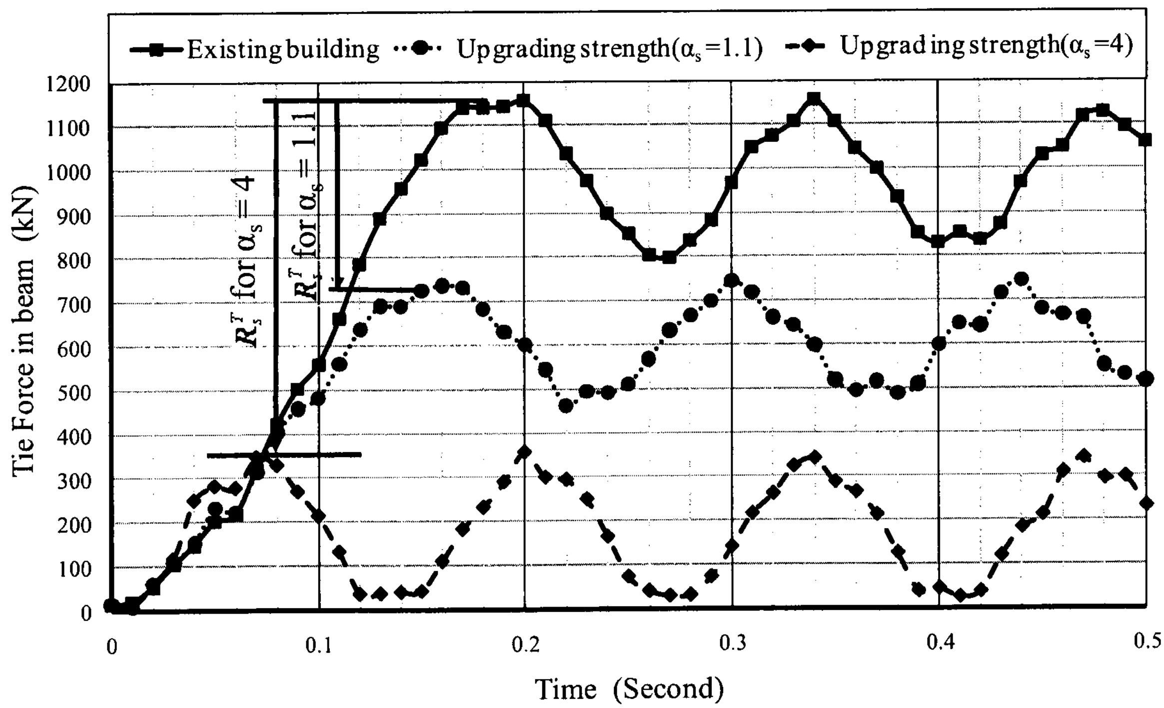
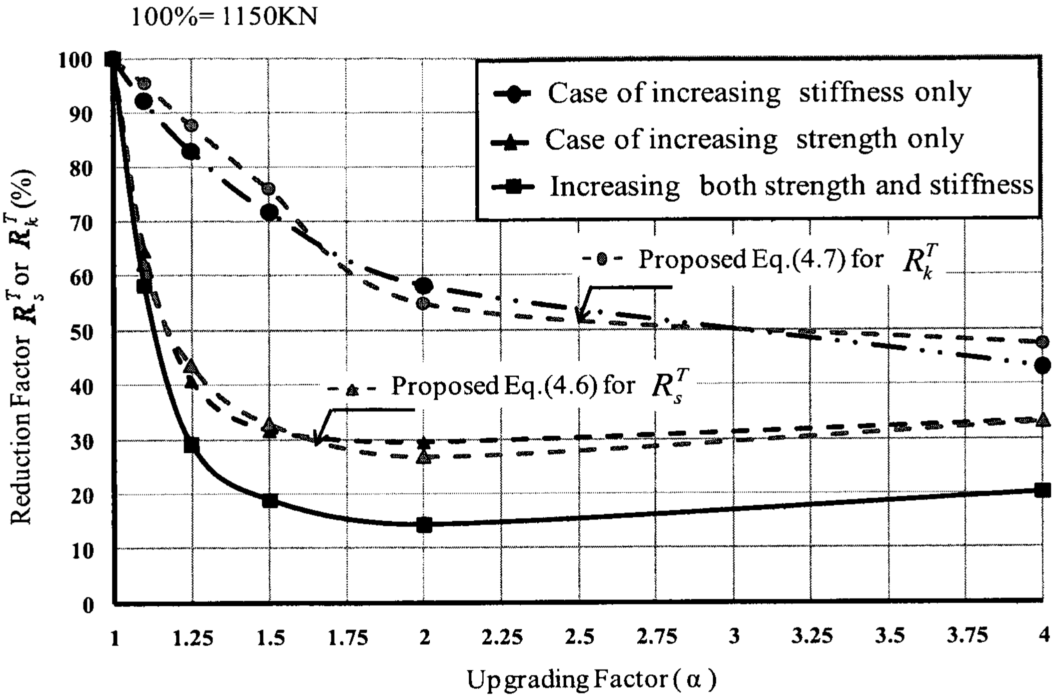
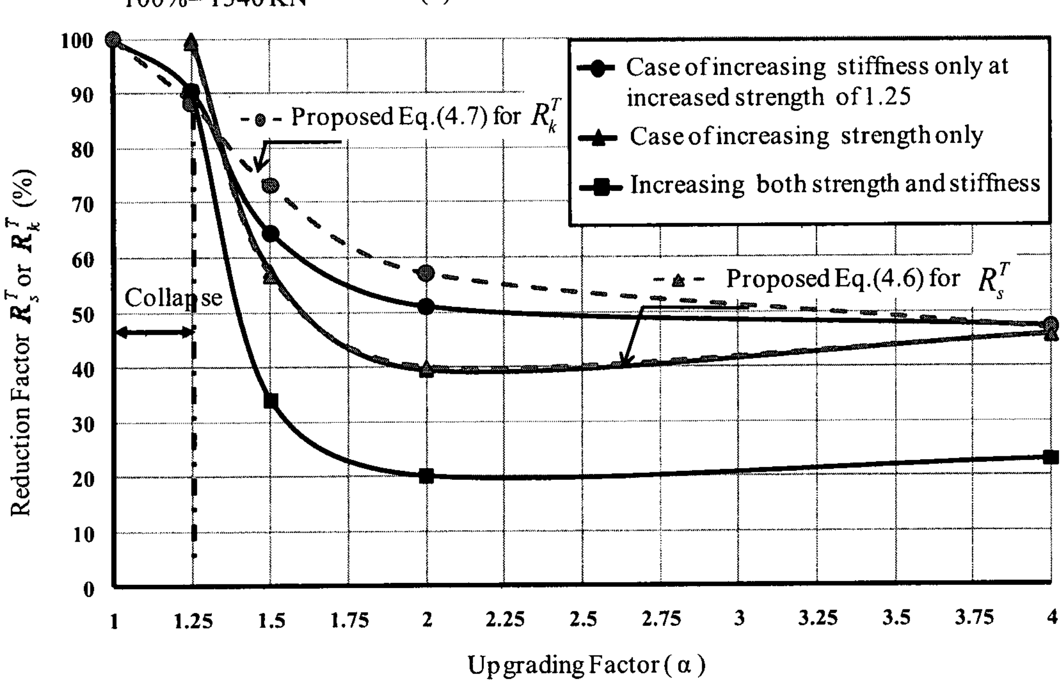


Figure 4.11. Time history response of tie force in beams under the removed Internal Column of the existing building before upgrading and when upgraded by strength factor of 1.1 and 4 showing the reduction factor for tie force (R_s^T).

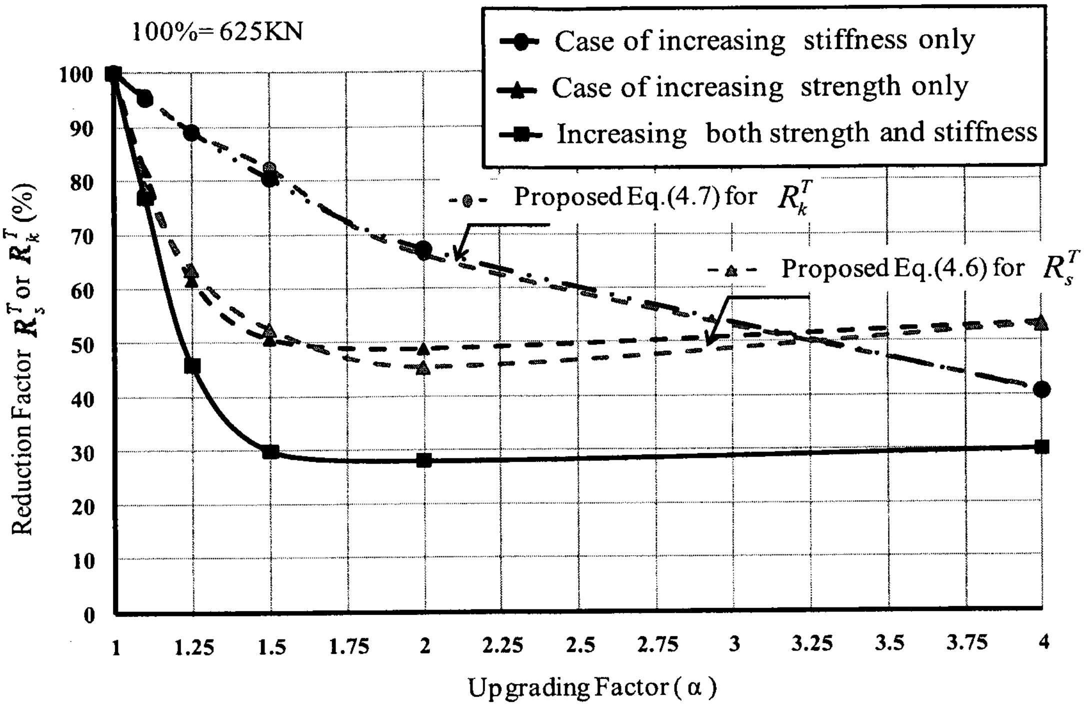


(a) GSA 2003

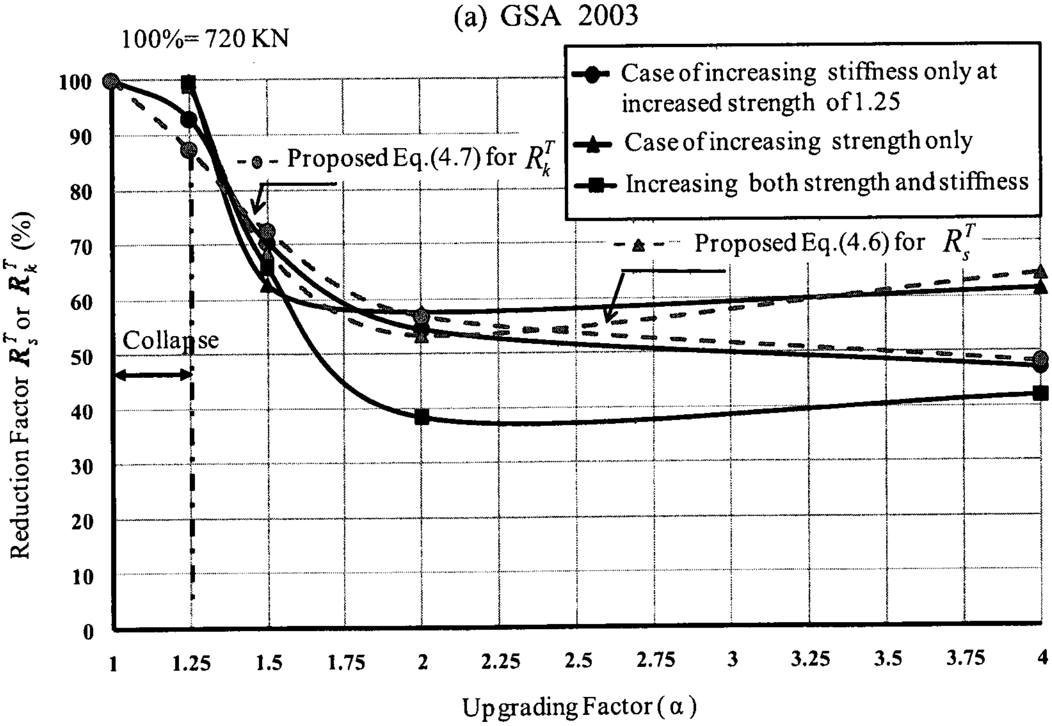


(b) DoD 2005

Figure 4.12. Reduction factors in Tie Force (TF) for case of removing the *Internal Column(IC)* after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: a) GSA 2003; b) DoD 2005.

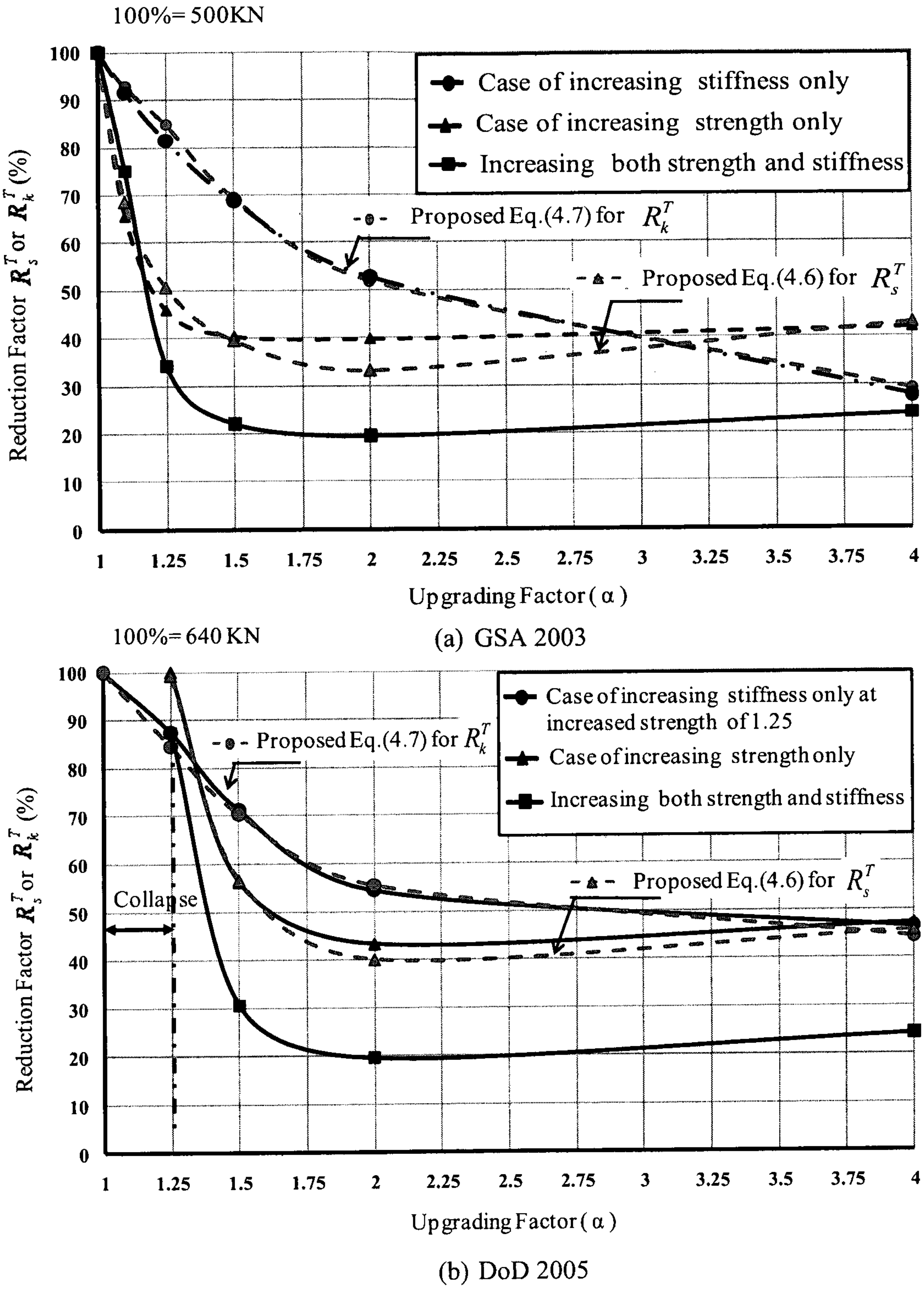


(a) GSA 2003



(b) DoD 2005

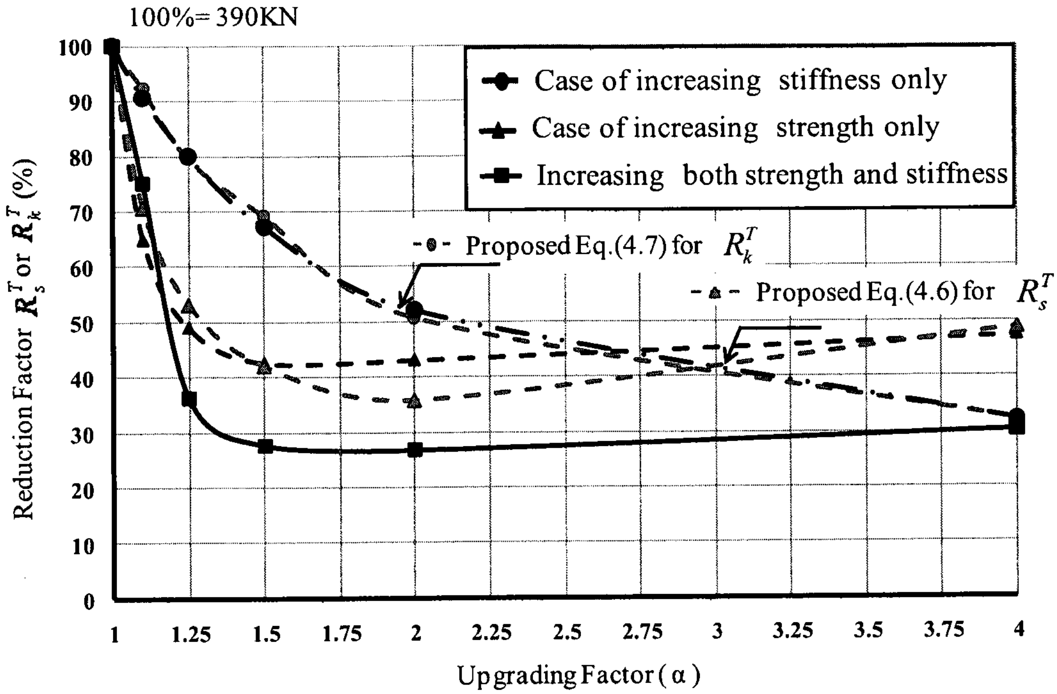
Figure 4.13. Reduction factors in Tie Force (TF) for case of removing the *First Internal Column (FIC)* after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: a) GSA 2003; b) DoD 2005.



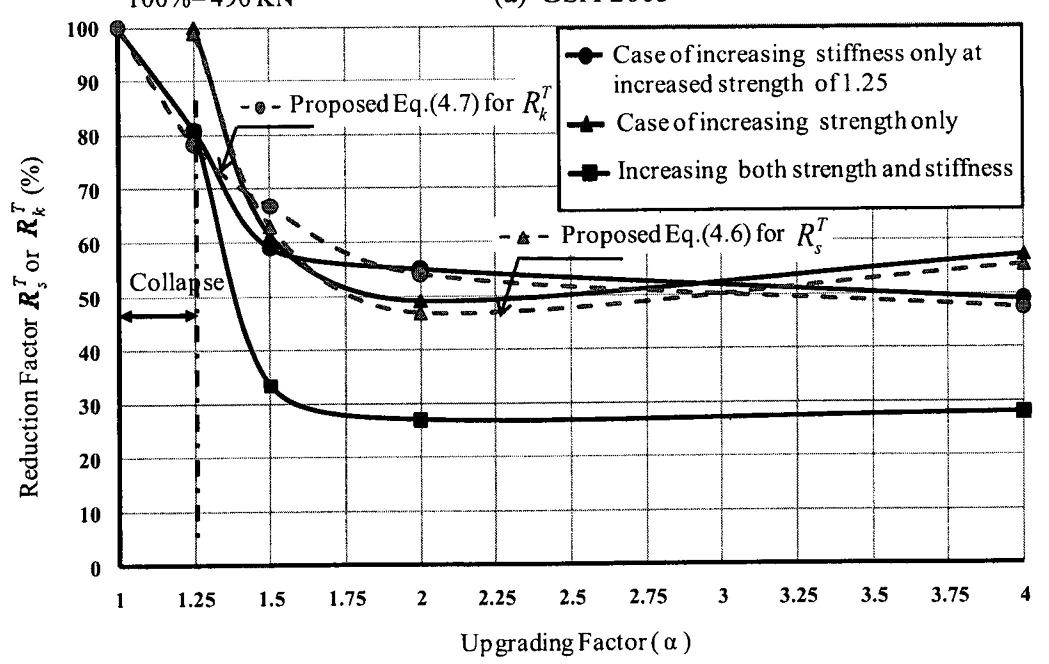
(a) GSA 2003

(b) DoD 2005

Figure 4.14. Reduction factors in Tie Force (TF) for case of removing the *Edge Long Column (ELC)* after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: a) GSA 2003;b)DoD 2005.

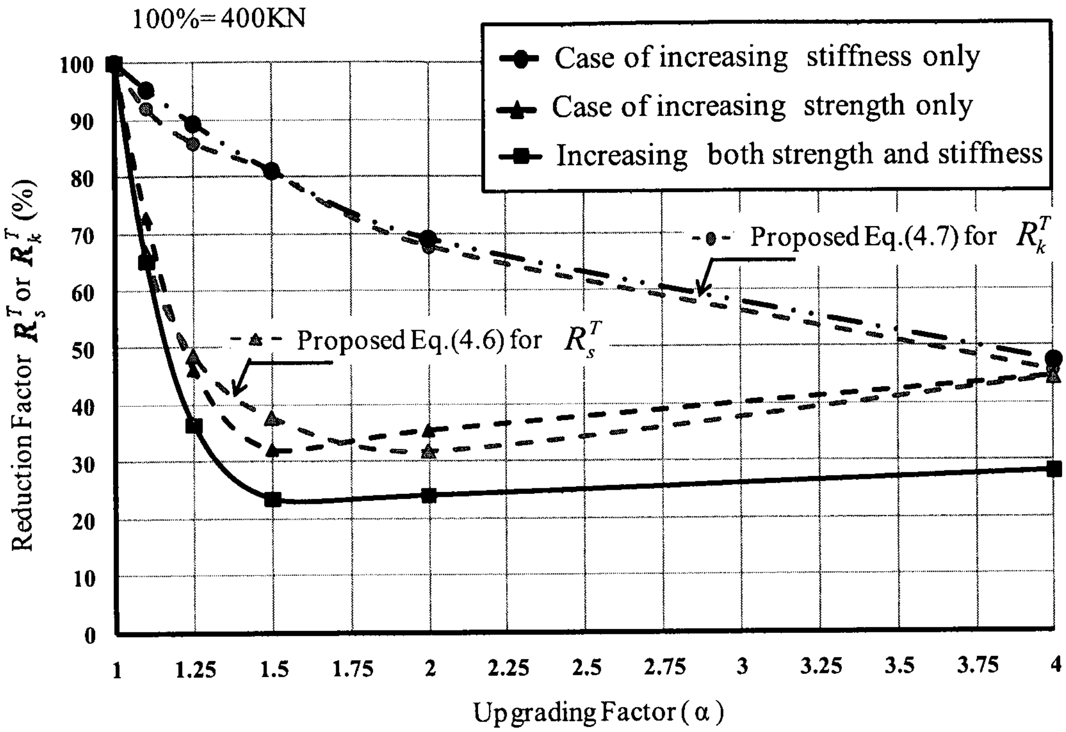


(a) GSA 2003

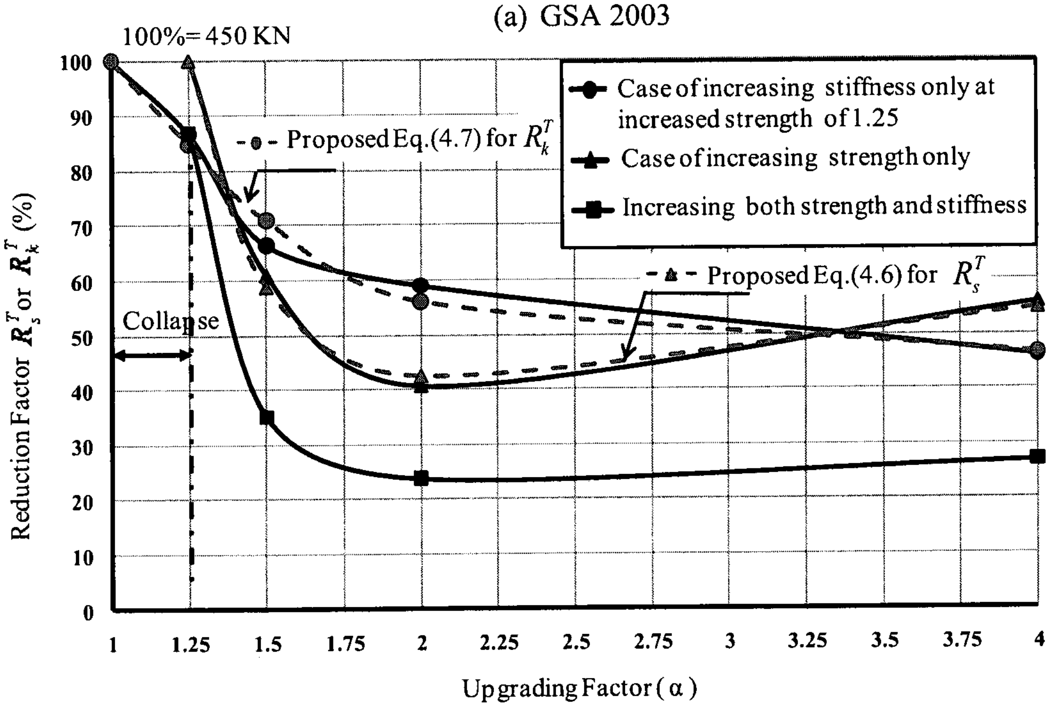


(b) DoD 2005

Figure 4.15. Reduction factors in Tie Force (TF) for case of removing the *First Edge Long Column (FELC)* after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: a) GSA 2003;b)DoD 2005.

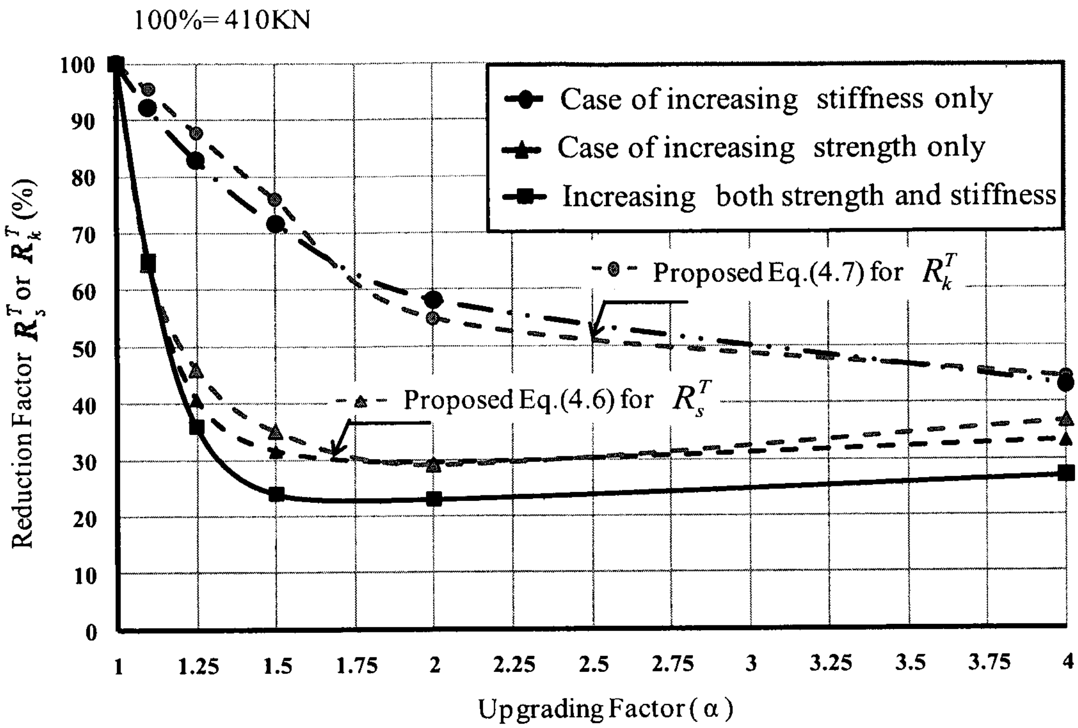


(a) GSA 2003

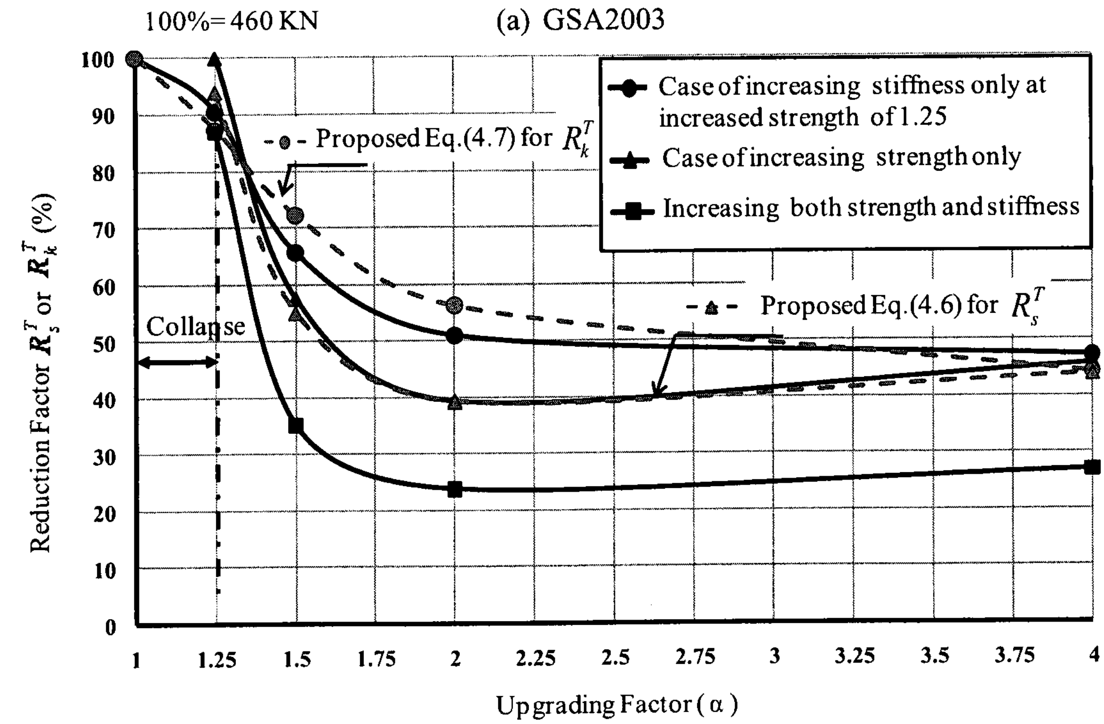


(b) DoD 2005

Figure 4.16. Reduction factors in Tie Force (TF) for case of removing the *Edge Short Column (ESC)* after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: a) GSA 2003;b) DoD 2005.

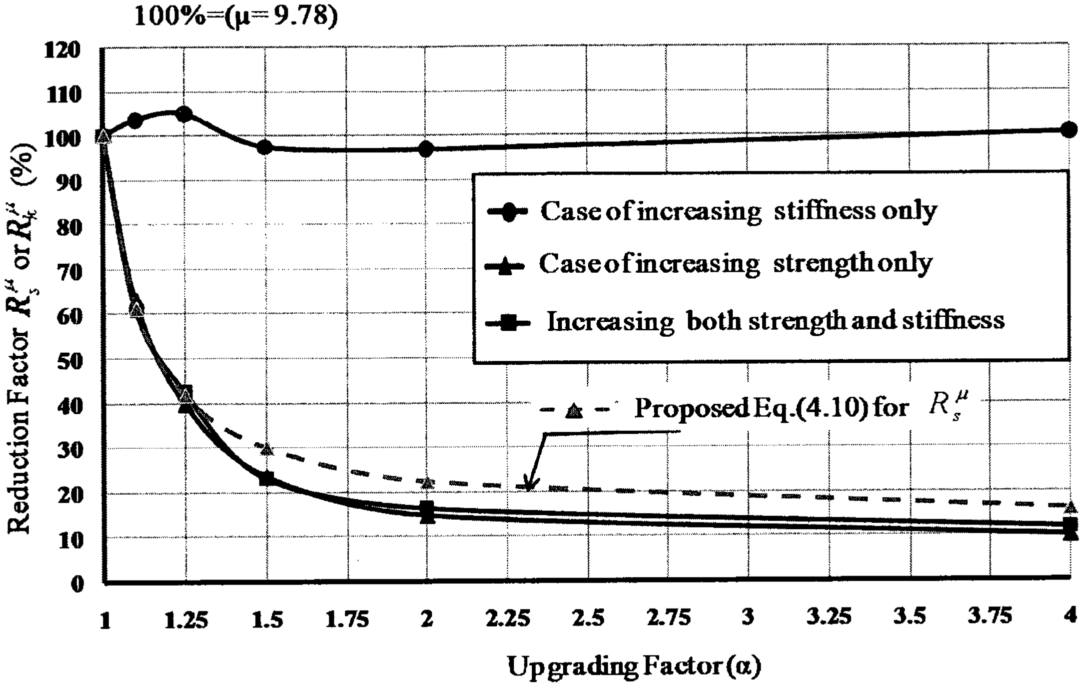


(a) GSA2003

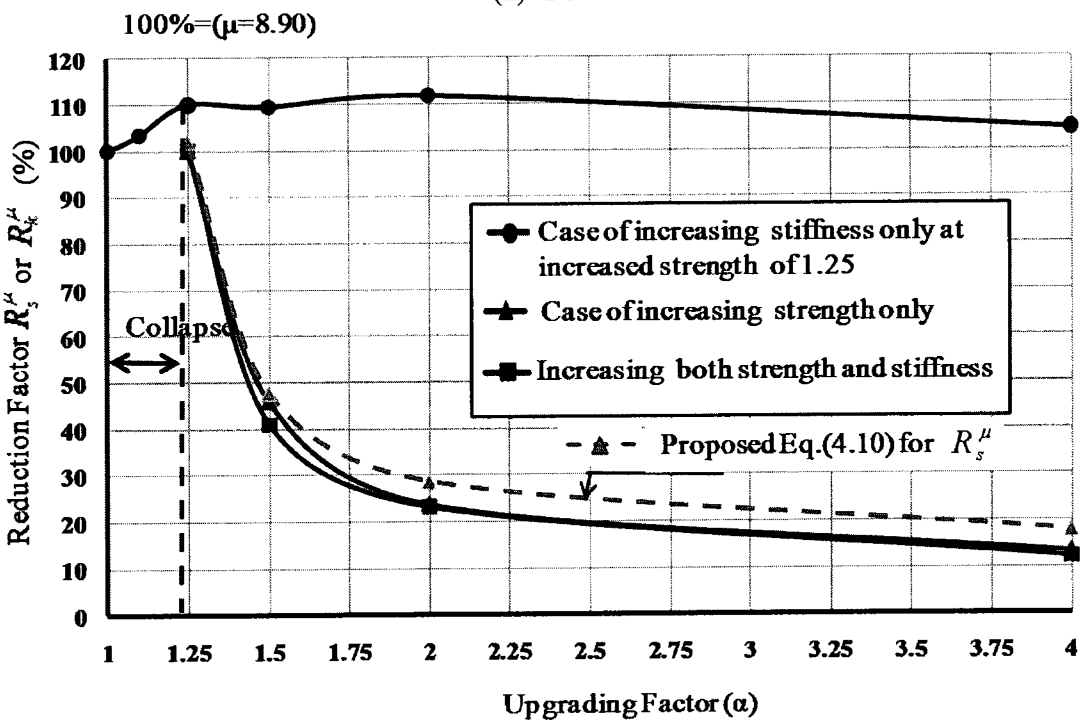


(b) DoD2005

Figure 4.17. Reduction factors in Tie Force (TF) for case of removing the *Corner Column (CC)* after increasing strength and/or stiffness only and the proposed equations for R_s^T & R_k^T for loading according to: a) GSA 2003;b) DoD 2005.

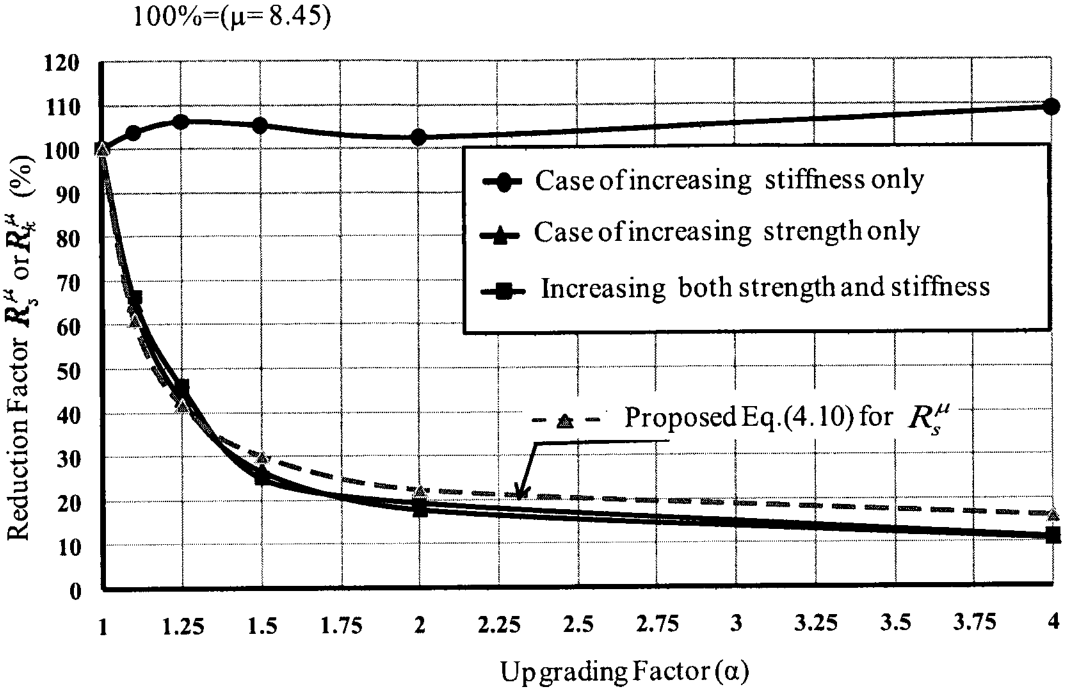


(a) GSA2003

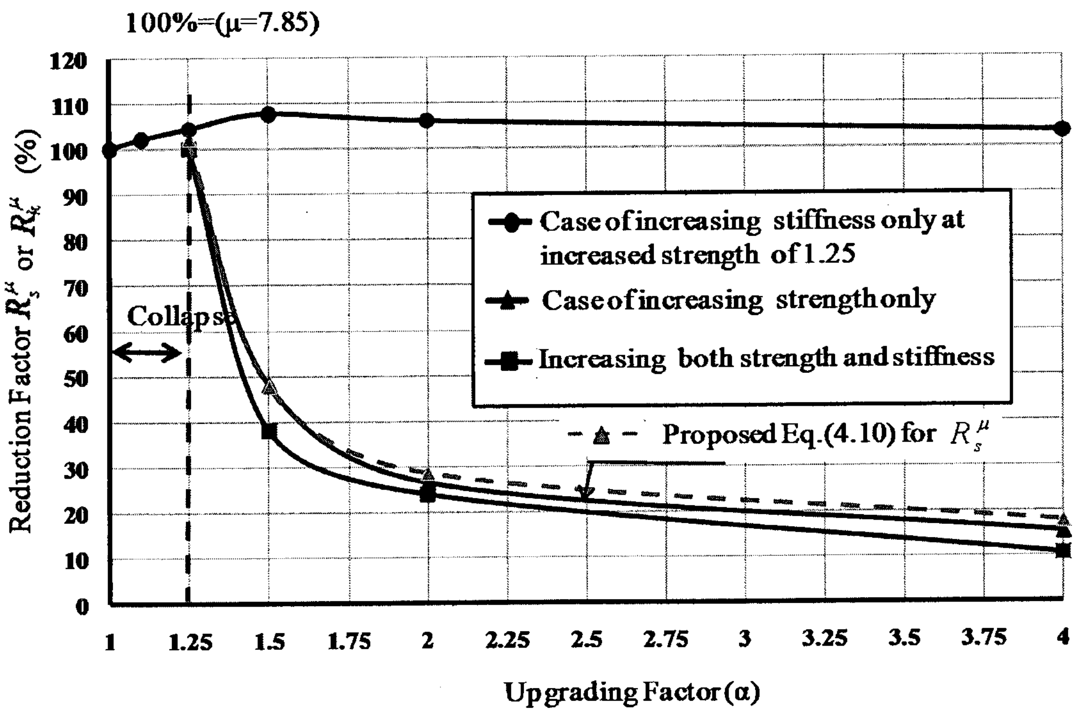


(b) DoD 2005

Figure 4.18. Reduction factors in displacement ductility demand (μ_Δ) for case of removing the *Edge Short Column (ESC)* after increasing strength and/or stiffness only and the proposed equations for R_s^μ & R_k^μ for loading according to: a) GSA 2003;b) DoD 2005.

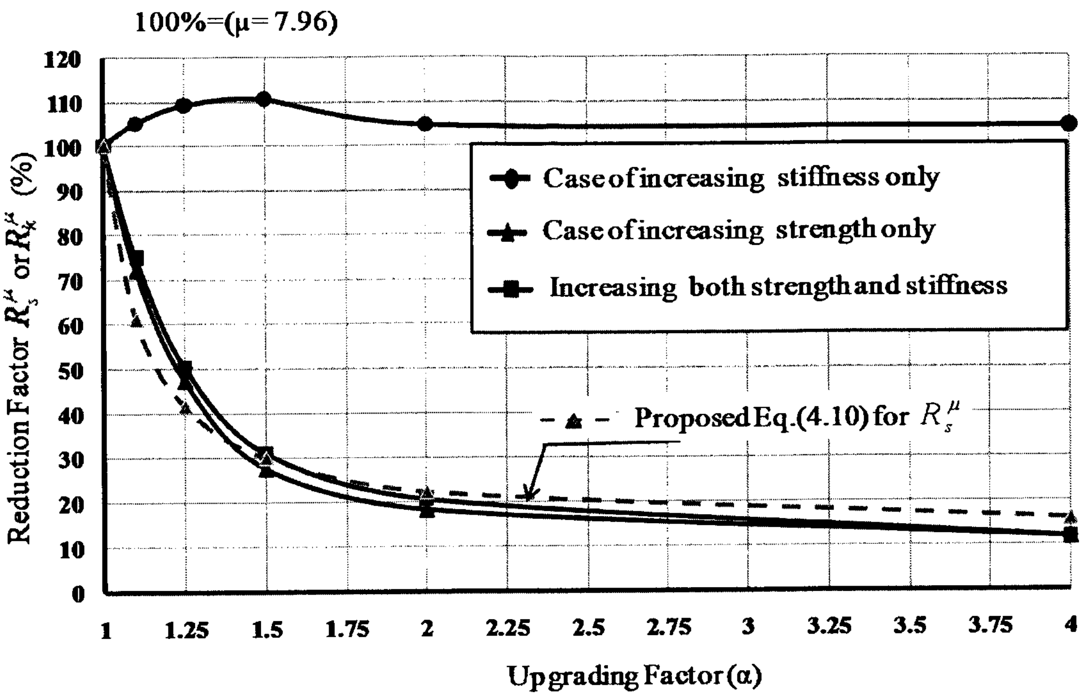


(a) GSA2003

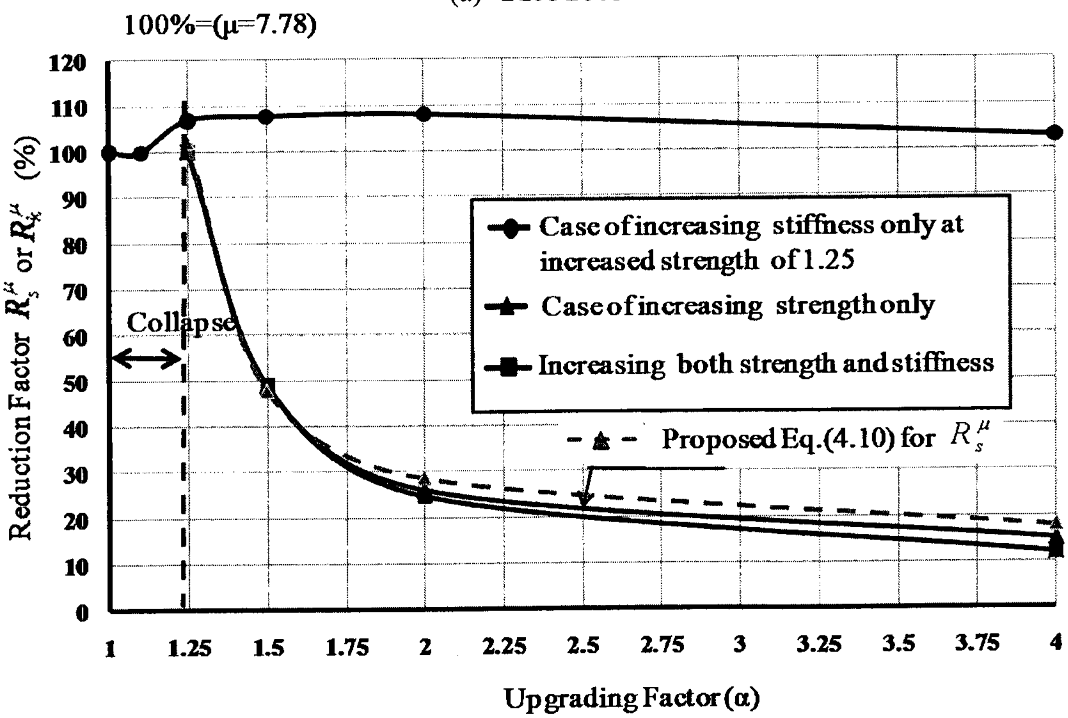


(b) DoD 2005

Figure 4.19. Reduction factors in displacement ductility demand (μ_Δ) for case of removing the *Corner Column (CC)* after increasing strength and/or stiffness only and the proposed equations for R_s^μ & R_k^μ for loading according to: a) GSA 2003;b) DoD 2005

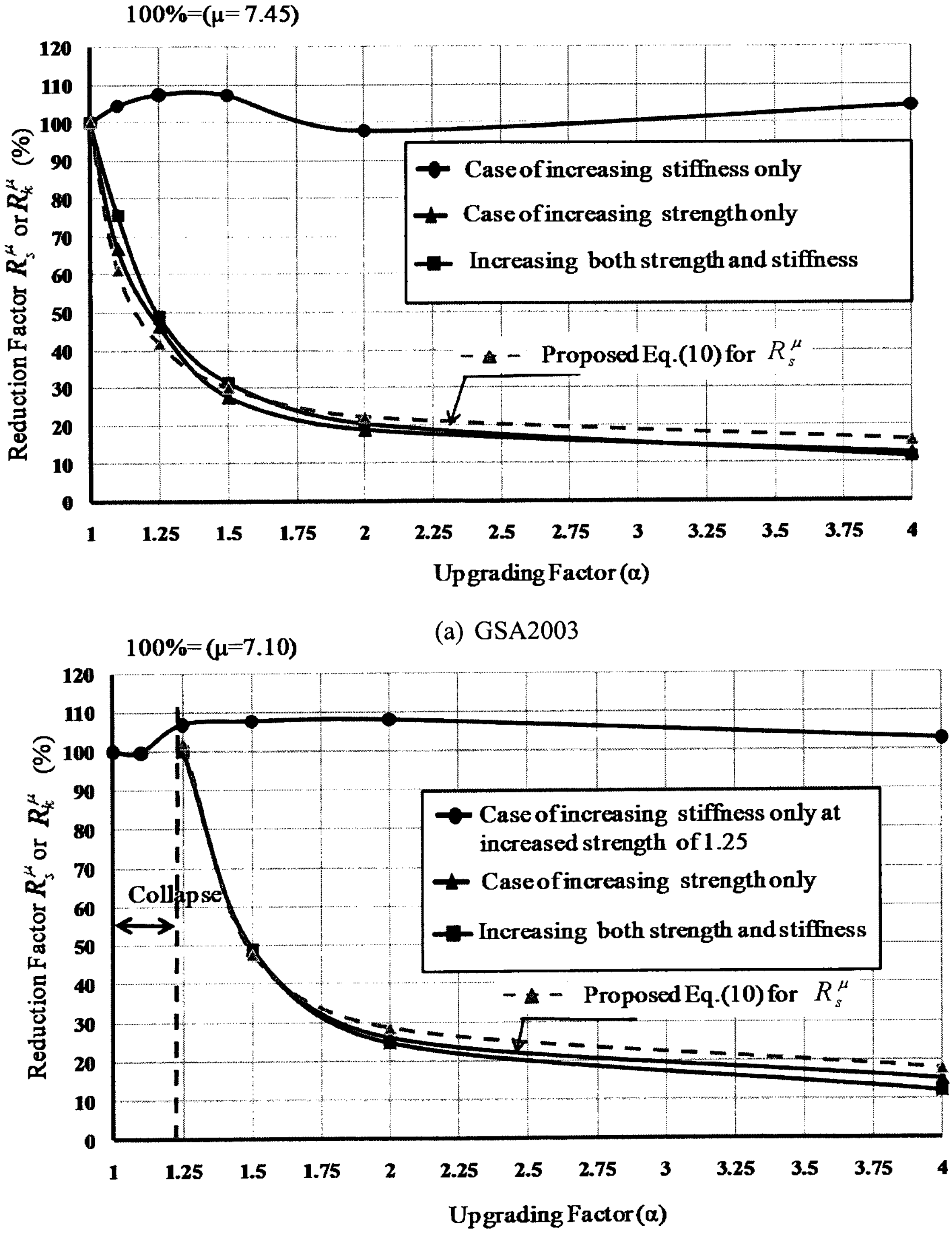


(a) GSA 2003



(b) DoD 2005

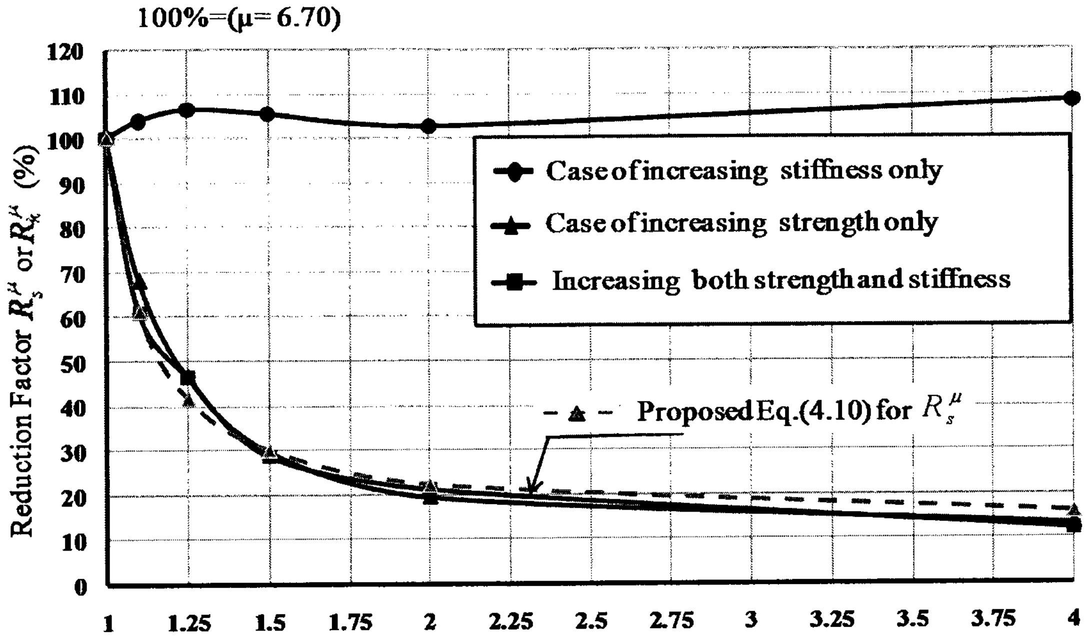
Figure 4.20. Reduction factors in displacement ductility demand (μ_Δ) for case of removing the *Internal Column (IC)* after increasing strength and/or stiffness only and the proposed equations for R_s^{μ} & R_k^{μ} for loading according to: a) GSA 2003;b) DoD 2005.



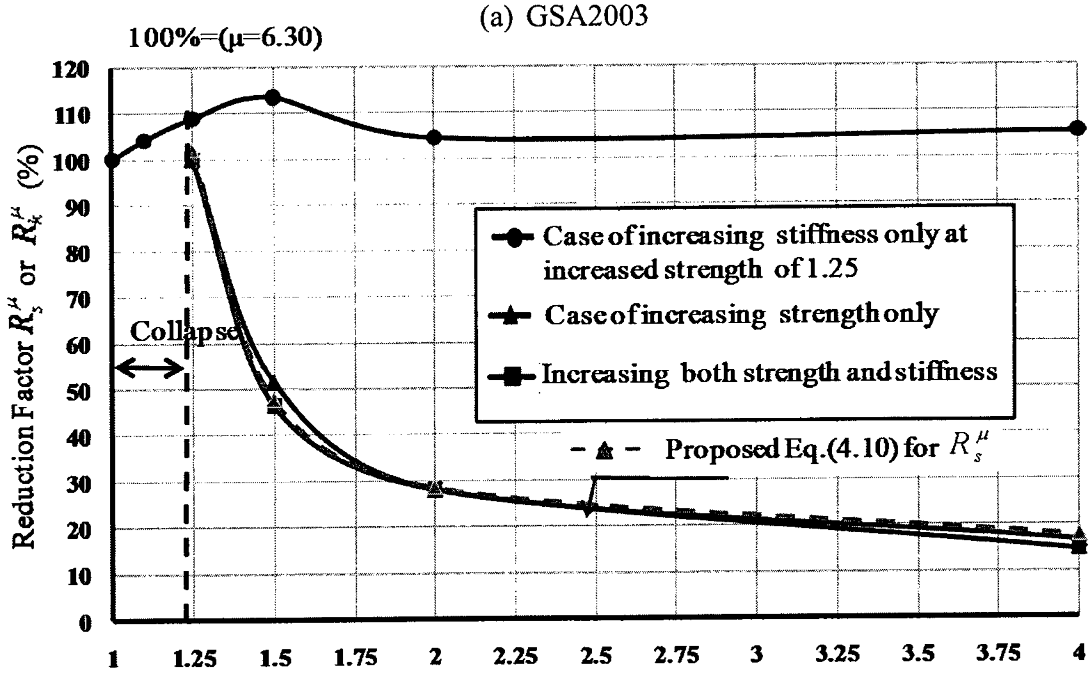
(a) GSA2003

(b) DoD2005

Figure 4.21. Reduction factors in displacement ductility demand (μ_Δ) for case of removing the *First Internal Column (FIC)* after increasing strength and/or stiffness only and the proposed equations for R_s^μ & R_k^μ for loading according to: a) GSA 2003;b) DoD 2005.



(a) GSA2003



(c) DoD 2005

Figure 4.22. Reduction factors in displacement ductility demand (μ_Δ) for case of removing the *Edge Long Column (ELC)* after increasing strength and/or stiffness only and the proposed equations for R_s^μ & R_k^μ for loading according to: a) GSA 2003; b) DoD 2005.

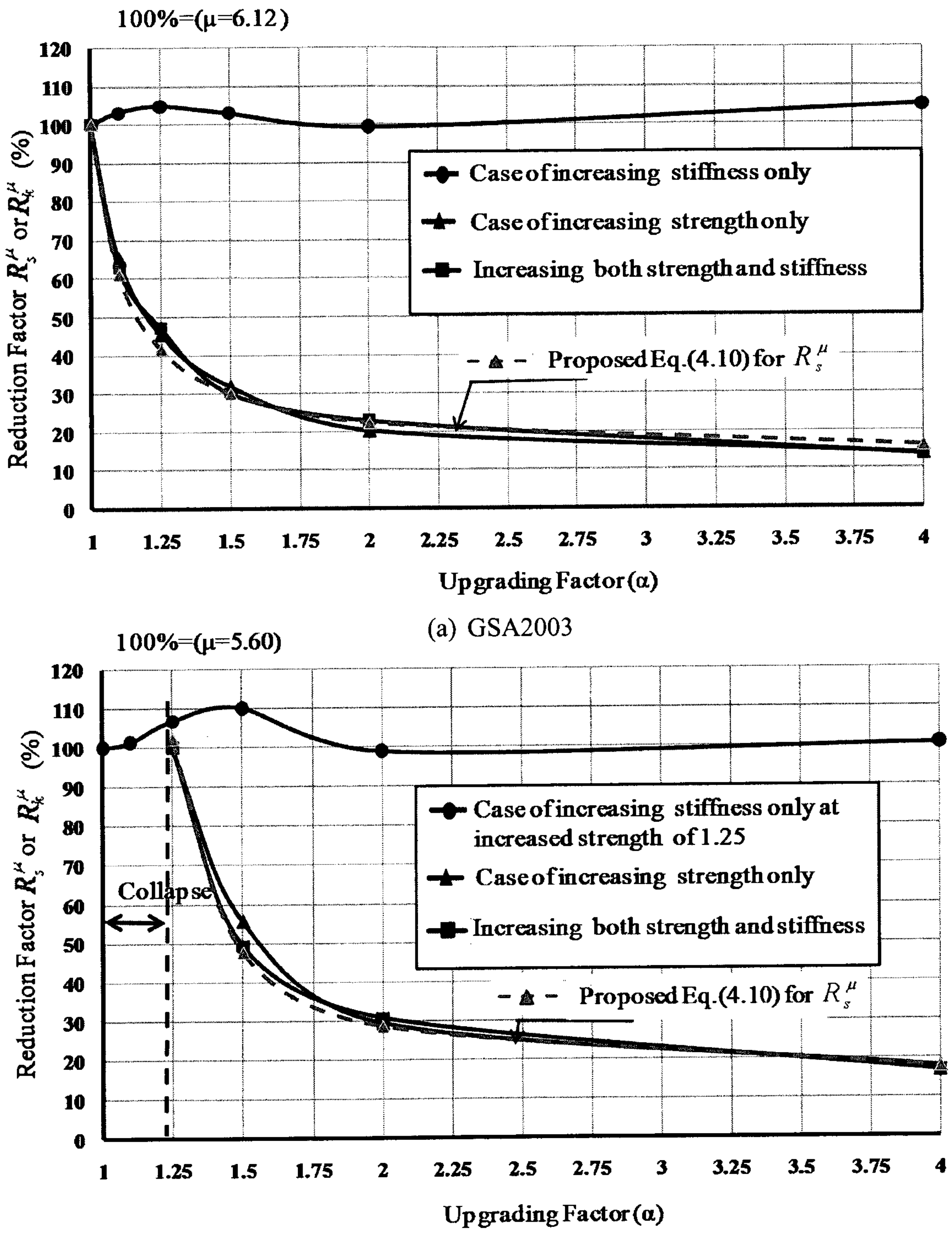
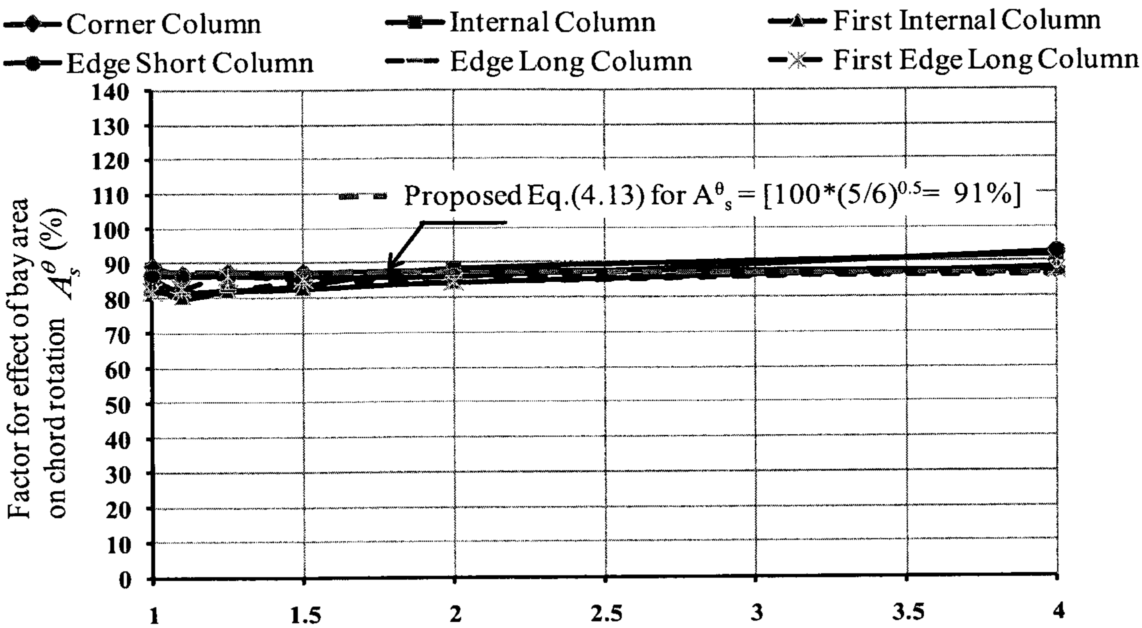
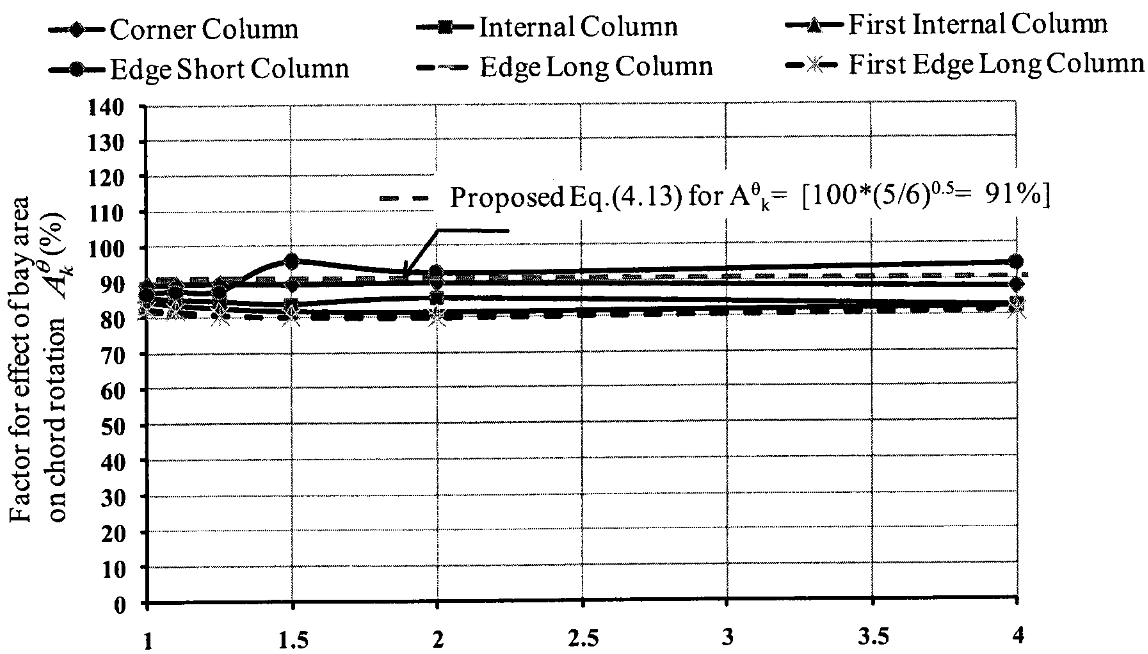


Figure 4.23. Reduction factors in displacement ductility demand (μ_Δ) for case of removing the *First Edge long Column (FELC)* after increasing strength and/or stiffness only and the proposed equations for R_s^μ & R_k^μ for loading according to: a) GSA 2003;b) DoD 2005.

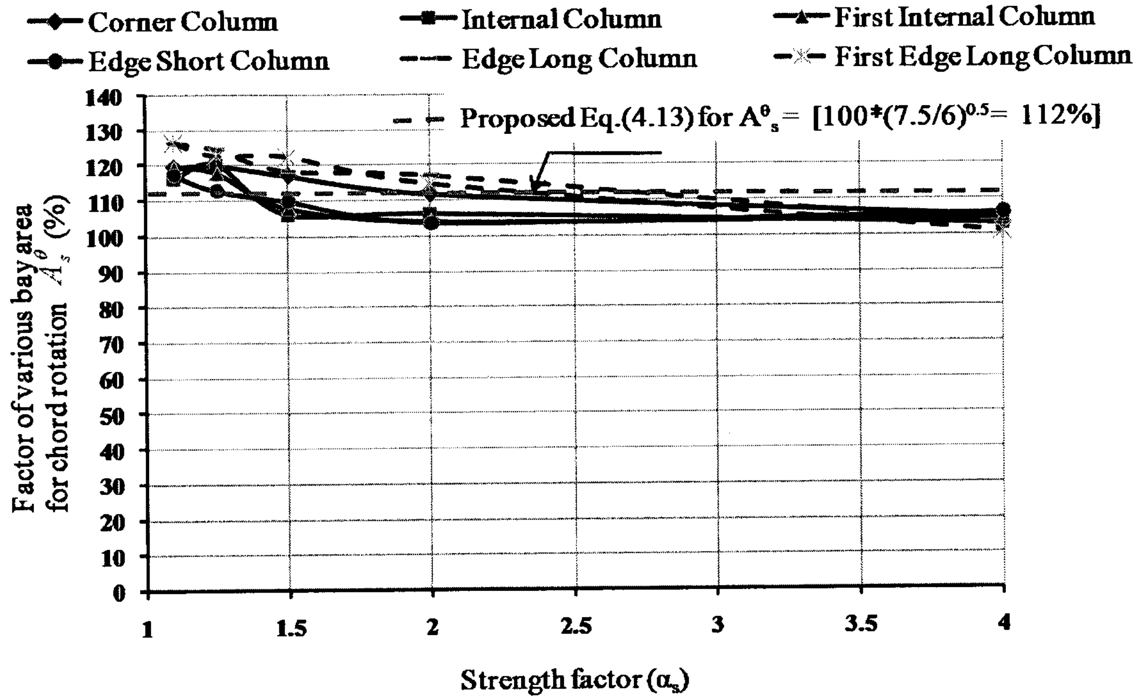


(a) Factor of A_s^{θ}

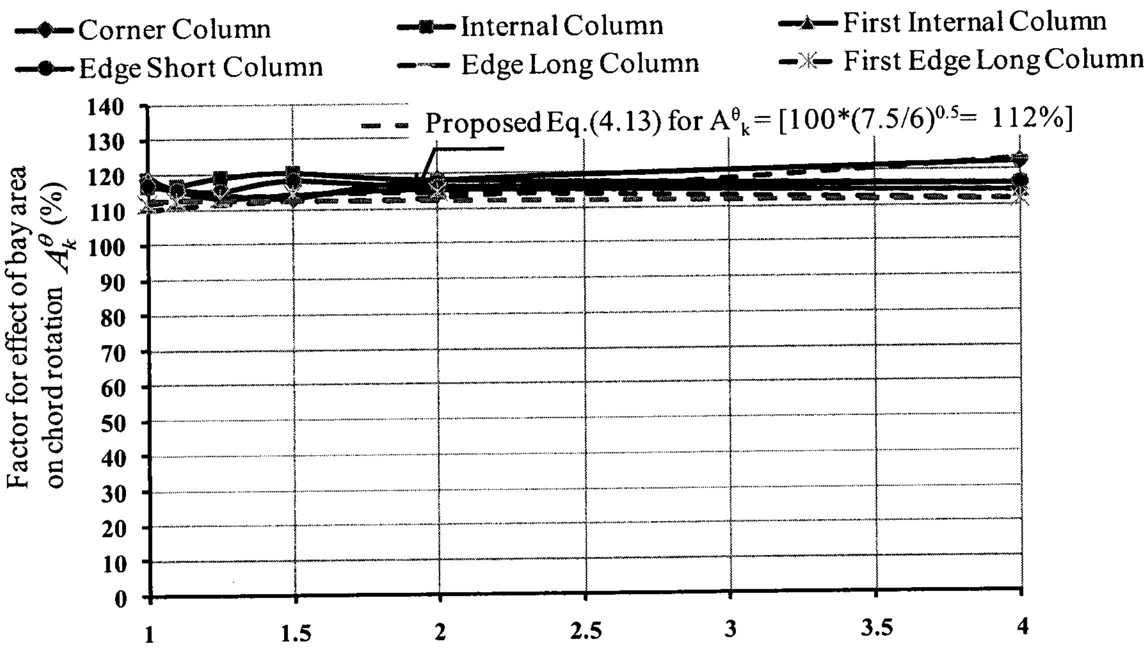


(b) Factor of A_k^{θ}

Figure 4.24. Factors for effect of bay area of building with bay area *5mx5m on chord rotation (θ)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for: a) A_s^{θ} ; b) A_k^{θ} .

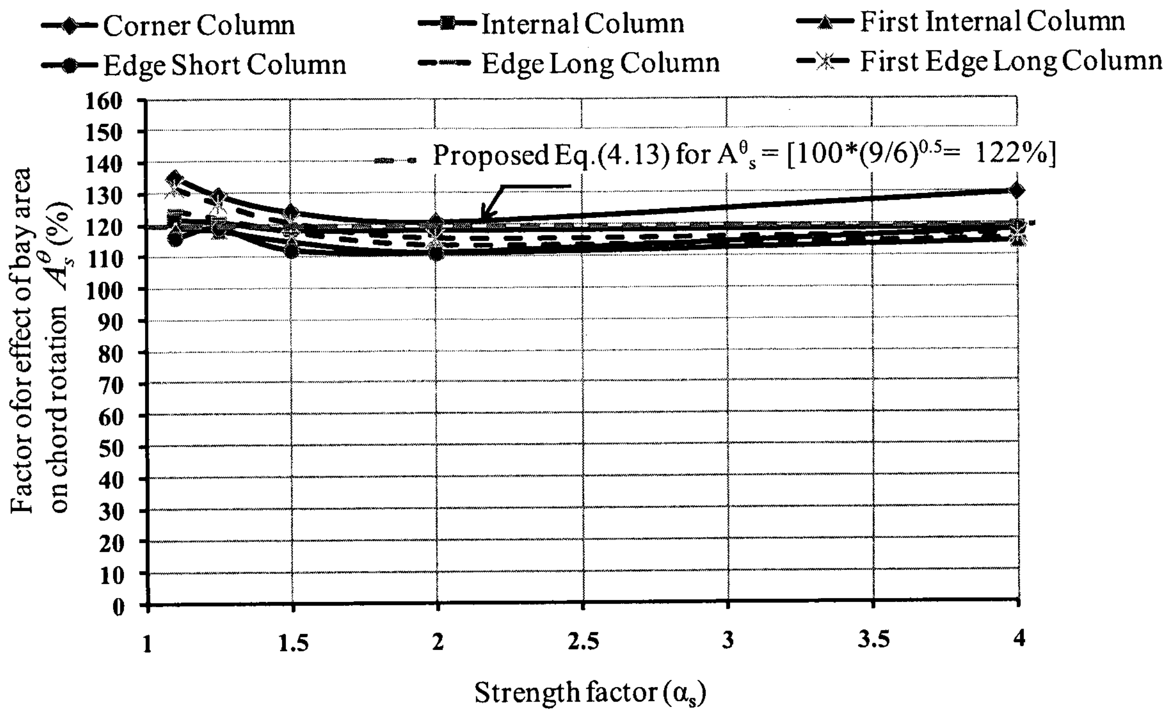


(a) Factor of A_s^θ

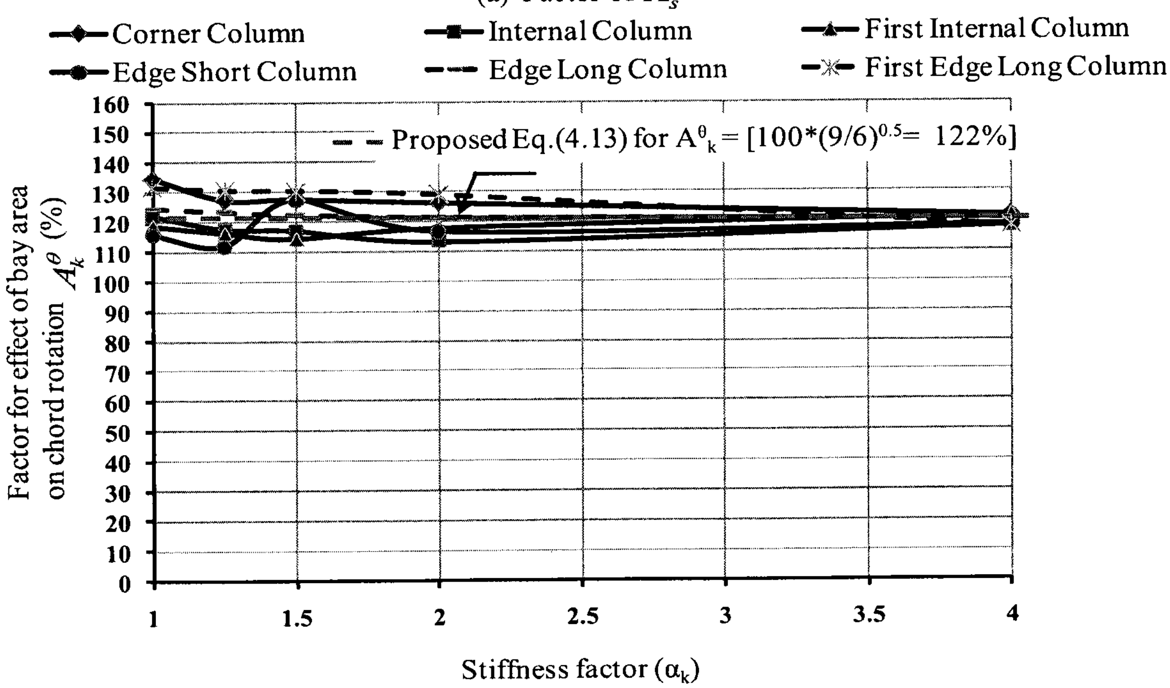


(b) Factor of A_k^θ

Figure 4.25. Factors for effect of bay area of building with bay area *7.5mx7.5m on chord rotation (θ)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for: a) A_s^θ ; b) A_k^θ .

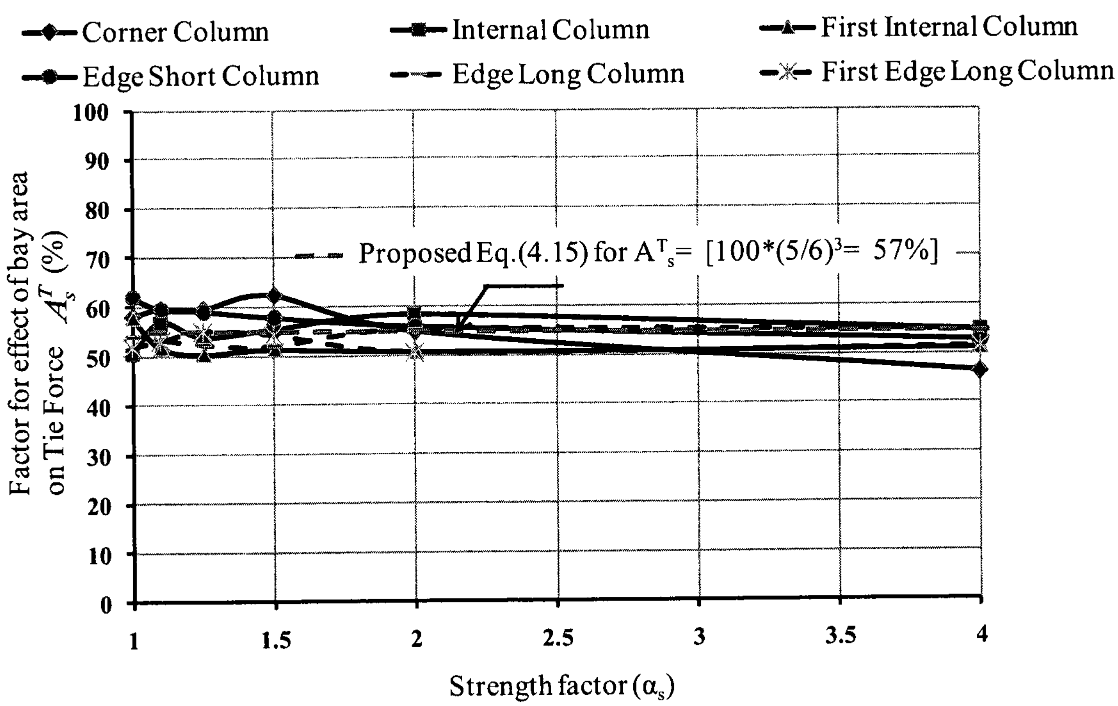


(a) Factor of A^{θ}_{s}

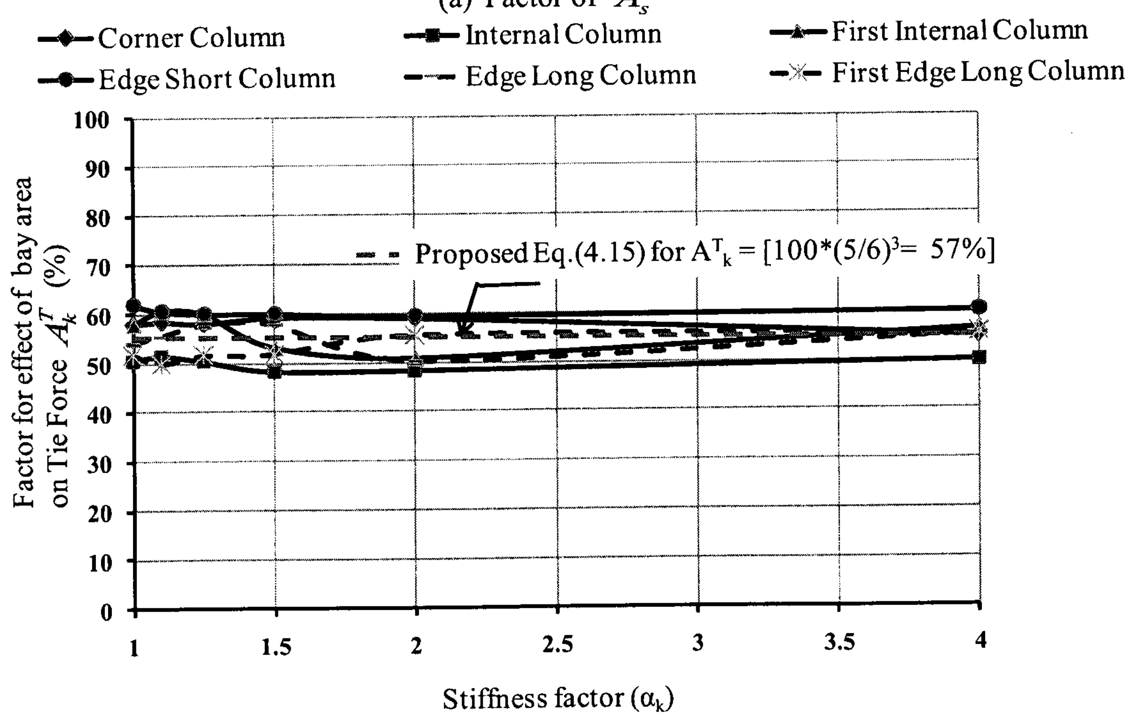


(b) Factor of A^{θ}_{k}

Figure 4.26. Factors for effect of bay area of building with bay area *9mx9m on chord rotation (θ)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for : a) A^{θ}_{s} ; b) A^{θ}_{k} .

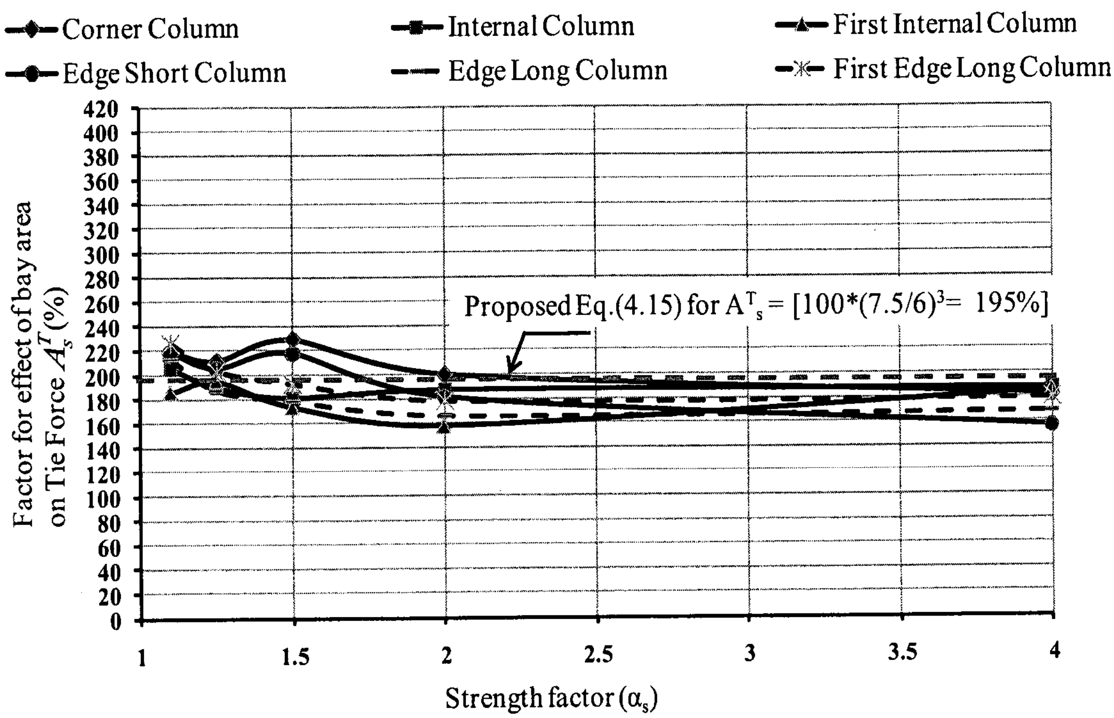


(a) Factor of A_s^T

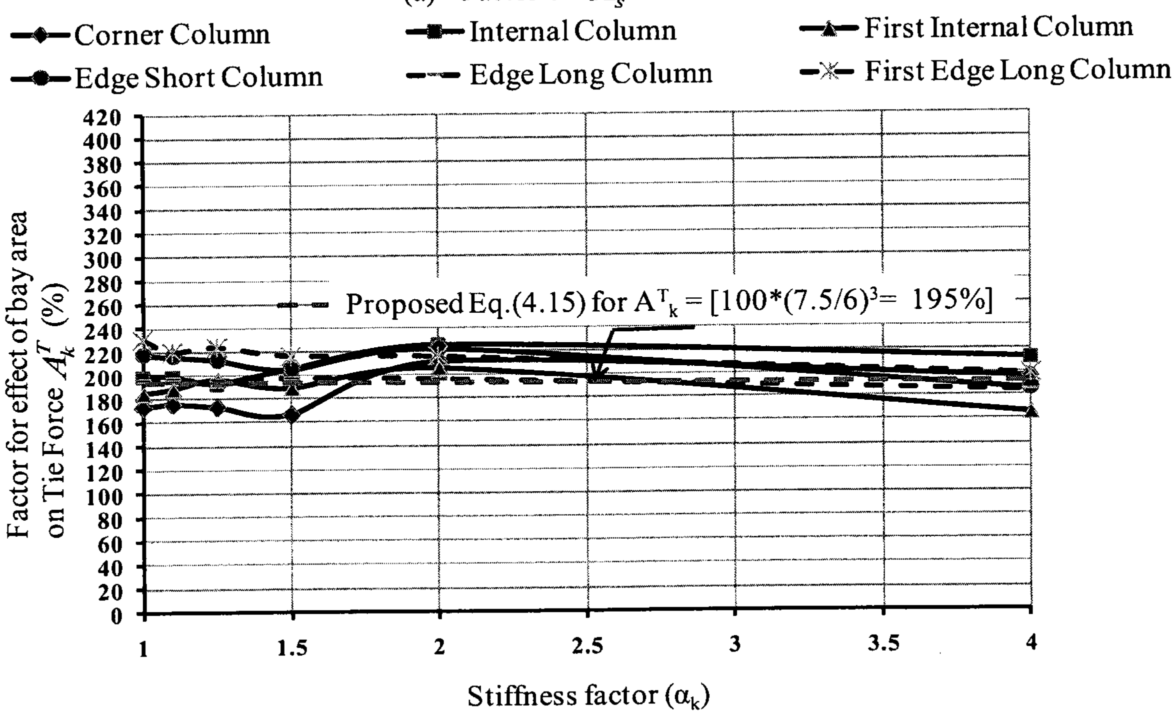


(b) Factors of A_k^T

Figure 4.27. Factors for effect of bay area of building with bay area *5mx5m on Tie force (TF)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for : a) A_s^{θ} ; b) A_k^{θ} .

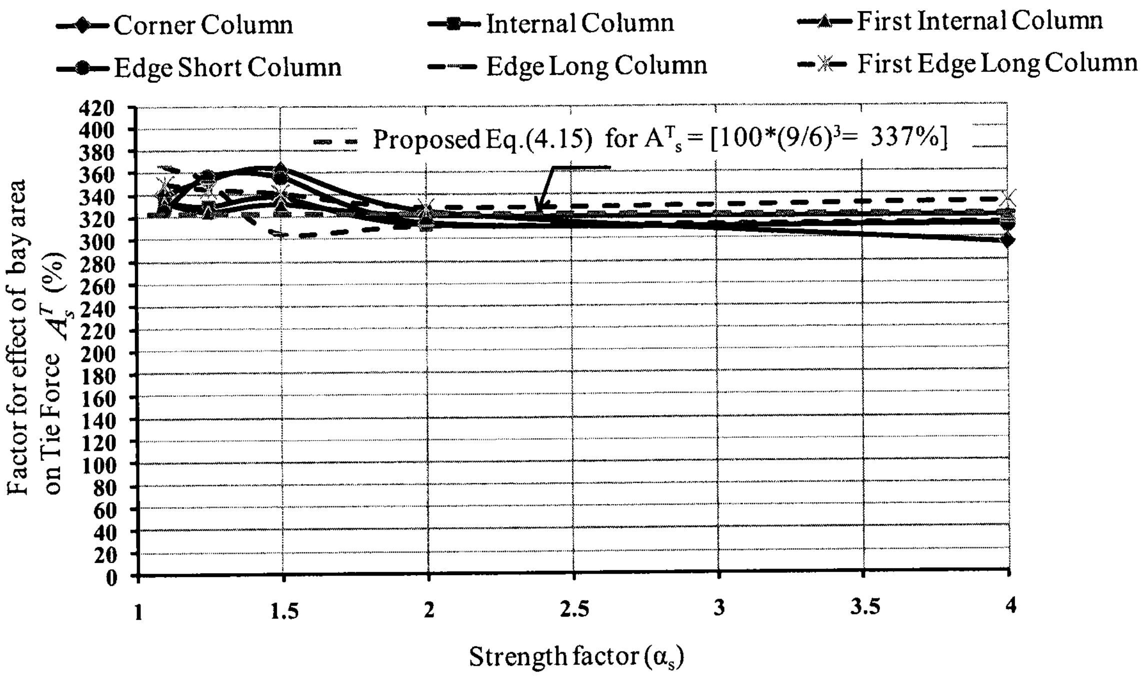


(a) Factor of A_s^T

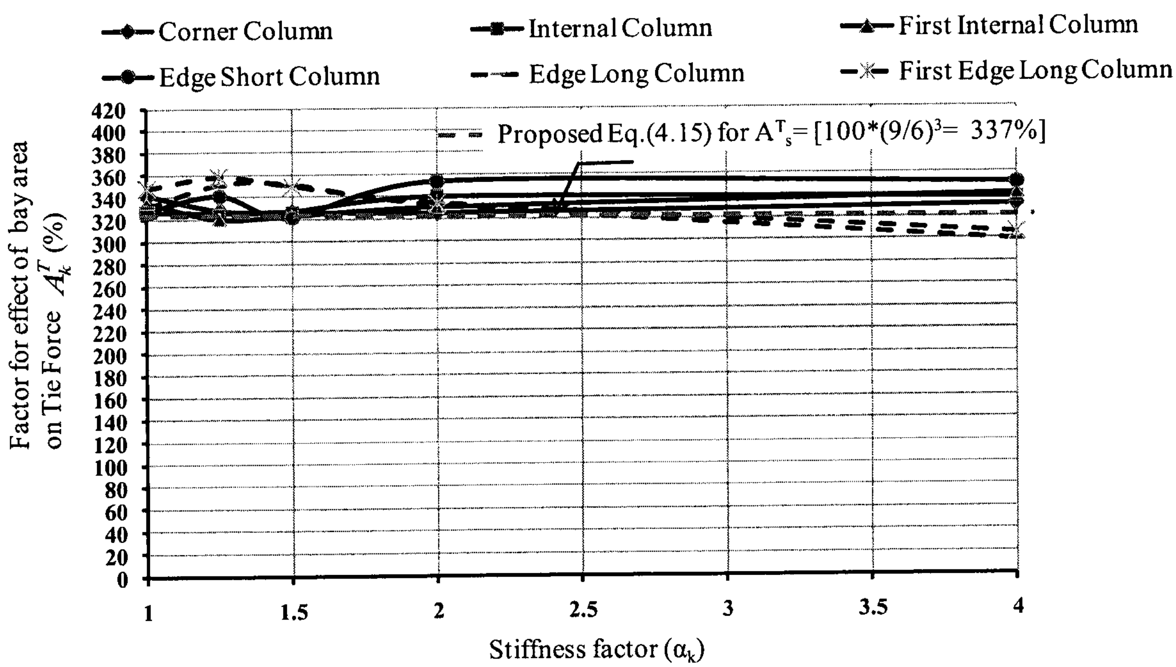


(b) Factor for A_k^T

Figure 4.28. Factors for effect of bay area of building with bay area *7.5mx7.5m on Tie Force (TF)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for : a) A_s^{θ} ; b) A_k^{θ} .

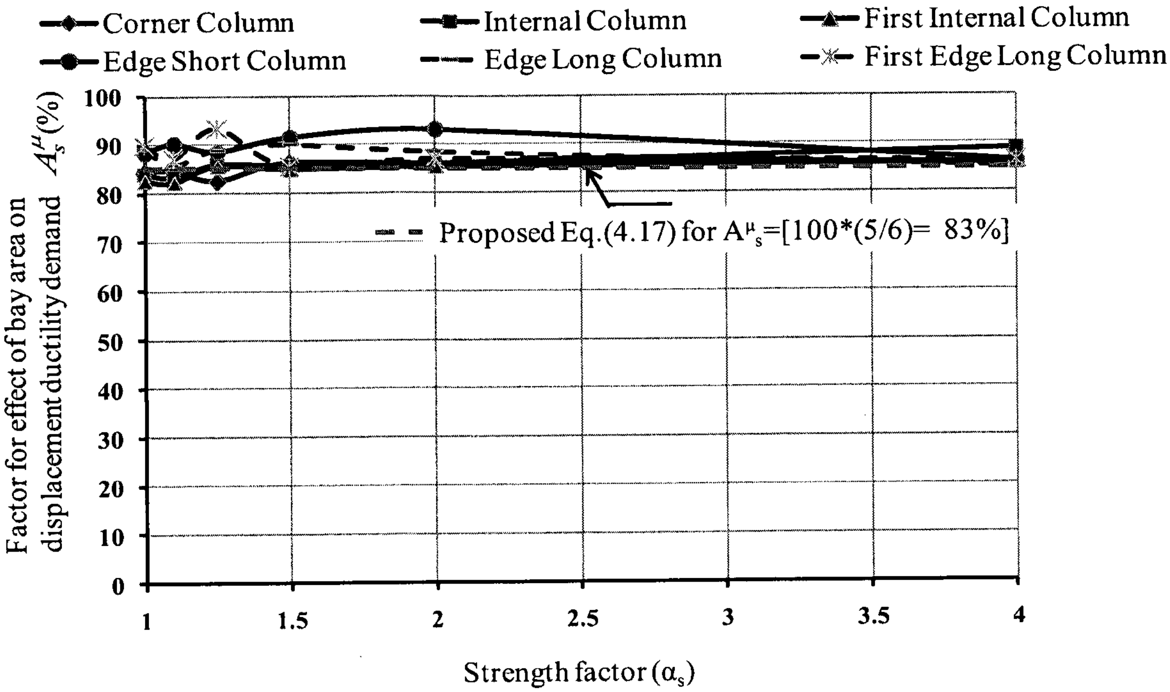


(a) Factor of A_s^T

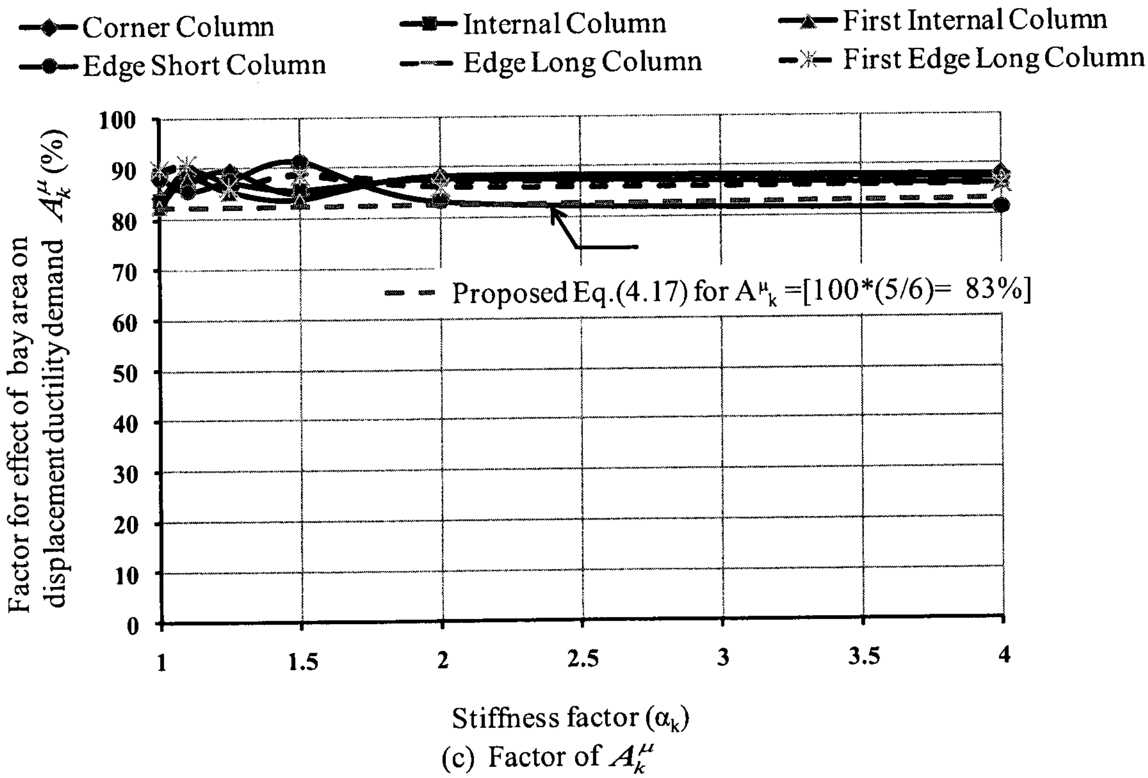


(b) Factor of A_k^T

Figure 4.29. Factors for effect of bay area of building with bay area *9mx9m on Tie Force (TF)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for: a) A_s^θ ; b) A_k^θ .

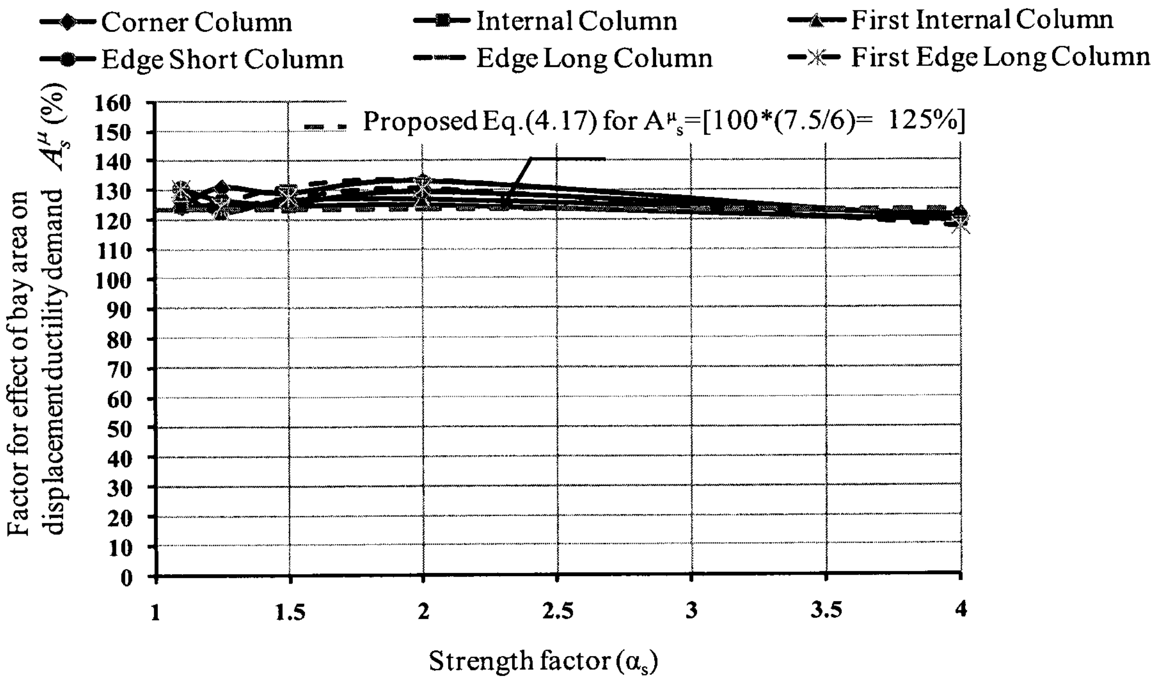


(a) Factor of A_s^{μ}

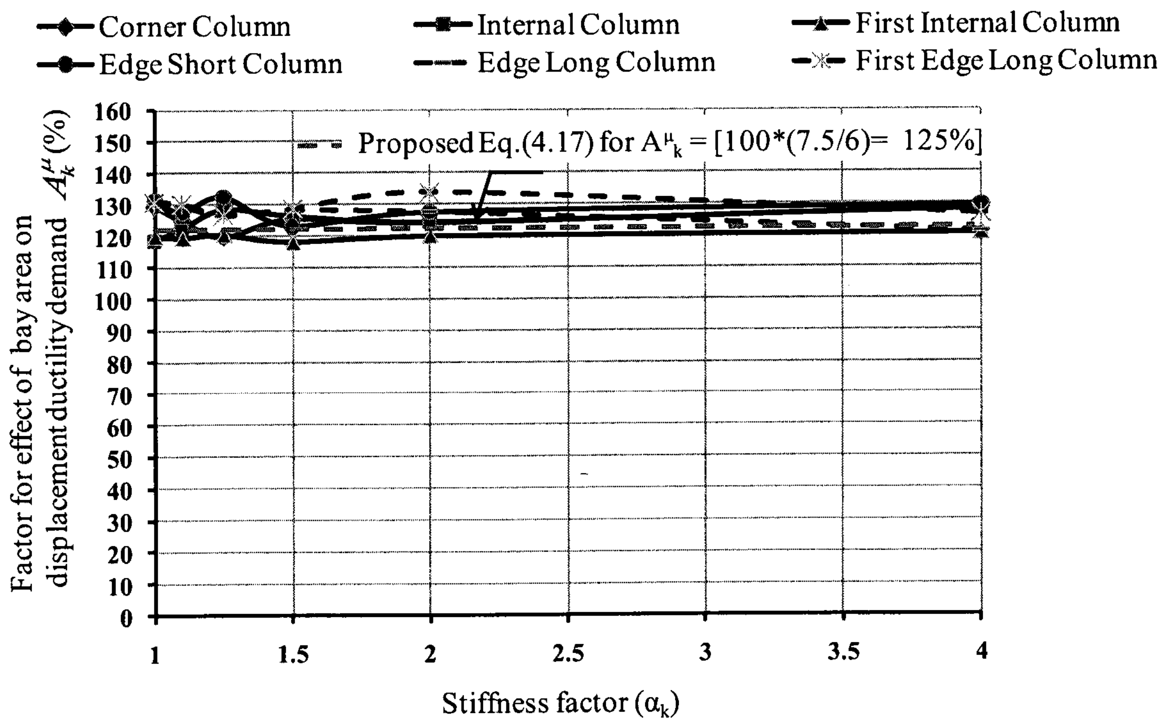


(c) Factor of A_k^{μ}

Figure 4.30. Factors for effect of bay area of building with bay area *5mx5m on displacement ductility demand (μ_Δ)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for : a) A_s^{θ} ; b) A_k^{θ} .

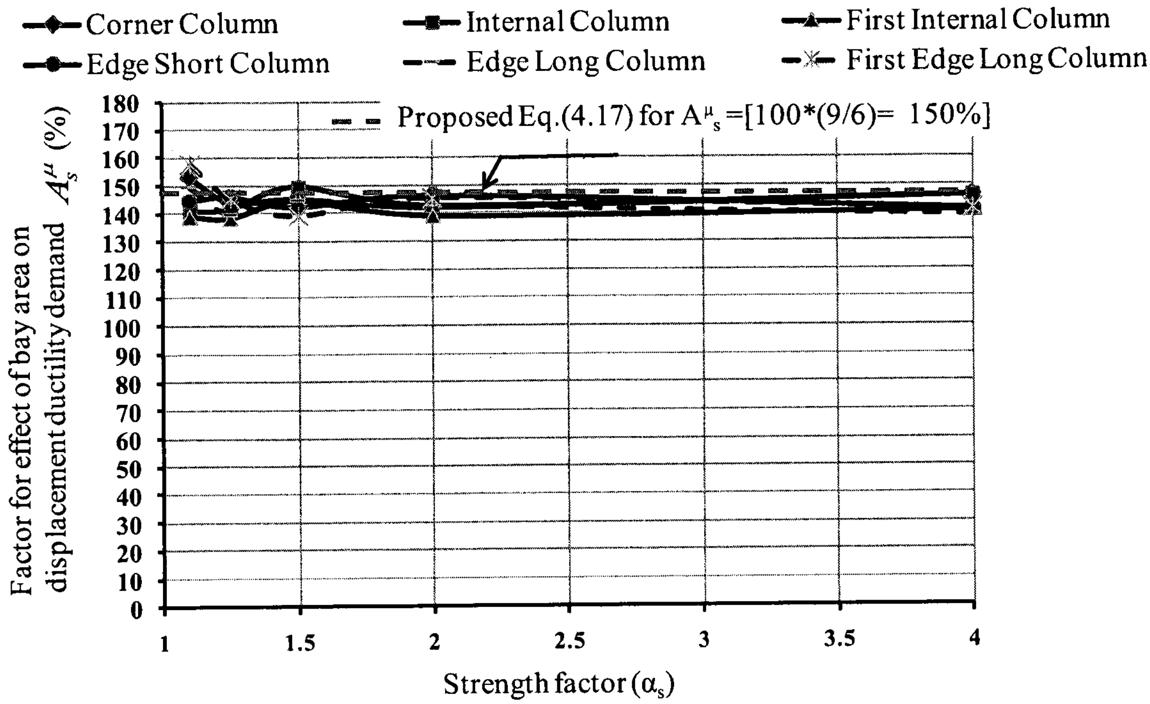


(a) Factor of A_s^{μ}

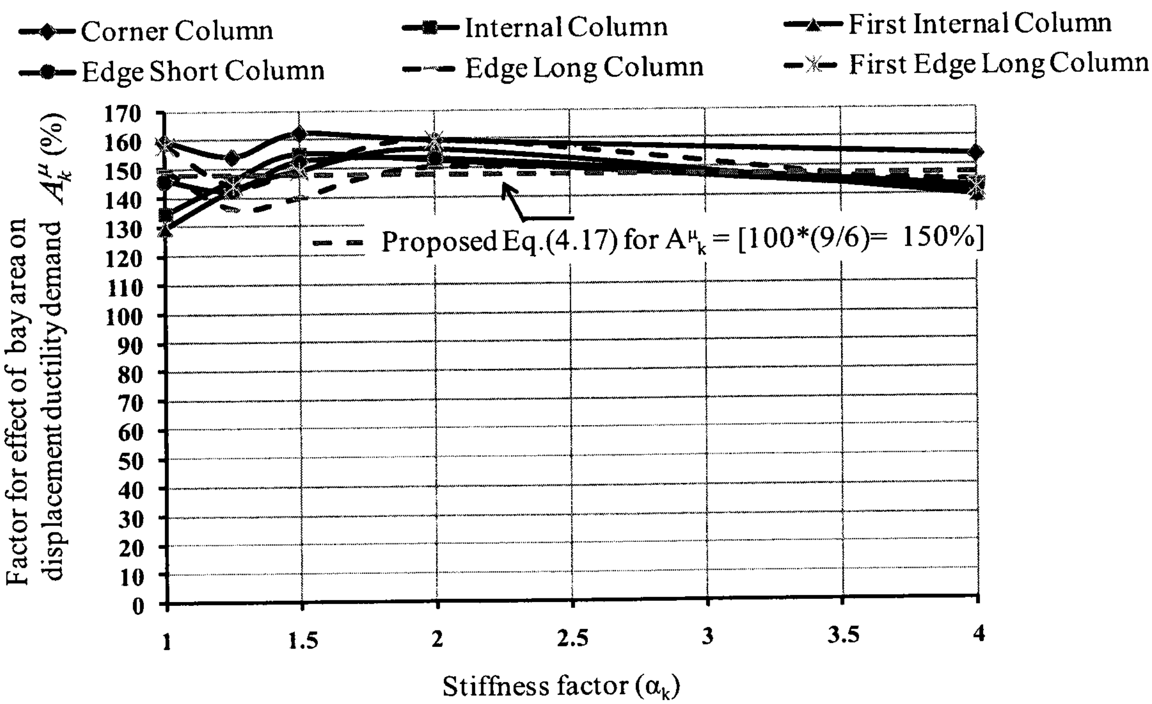


(b) Factor of A_k^{μ}

Figure 4.31. Factors for effect of bay area of building with bay area *7.5mx7.5m on displacement ductility demand (μ_Δ)* for all column loss scenarios as compared to bay span 6m after increasing strength or stiffness only and the proposed equations for: a) A_s^{θ} ; b) A_k^{θ} .



(a) Factor of A_s^{μ}



(b) Factor of A_k^{μ}

Figure 4.32. Factors for effect of bay area of building with bay area *9mx9m on displacement ductility demand (μ_Δ)* for all column loss scenarios as compared to bay span of 6 m after increasing strength or stiffness only and the proposed equations for: a) A_s^{θ} ; b) A_k^{θ} .

Chapter 5

Conclusions and Recommendations

5.1 SUMMARY

In this study, the Alternative Path Method (APM) analysis was conducted for a high-rise steel frame office building consisting of 18 typical floors with 3x6 bays in the plan that are designed under gravity load only. An initial bay span (i.e. in two directions) was taken as 6.0 m, which is designated as the reference model. This span was later varied to 5.0 m, 7.5 m, and 9.0 m in order to evaluate the effect of variation in bay area. Six removed column scenarios (internal, corner, edge long, edge short, first internal and first edge long column) were applied to the ground floor, assuming the ground floor is the most susceptible floor that can be subjected to extreme loading due to explosions or vehicle impact. A 3D nonlinear dynamic analysis was performed using Extreme Loading Structure (ELS), for progressive collapse analysis and tracking the failure of a building if it occurs.

For each removed column, two load combinations were applied on the reference model to represent GSA and DoD loading. Then the beam strength and stiffness were increased by factors 1.1, 1.25, 1.5, 2 and 4, which correspond to increases strength or stiffness by 10%, 25%, 50%, 100% and 300%, respectively.

Series of equations are proposed, and can be used to predict chord rotation, tie force and displacement ductility demand in the beams; and these predictions can help designers to upgrade an existing building to meet a certain level of protection, as stated in UFC 4-023-03. According to that document, for a Very Low Level of Protection (VLLOP) and a Low Level of Protection (LLOP), the maximum rotation can be taken as 12 degrees, while for a High Level of Protection (HLOP) and a Very High Level of Protection (VHLOP), it can be taken as 6 degrees.

The effect of variations in bay area was also investigated by studying different models with varying bay spans of 5.0 m, 7.5 m and 9.0 m subjected to the GSA load combination. A model with a bay span of 6.0 m was set as the reference model to monitor comparative variations in the other models for the three parameters.

5.2 CONCLUSIONS

A high rise steel frame was studied using the Alternative Path Method to predict three parameter indicators: chord rotation, tie force and displacement ductility demand after upgrading the beams. The two load combinations stipulated in the GSA and DoD guidelines were applied. Another objective was to predict the effect of variation of bay area on these parameter indicators. The results of the study can be summarized as follows:

[1] Upgrading the beams by increasing their strength only is more effective than increasing their stiffness only in enhancing the three performance indicators; chord rotation, tie force, and displacement ductility demand.

[2] The reduction factor in upgrading both strength and stiffness of the beams is found to be equal to the numerical product of the reduction factor arising from both upgrading strategies of increasing strength only and increasing stiffness only of the beams.

[3] In case of any building collapse due to column removal scenario under certain loading conditions, increasing the stiffness of the beams only will not safeguard the building from collapse, which makes increasing the strength of the beams a necessity.

[4] For the studied buildings, all column removal scenarios with building loaded according to DoD resulted in a collapse of the building, which was not the case when loaded according to GSA criteria this highlights the importance of further research for clear identification of the combination of loads that can better represent gravity loading in Alternative Path Method.

[5] The level of tie force exerted in beams of the existing building calculated from nonlinear dynamic analysis using ELS software is more than three times of the limits stated by DoD guideline for all studied buildings, which confirms similar findings by other researchers. This highlights a need for more research to identify appropriate estimations for tie forces.

[6] For all studied buildings, chord rotation, tie force and displacement ductility demand in case of loss of Internal and Edge Long Column scenario are more than those arising from case of First Internal and First Edge Long Column removal scenario, respectively for all studied buildings. This could be attributed to the orientation of the columns adjacent to removed one; the higher the number of adjacent columns oriented along their strong axes, the lower the chord rotation, Tie Force and displacement ductility demand.

[7] Tie force in the scenario of removing the Edge Long Column is higher than that exerted in the scenario of Edge Short Column removal for all the studied buildings due to the higher number of bays in the edge long direction.

[8] Upgrading the beams by increasing their stiffness only has no significant effect on the displacement ductility demand. Consequently, upgrading the beams by increasing its strength only will results in similar displacement ductility response compared to case of increasing both strength and stiffness.

[9] Effect of varying the bay span on chord rotation was found to be proportion to square root of the ratio between spans.

[10] Effect of varying the bay span on tie force was found to be proportion to (ratio between spans)^3, while in DoD guideline it is proportion to the area serviced (ratio between spans)^2.

[11] Effect of varying the bay span on displacement ductility demand is approximately direct proportion to the ratio between span of the bay.

From the above conclusions it can be seen that the choice of the most suitable rehabilitation scheme to safeguard against progressive collapse should consider the loading criteria, the targeted level of safety, and the desired performance parameter needed to be enhanced. It is important to clarify that the results drawn are for the specific studied case. More models for different structure configurations and capacities should be considered and more analysis including cost analysis is needed for the conclusions to be generalized.

5.3 RECOMMENDATION FOR FUTURE RESEARCH

During this research, the studied models were 18 typical floors with 6 x 3 bays. Further analyses are required to generalize the derived equations for expecting chord rotation and tie forces in the beams. These other models will have different story levels and different numbers of bays. Two additional important topics that need to be studied are:

1) The response of steel frame buildings when two or more columns are lost in high hazard events.

2) Mitigation of steel frame buildings using innovative structural systems such as mega truss at the top of the building or a bracing system.

REFERENCES

1. ASCE, 2005, "Minimum design loads for buildings and other structures", ASCE/SEI 7-2005, New York.

2. ATC-40, 2000, "Seismic evaluation and retrofit of concrete building", Applied Technology Council, Vol.1.

3. British Standards Institution, 2001, "BS 5950: Structural use of steelwork in buildings, Part 1: Code of practice for design _ Rolled and welded sections", London, England, 2001.

4. Canadian Institute of steel construction (CISC-95), 1995, *Handbook of steel construction*.

5. Crawford J.E., 2002, "Retrofit Measures to Mitigate Progressive Collapse", NIST/NIBS, Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, IL.

6. Chopra A.K., 2002, "Dynamics of structures: Theory and Applications to Earthquake engineering", California, USA.

7. Ellingwood BR., Smilovitz R., Dusenberry DO., Duthinh D., Lew HS., and Carino NJ., 2007, "Best practices for reducing the potential for progressive collapse in buildings, National Institute of Standards and Technology (NIST)", NITIR 7396, Washington D.C.

8. ELS®, 2006, "Extreme loading® for structures technical manual", Applied science International, Vol.2.2.

9. EuroCode 1, 2005, "Actions on structures. Part 1.7: General Actions - Accidental actions", European Committee for Standardization.

10. Fu F., 2009, "Progressive collapse analysis of high-rise building with 3-D finite element modeling method", *Journal of Constructional Steel Research*, 65(6):1269-1278.

11. Ghobarah A., El-Attar M., and Aly N.M., 2000, "Evaluation of retrofit strategies for reinforced concrete columns", *Journal Engineering Structures*, P.490–501.

12. Grierson D.E., Safi M., Xu L., and Liu Y., 2005, "Simplified methods for progressive collapse analysis of buildings", Conference paper, ASCE.

13. Griffin J.W., 2008, "Experimental and Analytical Investigation of Progressive Collapse through Demolition Scenarios and Computer Modeling", Masters of science thesis, North Carolina State University, Raleigh, North Carolina,.

14. GSA, 2003, "Progressive collapse analysis and design guidelines for new federal office buildings and major modernizations projects", The U.S. General Services Administration.

15. Hayes Jr. R., Woodson S. C., Pekelnicky R. G., Poland C. D., Corley W. G., and Sozen M., 2005, "Can Strengthening for Earthquake Improve Blast and Progressive Collapse Resistance?", *Journal of Structural Engineering*, Vol. 131, No. 8, P 1157-1177.

16. Houghton D.L., 1998, "The SidePlate Moment Connection System: A Design Breakthrough Eliminating Recognised Vulnerabilities in Steel Moment-Resisting Frame Connections", *Journal of Constructional Steel Research,* Vol.46, No. 1-3, P 260-261.

17. ISC, 2001, "ISC Security Criteria for New Federal Office Buildings and Major Modernization Projects", the interagency Security Committee, Washington, DC.

18. Isobe D., Toi Y., 2000, "Analysis of structurally discontinuous reinforced concrete building frames using the ASI technique", *Computers & Structures*, Vol. 76, No. 4, P 471-81.

19. Izzuddin B.A., Vlassis A.G., Elghazouli A.Y., Nethercot D.A., 2008, "Progressive collapse of multi-storey buildings due to sudden column loss- Part I: Simplified assessment framework", *Engineering Structures*, Vol. 30, No. 5, P 1308-1318.

20. Kaewkulchai G., 2003, "Dynamic progressive collapse of frame structures", PHD dissertation, The University of Texas at Austin, Austin, Texas.

21. Kappos A.J., 2002, "Dynamic loading and design of structures", New York, USA.

22. Karns J.E., Houghton D.L., Hall B.E., Kim J., and Lee K., 2006, "Blast testing of steel frame assemblies to assess the implications of connection behavior on progressive collapse", Proceedings of the Structures Congress and Exposition, Structures Congress 2006: Structural Engineering and Public Safety- Proceedings of the 2006 Structures Congress, P.32.

23. Kim J. and Kim T., 2009, "Assessment of progressive collapse-resisting capacity of steel moment frames", *Journal of Constructional Steel Research*, Vol. 65, No.1, P 169-179.

24. Kim J. and Park J., 2008, "Design of steel moment frames considering progressive collapse", *Steel and Composite Structures*, Vol. 8, No. 1, P.85-98.

25. Kim J. and AN D., 2008, "Evaluation of Progressive Collapse Potential of Steel Moment Frames Considering Catenary Action", *the structural design of tall and special buildings*, Vol. 18, No. 4, P 455 - 465 .

26. Khandelwal K. and El-Tawil S., May 2007, "Collapse Behavior of Steel Special Moment Resisting Frame Connections", *Journal of Structural Engineering*, Vol. 133, No. 5, P 646-655.

27. Khandelwal K., El-Tawil S., Kunnath S.K., and Lew H.S., 2008, "Macromodel-Based simulation of progressive collapse: steel frame structure", *Journal of structural engineering*, Vol. 134, No.7, P 1070-1078.

28. Khandelwal K., El-Tawil S., and Sadek F., 2009, "Progressive collapse analysis of seismically designed steel braced frames", *Journal of Constructional Steel Research*, Vol. 65, No. 3, P 699-708.

29. Lee C., Kim S., Han K., and Lee K., 2008, "Simplified nonlinear progressive collapse analysis of welded steel moment frames", *Journal of constructional steel research*, Vol. 65, No. 5, p 1130-7.

30. Liu R., Davison B., and Tyas A., 2005, "A study of progressive collapse in multi-storey steel frames", Proceedings of the Structures Congress and Exposition, Metropolis and Beyond - Proceedings of the 2005 Structures Congress and the 2005 Forensic Engineering Symposium, NY, ASCE, P.2211-2219.

31. Lu X.Z., Li Y., Ye L., Ma Y., and Liang Y., Oct. 2008, "Study on the design methods to resist progressive collapse for building structures", Proceedings of the Tenth International Symposium on Structural Engineering for Young Experts, Changsha:478-483.

32. Marchand A.K. and Alfawakhiri F., 2005, "Facts for steel buildings: blast and progressive collapse", AISC, No.2.

33. Marjanishvili S. and Agnew E., 2006, "Comparison of various procedures for progressive collapse analysis", *Journal of performance of constructed facilities*, ASCE, Vol. 20, No. 4, p 365-374.

34. Marjanishvili S.M., May 2004, "Progressive Analysis Procedure for Progressive Collapse", *Journal of Performance of Constructed Facilities*, Vol. 18, No. 2, p 79-85.

35. Meguro K. and Tagel-Din H., 2002, "Applied Element Method for Structural Analysis: Theory and Application for Linear Materials", Structural Eng./Earthquake Eng., *International Journal of the Japan Society of Civil Engineers (JSCE)*, Vol. 17, No.1, P 25-35 .

36. Powell G., 2005, "Progressive Collapse: Case Studies Using Nonlinear Analysis", Proceedings of the Structures Congress and Exposition, Metropolis and Beyond - Proceedings of the 2005 Structures Congress and the 2005 Forensic Engineering Symposium, NY, ASCE, P 2185-2198.

37. Ruth P., Marchand K.A., and Williamson E.B., 2006, "Static equivalency in progressive collapse alternative path analysis: reducing conservatism while retaining

structure integrity", *Journal of performance of Constructed facilities*, ASCE, Vol. 20, No. 4, P 349-364.

38. Saad A., Said A., and Tian Y., 2008, "Overview of progressive collapse analysis and retrofit techniques", International Committee, Los Angeles Section, 5th International Engineering and Construction Conference (IECC'5).

39. Song, B.I. and Sezen H., 2009, "Evaluation of an existing steel frame building against progressive collapse", conference proceedings of 2009 Structures Congress, Austin, Texas, ASCE. P 1-8.

40. Tagel-Din H and Meguro K, 2002, "AEM used for large displacement structure analysis", *Journal of Natural Disaster Science*, Vol.24, No.2, P 65-82.

41. TM 5-1300, 1990, "Structures to resist the effect of the accidental explosion", Department of army, the navy, and the air force.

42. Toi, Y. and Isobe, D., 1993, "Adaptively Shifted Integration Technique for Finite Element Collapse Analysis of Framed Structures", *International Journal for Numerical Methods in Engineering*, Vol. 36, No.14, P 2323-2339.

43. Toi Y., Isobe D., 1996, "Finite element analysis of quasi-static and dynamic collapse behaviors of framed structures by the adaptively shifted integration technique", *Computers and Structures*, Vol. 58, No. 5, P 947-955.

44. Unified Facilities Criteria (UFC)-DoD, 2005, "Design of buildings to resist progressive collapse", Department of Defense.

45. Vision 2000, 1995, "Performance Based Seismic Engineering of buildings", Structural Engineers Association of California, Vol.1.

46. Wibowo H. and Lau D.T., 2009, "Seismic Progressive Collapse: Qualitative Point of View", *Civil Engineering Dimension*, Vol. 11, No. 1, P 8-14.

47. Yellin M., 2007, "Progressive collapse in steel moment frame structures", Master thesis, Alberta university, Canada.

48. Zajac I.J.E., 2007, "Nonlinear Analysis of Reinforced Concrete Frames Subjected to Abnormal Loads", University of Waterloo, Master thesis, Waterloo, Ontario, Canada.