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Finite Element Analysis of Progressive Collapse Resistance of Reinforced Concrete Framed Multi-Storey Building Subjected To Extreme Loadings

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***Abstract*— This study investigates the progressive collapse of Reinforced Concrete framed structure due to sudden impact or resulting from blast such as terrorist attack, plane crash, vehicle collision and other abnormal loading. Nonlinear finite element analysis (NLFEA) was used to investigate the deformation of frame assembly subject to edge column removal scenario. Prior to implementation of the NLFEA, a modelling scheme and its associated material parameters were developed and validated using published experimental data. The validated model was further employed to investigate the load transfer mechanisms and explore parameters influencing the robustness of the frame system subject to edge column removal. Measures to increase robustness of the frame such as; inclusion of increase in material strength were proposed.**

***Index Terms*— Progressive collapse, Disproportionate Collapse, Total collapse, Extreme loadings, Edge column, Tie beam, Reinforced concrete frame..**

I. Introduction

In recent times, multi-storey and other type of buildings may be subjected to accidental or extreme loadings caused by natural or man-made catastrophe such as hurricanes, tsunamis, earthquakes, explosions, terrorist attack, vehicle impacts, fires, plane crash, etc. Buildings are further aggregated by terrorists' attacks. Magnificent building has become a symbolic target by terrorists in order to cause a news worthy event. These events usually lead to local damage to the structural elements which can trigger or eventuate to total collapse or progressive collapse of building. Progressive collapse is a phenomenon where local failure of a vital component of a structure leads to collapse of adjacent members of the structure which eventually results to total or disproportionate collapse. This phenomenon of progressive collapse was first brought to engineer's attention after the collapse of a 22 storey building in Roma Point London (UK) IN 1968 as a result of gas explosion. Other examples of progressive collapses that have been published or spotlighted by the press include the A.P Murrah federal building (Oklahoma, 1995), the Achimota Melcoms Shipping centre (Acra, 2012), and a lot more.

A new dimension of progressive collapse occurred at the world trade center, New York, due to ferocious attack by terrorist (New York, 2001), In Nigeria, terrorists attack UN building in Abuja in the year 2010 which led to the partial collapse of some structural elements, and few casualties.

This phenomenon in recent times, had also prompted its occurrence at different places in Nigeria, the Lekki building collapse in Lagos on March 8, 2016., the Bank of Industry building (Lagos, 2006); where the 9th floor of the 21 storey building collapsed after the building had experienced fire outbreak in two of the floors.

Based on these events, Engineers now developed several mitigation measures against progressive collapse of building. The basis of this approach is the need for robust buildings that can sustain local damage without suffering disproportionate collapse. One of main challenges associated with this new design approach is that it is difficult to predict the probability of occurrence and the magnitude of progressive collapse. A new mitigation approach or design concept known as Robustness which is defined as the insensitivity of a structure to local damage (Alogla et al. 2016).

.(Xu et al. 2011), in an investigation on progressive collapse observed that, a building structure local damage usually occurs when one or more vertical load-bearing component's (columns, walls) failed, leading to chain of failures that ends in the total collapse of the whole building or larger portion of it.

One of the measures to avoid progressive collapse is to create an Alternative Load Path (ALPM) for loads supported by damaged load bearing element to be transferred to neighboring component. If an adjacent load path is not provided the risk of progressive collapse remains high.

In a situation where framed building is used, five resistive measures to help provide alternative load path, thereby; minimizing the risk of progressive collapse are;

I. Excessive bending of the beam where the column has failed (the beam has to be over dimensioned)
II. Catenary/membrane behavior of beams/slabs, bridging the damaged column by means of large rotation and displacements.
III. Contribution of non-structural elements such as external walls and partitions.
IV. Vierendeel behavior of the frame over the failed column
V. Arch effect of the beam where the columns has failed,

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efficient when horizontal displacement of the neighboring column is small.

The catenary behavior of beams, is the most widely examined mechanism, this is considered as the building last line of defense against progressive collapse.(Adam et al. 2018).

A quantitative, transparent and rational approach to mitigate consequences from progressive or disproportionate collapse is based on risk reduction via structural robustness which allow either a new or an existing structure to meet additional performance objective of collapse prevention under abnormal loading. Based on principle, the actual robustness of a structure is a threat dependent property of the entire structural system because it depends not only on the system's characteristics (such as, strength, ductility, redundancy, continuity), but also on the type of abnormal loading (e.g. single or multiple, dynamic or impulsive monotonic or cyclic).

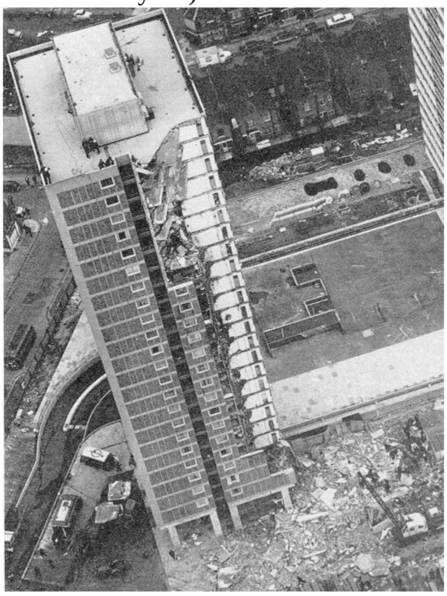

Figure 1.0: Ronan Point collapse: a gas explosion on the 18th Floor Resulted in Progressive Collapse

Investigations based on experimental test have made huge advancement in the area of progressive collapse, it had relatively improved the existing codes and design recommendations including calibrations of numerical models. The rapid availability of data of recent times depicts that more engineers are currently interested in this aspect of experimental study. In the cause of this research, the simulated failure of one or more columns were considered as threat independent scenario and are categorized into four types, such as;

(i) Sub-assemblages usually formed by two beams and one or more columns
(ii)Frames formed by beams and columns
(iii) Building structures constructed solely for experimental purpose
(iv) Actual building condemned to demolition.

1.2 Previous study on numerical models

The evolution of computers and software for structural calculations and analyses has made it possible to detail structural failures. Progressive collapse is a complex phenomenon involving non-linear material behaviours, impact and collision, deformation, and dynamic modes of failure have been studied using advanced computational methods. Computational techniques adopted so far include,

I. Finite Element Model (FEM)
II. Discrete Element Model (DEM)
III. Applied Element Model (AEM)
IV. Cohesive Element Model (CEM)

The accuracy and reliability of these models are ascertained by experimental test results and confirmed with standard codes.

Koh et al. (2016), adopted progressive collapse analysis to evaluate robustness of buildings experiencing sudden column loss. The approach is a highly nonlinear dynamic problem method that uses a "midway" scenario to evaluate for column buckling, semi rigid connection for end joints, and membrane action of slab. The accuracy of the efficient progressive collapse analysis (ePCA) is validated by using published experimental study. It was suggested that robustness can improve cost effectively by introducing minor change in steel connection and slab reinforcement, model has benefit of capturing distinct failure behaviour of the structural component. It was established that the robustness of composite building can be improved by change in deck thickness, amount of slab reinforcement and steel connection strength.

Li et al. (2018), studied the progressive collapse resistance of reinforced concrete (RC) beam-slab substructures using high performance solid element implemented in finite element models. The model was calibrated using material and geometric parameter of the experimental study. The results obtained were compared to the experimental data. The validated model was loaded and simulated under perimeter column removal scenario. It was observed that some parameters affect robustness of reinforced concrete beam-slab structures, such as, (i) lateral restraint stiffness (ii) loading procedure (iii) slab thickness, and (iv)) the reinforcement details in slab. The result shows that progressive collapse resistance of RC beam-slab substructure is developed primarily by actions such as, (i) compressive arch action of longitudinal beams, (ii) flexural mechanism of the transverse beam at small deflection (iii) catenary action of beams, (iv) tensile membrane action (TMA) of slab at large deflection.

1.3 Review on Finite element models

Finite element model is the most frequently used method in the numerical simulation of structures, and it is also primarily used for simulating progressive collapse of structures. FEM simulations for progressive collapse carried out using different levels of approximation and complexity, such as;

I. Using micro or macro models
II. With linear or non-linear analysis
III. Static or dynamic behaviours
IV. With 2-D or 3-D models, and
V. With implicit or explicit calculations.

The choice of approach depends on the type of analysis and size of structure, for instance, using micro or macro models depends on the dimension of the structure and the level of precision required of the analysis. Micro models are often preferred for small elements, while macro models are used to simulate the entire buildings. Non-linear simulation efficiently predicts structural failure with more accuracy compared to linear finite element analysis, although codes and recommendations allow either linear or non-linear analysis to be used. Innovations in computational calculation has made non-linear dynamic analysis which is very complex and having high computational cost been usable, it's accuracy over linear static analysis makes researchers to adopt it in the study of progressive collapse. The use of a three-dimensional model stimulation and a computational calculation applying explicit calculations is recommended by codes and regulation over two-dimensional and implicit calculation, for a robust and accurate result to be achieved.

Macro models is one of the most suitable options used for simulating an entire building using beam and shell element. It has a high computational efficiency and accuracy. Few researchers such as, Izzuddin et al. (2008), Kwasniewski (2010), Fu (2010), Li et al. (2016), Bermejo et al. (2017), Song and Sezen (2013), Vlassis et al. (2008) and Kunnath et al. (2017), have applied macro models in progressive collapse analysis. Beam type elements are usually used to simulate beams and columns, while shell elements are used to simulate floor slabs or steel columns of building structures. However, beam-slab and beam-column are the most complex zone of a building, the beam-slab behaves as catenary membranes and also suffer large rotations and deformations when subjected to shear, axial, and flexural loadings.

Previous experimental studies on progressive collapse of reinforced concrete (RC) beam-slab structures were mainly focused on the corner and interior column removal scenarios. Most results showed that slab can significantly enhance progressive collapse resistance. In recent times research has taken a new dimension on reinforced concrete (RC) beam-slab structures, and has been extended to perimeter column removal scenario.

Fu (2010), investigated progressive collapse using a 3-Dimensional non-linear finite element model. In this study ABAQUS software was employed to model a high rise composite steel framed building, which replicate a 20-storey structure subjected to column removal scenario. The responses of the structure were observed under the parametric measures including, steel strength, concrete strength, and reinforcement mesh size. Based on this studies the findings are, that,(i) the plasticity is observed to develop in the steel member under one column removal scenario, and the plastic hinge is formed at the beam (ii) cross bracing lateral restriction system is less vulnerable to progressive collapse (iii) The mesh size of the model has effect on the behavior of the structure (iv) increasing the steel mesh will increase deflection due to increased rotation capacity (v) for more column removal scenario increased mesh size, the ductile joint allow redistribution which enhances mitigation of progressive collapse.

Dat and Tan (2015), investigated three 1/3 scaled beam-slab structures under a penultimate perimeter column removal scenario. Parametric study was also carried out on effect of slab longitudinal reinforcement above the removed column.

Qian et al. (2016), carried out studies on two 1/4 scaled reinforced cncrete beam-slab structures. One of the specimens has a penultimate perimeter column removal scenario, while the other specimen was subjected to simultaneous removal of penultimate perimeter column and corner column. The two results show that beam-slab structures could develop tensile membrane action (TMA) to enhance progressive collapse resistance beyond that which was predicted by yield line theory.

Rahnavard et al. (2018), investigated progressive collapse resistance using three-dimensional model. Finite element method (FEM) was developed to study the progressive collapse of high-rise building**s** with composite steel frames. The nonlinear dynamic analysis examined the responses of the building under two column removal scenarios. Two different types of lateral resistance systems were analysed and compared. The buildings included regular and irregular plans. The response of the building was studied in detail, and measures are recommended to mitigate progressive collapse in future designs. The results of this study showed that side case removal in moment frame and moment with centrically braced frame systems was more destructive compared to corner case removal. It also showed that moment force in a regular building is greater than that of an irregular building.

Shan Gao (2019). Investigated the progressive collapse resistance using a nonlinear finite element analysis subjected to middle column removal scenario. In the study, the damage plasticity model (DPM) was used to define failure of concrete and the Von Mises failure criterion was used to model ductile fracture of steel reinforcement. The results from the study show that the stiffness in the column and increasing the rebar ratio in the RC slab enhanced resistance to failure the model. The bolts with good ductility was suggested to be used in the composite joint to enhance resistance potential of the reinforced concrete slab.

Yu et al. (2018), investigated progressive collapse resistance of reinforced concrete RC beam-slab substructures, using high performance solid element implemented in finite element model. The model was carefully calibrated using material parameters used in the experimental study. The exact model was replicated in numerical model. The model was validated using experimental data. The validated model was used to study the responses of structural components subjected to varying loading conditions. Various factors that influence the robustness of reinforced concrete beam-slab structures subjected to perimeter column removal scenario were investigated. Investigated parameters are; lateral restraint stiffness at the structure boundary, loading schemes in the testing, slab thickness, and reinforcement details in the slabs. The numerical results depict that progressive collapse resistance of the beam-slab structures were developed mainly by these actions, such as, (i) compressive arch action of longitudinal beams, (ii) flexural mechanisms of the transverse beam, (iii) catenary action of beams and (iv) tensile membrane action (TMA) of slab at large deflection. It was also concluded that the slab enhanced structural resistance to progressive collapse through L and T section compatibility.

1.4 Review on Design codes and recommendations

In response to the Ronan Point collapse incident (1968), which was caused by a natural gas explosion, the UK Building Regulations (BSI, 1997, 2000), aimed at ensuring the strengthening of structural integrity, to replace American and Canadian codes of practice (ASCE, 2002; NBCC, 1995). In spite of aforementioned efforts, the codes amended could not control the progressive collapse assessment methods used to analyze frames theoretical or speculative column removal. Based on the above, Oklahoma City bombing in 1995 and by the attack on the Twin Towers in 2001, events which influenced the introduction of progressive collapse assessment method guidelines by the US General Service Administration(GSA, 2003) and by the US Department of Defense (US DoD,2009), shows that there is need for control measures in the analytical procedures.

In this study, two codes for the progressive collapse are compared. The two codes are; Department of Defence (DoD) and General Service Administration (GSA).

Both codes used the Alternate Path method to satisfy the progressive collapse requirements for the removal of specific vertical load-bearing elements that are prescribed. Specifically, edge column removal.

The basic procedure in the alternative load path analysis given by US GSA and US DoD involves analyzing the damaged structure hinged on specific loadings, to check the possible initial damage propagation, basically introduced by theoretically or speculatively removing one primary load-bearing member at a time. The US GSA approach recommends the middle of the long side, middle of the short side and the corner of the building, only at the ground level, as locations of column removal (one at a time), whereas the locations of column removal specified in the US DoD approach are the same but columns at each floor level should be considered. However, there are four major analytical approaches for alternate load path analysis approved by the US GSA and the US DoD are; linear static, nonlinear static, linear dynamic and non-linear dynamic analysis.

Nevertheless, limitation of the alternative load path method depends only on the necessity of removal of one important element at a time, to examine the ability of the structure to redistribute loads without leading to a progressive collapse. Both the DoD and GSA guidelines use similar scenario-based methods to aid designers in avoiding progressive collapse; however, the DoD guidance also provides a tie force procedure to allow large deformations through catenary action (Ellingwood et al., 2009).

II. Materials and Method

In order to reduce cost intensive laboratory experiment, a detail non-linear finite element analysis (NLFEA), was employed to model the progressive collapse of a multi-storey subjected to mostly lateral (Horizontal) loading which carries excessive vertical displacement or failure, for assurance and reliability of adopted model, the nonlinear finite element analysis (NLFEA) was validated using published experimental set up on progressive collapse of a RC multi-storey building frame. Based on good correlation between the experimental and numerical results, the modelling procedure was implemented in this study.

2.1 Material Characterization

Design approaches are often carried out by specifications and is mainly applicable to structural building system. The specification requirements are primarily found in codes and recommendations in the form of additional detailing rules. Indirect design approach such as, tying detailing requirement and reinforcement stated in Euro code (2004), GSA (2003), DoD (2005) and others guidelines used in the Europe, United States, and Canada. Specification requirements are provided as guidelines to obtain relevant parameter that enhance robustness for different type of construction material.

Table 2.1: Material Model Adopted in NLFEA

Material	Model
Plain concrete Compression	TS- Fixed crack model (Thorenfeldt)
Plain concrete Tension	TS- Fixed crack model (Exponential)
Reinforced concrete	TS- Fixed crack model (Exponential &Thorenfeldt)
Reinforcement	Von Mises (Perfect Elastic)
Steel Plate	Linear Elastic
Concrete Column	Linear Elastic

2.3 Concrete compressive strength

Recently more priority is given to the cylinder strength than cube strength. Eurocode 2 and ACI adopted the cylinder strength. However, BS8110 retains the cube strength.

The cylinder strength is related to the cube strength as presented in Eq. 3.1

$f_{ck} = 0.8f_c$ (3.1)

The cylinder strength is adopted in the Thorenfeldt compression model. The characteristic strength is determined from the mean strength as,

$f_{ck} = f_m - 1.65\delta$ (3.2)

Where δ is the standard deviation.

2.4 Concrete tensile strength

The tensile governs the crack behaviour or response of the concrete. It affects the body displacement symmetrically

The tensile strength is derived from the compressive strength according to CEB FIP (1990) which is implemented in MIDAS FEA (1989) as presented in (3.3)

The mean tensile strength

$f_{ct,m} = f_{ct,ko,m}\left[\frac{f_{ck}}{f_{cko}}\right]^{2/3}$ (3.3)

Where, $f_{ctko,m}$ = 1.40N/mm^{2}and f_{cko} = 10N/mm^2

Lee et al. (2008), performed a direct tensile test on large plain normal weight concrete specimen, according to their test data, the tensile strength varied between; f_t = 0.27$\sqrt{}$fc and $f_t = 0.37\sqrt{fc}$

Where, f_t - tensile strength, and f_c– cylinder compressive strength

2.5 Fracture Energy in Tension

The fracture energy G_f (J/mm^2) is a material property and can be defined as the amount of energy required to create one unit area of crack surface. It can be determined by integrating the shear strain softening diagram as presented in Fig 3.1566

$$G_f = \int_t^{\delta} f\delta.\,d\delta.\,t \quad (3.4)$$

Where, ftδ denotes crack as a function ofcrack opening. According to CEB (1990) the fracture energy can be obtained as follow;

$$G_{fo} = 0.024 + \frac{0.0053A_g^{0.05}}{g} \quad (3.5)$$

$$G_f = G_{fo}\left(\frac{fcm}{fcmo}\right)^{0.7} \quad (3.6)$$

Where,

A_g - is the maximum aggregate size,

Gc – Fracture energy compression,

Gf - Parabolic compression,

Gc - is roughly approximated to value of 100.

2.6 Material Constitutive Models

This section explains the various models used in this investigation, some of such deisign models are, (Tensile model, concrete model, Thorenfeldt model, shear model, steel model and steel reinforcement model).

2.6.1 Tensile Model

At the point on the stress-strain curve, at the point where the tensile stress due to applied load exceeds the tensile strength of the concrete, exponential softening begins i.e, it is assumed that exponential softening occurs when the tensile stress exceeds the tensile strength of the concrete. The slope of the softening is a function of the fracture energy in tension (G_f) and the crack band width, as depicted in Fig 3.1.

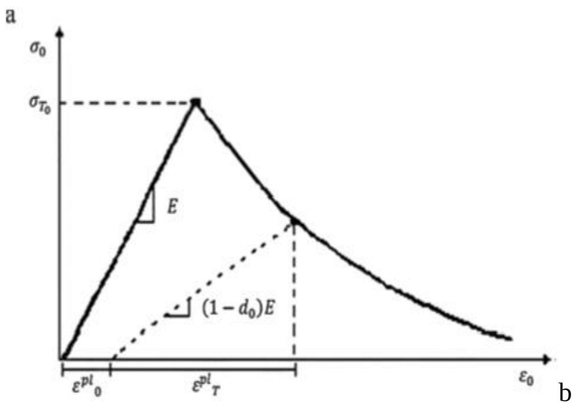

Figure 2.1 Concrete Tensile Behaviours

2.6.2 Concrete Model

The "Total strain crack model" (TS Model) was used to model the concrete, and its failure criterion or behavior is defined by using fixed crack concept. Fixed crack concept assumes that the axes of cracks remain fixed once crack axes are defined. It has been proved that fixed crack model accurately represents the physical characteristics of the crack phenomena more than the rotating crack model. The total strain crack model is based on the modified compression field theory of Vecchio and Collins (1986).

The TS model, models the mechanical behaviour of concrete using three sub-models, such as, Tensile, comprehensive, and shear model. See Fig. 3.2, for typical finite element compressive behaviour

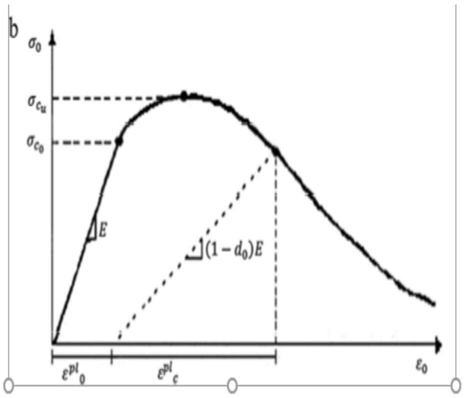

Fig 2.2 Concrete Compressive Behaviours v

2.6.3 Thorenfeldt Model

Concrete subjected to compressive stresses shows a pressure-dependent behavior, i.e., the strength and ductility increase with increasing isotropic stress. Due to the lateral confinement, the compressive stress–strain relationship is modified to incorporate the effects of the increased isotropic stress. Furthermore, it is assumed that the compressive behavior is influenced by lateral cracking. To model the lateral confinement effect, the parameters of the compressive stress–strain function, fc and α_c, are determined with a failure function. The failure function gives the compressive stress, which causes failure as a function of the confining stresses in the lateral directions. If the material is cracked in the lateral direction, the parameters are reduced with the factor $\beta_{\varepsilon\alpha}$ for the peak strain, and with the factor, $\beta_{f\sigma}$ for the peak stress. It is tacitly assumed that the base curve in compression is determined by the peak stress value and the corresponding peak strain value. The effect of these coefficients will be explained in detail in section

$$f_c = \beta_{f\sigma}\sigma_p \quad (3.7)$$

$$\alpha_c = \beta_{\varepsilon\alpha}\varepsilon_p \quad (3.8)$$

The base function in compression, with the parameters fp and αp, is modeled with a number of different predefined and user-defined curves. The predefined curves are the constant curve and the brittle curve, and the linear and exponential softening curves based on the compressive fracture energy, Gc. The linear hardening and the saturation hardening curves are available. Figure 2.3 shows the available hardening-softening curves in compression.

Hardening-softening lines, which are available to express compression, are divided into the parabolic, parabolic exponential, hardening curve suggested by Thorenfeldt et. al. (1987).

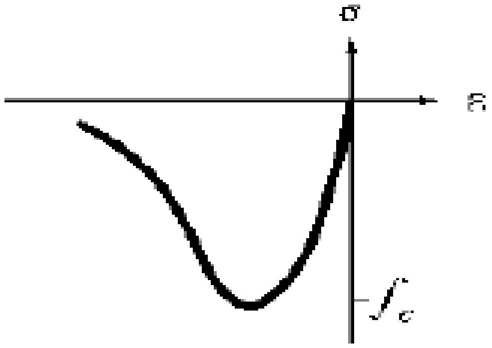

Figure 2.3 Thorenfeldt Compression Model

2.6.4 Steel model

The steel reinforcement was modelled as a Von Mises material. The Von Mises model is mostly applicable to define failure of ductile materials. It assumes that yielding occurs when a regular octahedral shear stress (ĭoct) reaches the limited which is formulated as,

$$f = \tau_{oct} - \frac{2k}{3} \qquad (3.9)$$

Expressing in terms of principal stress, $\sigma_1, \sigma_2, and\ \sigma_3$

$$(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 = 6k^2$$

(3.10)

Where K = yield stress under pure shear.

The Von Mises yield surface is depicted in Fig 3.3.

Considering the effect of isotropic steel hardening, the Von Mises hardening hypothesis which assumes that hardening progresses with increase in plastic deformation. The dependence of hardening parameter K on the plastic strain ε_p is defined in Equation 3.10

$$K = I\left[\frac{2}{3}\left(\varepsilon_{p1}\epsilon_{p1} + \varepsilon_{p2}\epsilon_{p3} + \varepsilon_{p3}\epsilon_{p1}\right)\right] (3.10)$$

2.6.5 Shear Model

The degradation in shear modulus (G) due to the progressive damage of concrete in the post cracked regime is modelled using the shear retention factor (β). It is a fraction of the uncracked shear modulus (Ĝ) of the concrete as presented in Equation(3.11)

$$\breve{G} = \beta G \qquad (3.11)$$

Where Ĝ is the non-cracked concrete shear modulus, β is shear retention factor, 0 ≤ β ≤1. For the rotating crack concept, the shear retention factor can be assumed equal to one.

2.6.6 Steel Reinforcement Modelling

Rather than defining reinforcement with distinct finite elements, the concept of embedded reinforcements was used in Midas finite element analysis (FEA). In this approach, the stiffness of the reinforcements is added to the reinforcement continuum element in which the reinforcement is embedded. This embedded reinforcement continuum element is also known as the mother elements.

The reinforcement and their mother elements are assumed to be perfectly bonded. Reinforcements strain are obtained from the displacements of its mother elements.

Table 2.2 (a) M aterial Properties for Concrete

S/NO	Properties	Values
1	*Elastic Modulus in Compression [Ec] (kN/mm^2)*	*35.5*
2	*Elastic Modulus in Tension [Et] (kN/mm^2)*	*35.5*
3	*Shear Modulus (G) (kN/mm^2)*	*22.188*
4	*Poisson's Ratio*	*0.2*
5	*Weight Density (ρ) (kN/mm^2)*	*2.4x10^4*
6	*Compressive Strength (fcu)*	*45*
7	*Fracture Energy in Tension (Gf) (J/mm^2)*	*0.1015*
8	*Fracture Energy in Compression (Gc) (J/mm^2)*	*10.15*
9	*Shear Retention Factor*	*0.2*
10	*Tensile Strength (ft) (N/mm^2)*	*2.21*

Table 2.2 (b) Material Properties for Steel

S/No	Properties	Value
1.	***Elastic Modulus (N/mm^2)***	***210***
2.	***Yield Stress (N/mm^2)***	***500***
3.	***Poison's Ratio***	***0.3***

2.7 Description of Experimental Study Used for Validation

The non-linear finite element model was validated using published experimental study of Lu et al. (2017). The authors tested four quarter scaled reinforced concrete (RC) beam-slab substructures with an additional companion beam-column substructure under a perimeter middle column removal scenario, see Fig 3.4 (a) & (b) for validation model and grid respectively. The dimension of the prototype structure and presented in Table 3.3.

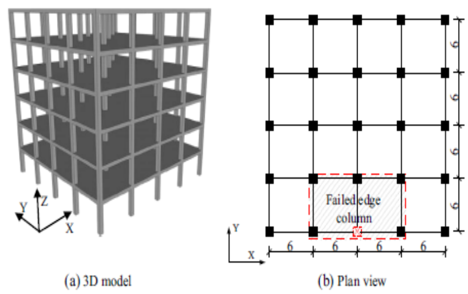

Figure 2.4 (a); 3-D Validation Model and (b) Plain View of the Grid

The prototype structure is a six-storey reinforced concrete RC frame, designed according to the Chinese code for the design of concrete structures (GB50011-2010-40 and the code for the seismic design of building). The first storey is

4.2m and the remaining storey is 3.6m in height. The span in both directions is 6m. The design dead and live loads are 5.0 kN/m^2 and $2.0kN/m^2$ respectively.

A two bay beam-slab substructure on the first floor was isolated from the entire structure, as shown and the column that experienced sudden removal is depicted in Fig 2.4 (b).

A quarter scaled ratio was adopted in order to obtain a manageable size that can be lifted by crane in the laboratory. In the real case, the test area is restrained by the surrounding slabs and beams. Taking cognisance of this, in the experiments, apart from the free edge, the other three perimeter edges of the specimen, both translational (dx, dy, dz) and rotational (Øx, Øy, Øz) were restrained to create ideal fixity. This type of restraint was achieved by adding three additional boundary beams as shown in Fig 3.5. For ease of lifting, a lifting beam was cast at the top of the specimen.

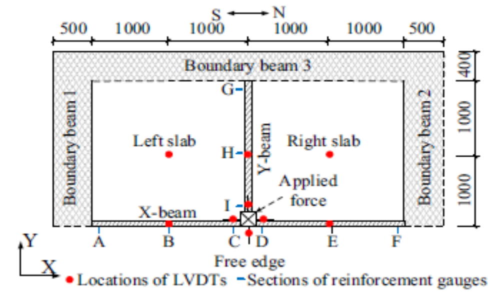

Fig 2.5: Specimen Tested in the Laboratory

2.8 Numerical Simulation Procedure

This section presents detail of the nonlinear finite element procedure employed for validation of the experimental study of Lu et al. 2017. The full scale of the beam–slab specimen described in the experiment was modelled, the beam-slab assembly was discretized into 8440 elements and 9687 nodes as shown in Fig 3.8

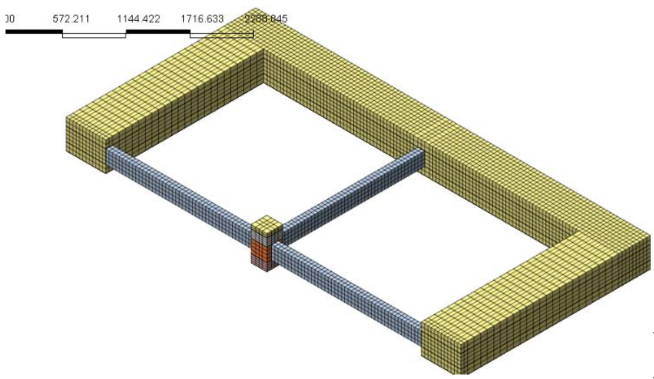

Figure 2.8 Validation Model

The mesh was refined within the slab vicinity where stresses and deformations could be significant. A full integration scheme was adopted throughout in order to overcome convergence problems. However, reinforcement concept was used, and a perfect bond was assumed between reinforcement and concrete. The translational (dx, dy, dz) and the rotational displacement (θx, θy, θz) were restrained at the base of the supporting

Column, in order to provide the ideal fixity in real life. Von Mises yield criterion with isotropic hardening was adopted for reinforcement modelling. Concrete was defined using Total strain (TS) crack model.

The column stub was modelled using three-dimensional solid element and defined with the elastic failure criterion. The concrete beam and column are modelled with high fidelity three-dimensional solid element. The Steel plate placed on top of the column stub is modelled with 3D solid element defined by Von Mises failure criterion. An incremental quasi-static displacement loading was applied at the top of the column stub and the corresponding load values were recorded. In the solution phase, Newton Raphson iteration scheme was adopted. A quasi-static displacement of 600 mm in 115 incremental load steps and 20 iterations was used. The Force norm and Energy norm was activated.

2.9 Description of the prototype Model of the Main Structure

The model is prepared using conventional dimension and geometric parameters widely used for multi-storey building. The prototype building used in this study, is a ten storey reinforced concrete RC frame building. It is made of unbraced column i.e it is expected that the column should resist both gravity and lateral loadings. All beams are 240mm x 240mm square section, supporting a slab of thickness 150mm. All columns are equally spaced at 3m throughout. For computational efficiency, the entire building was scaled down to 50%. The slab reinforcements were equally spaced at 148mm in both directions. See Fig 2.8 and Fig 2.9 for the prototype model and the grid respectively. The beam, column, and sab dimensions are illustrated in Tab 3.3 below.

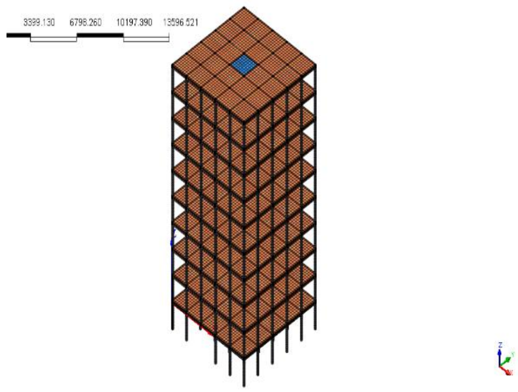

Fig. 2.9 Main Model Prototype

Description	Span	Beam	Column
Prototype Structure	6000	250X500	600X600
Specimen	2000	85X170	200X200

Table 2.3: Dimension of prototype structure and scaled specimen (unit: mm)

2.9.1 Model Loading Procedure

The loading procedure according to General Service Administration GSA (2013) require that, the model is loaded starting from zero and having a monotonic and proportionate increase of gravity load in the entire model (i.e. the column section has not been removed yet) until equilibrium is attained.

After equilibrium is reached for the frame structure, the column was removed. It is more preferable, to remove the column section instantaneously, the duration for removal is less than 0ne tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column as determined from the analytical model with the column removed.

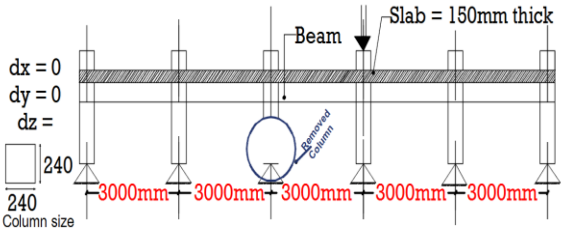

Fig. 3.10 Removed column and stub for loading

The analysis was then continued until the maximum displacement was reached or one cycle of vertical motion occurred at the column section removal location.

2.9.2 Model Loaded with Floor Slab

The model was in this case was designed with a floor slab and loading was then applied, first it was simulated without internal tie beam, thereafter simulated with an internal tie beam. The tie beam is embedded in the transverse direction of the model floor space. The responses of both scenarios were analysed and evaluated.

The concept of internal tie beam design and the maximum total tie strength is recommended according to Department of Defence, DoD (2013). See Fig. 3.11, for illustration. The design code also emphasized that, the tie beam should be embedded into the floor slab and not in the beam. The basic model approach of the internal tie beam is to distribute moment to the peripheral beams, to avoid excessive bending of the floor slab. The design model acceding to DoD (2013), assumes that the bending resistance of the internal tie beam on its floor slab is provided by the tie strength at the beam-wall were the tie beam is concealed. The tie beam is embedded at transversely at the centre of each floor space, it is provided by design code in DoD (2013), that the total tie strength is twice the required tie strength to provide adequate resistance. The tie beam is embedded either transversely or longitudinally and the width is provided to be 0.2LL for transverse embedment or 0.2LT for longitudinal embedment.

The gravity loading for the non-linear dynamic analysis for the entire building is expressed as

$$P_g = 1.2DL + 0.5LL \quad (3.12)$$

Where Pnd – Gravity load for non-linear dynamic analysis, D – Dead load including façade load (KN/m^2), L – Live load including live load reduction per section (kN/m^2).

2.9.3 Determination of Required Tie Strength and Maximum Total Tie Strength

A transversely embedded tie beam is designed, analyzed and evaluated in the model. Tie beams are generally designed sections that are fortified with reinforcement around the region where flexural bending is great, it enhances the floor in load bearing. The tie beam was connected to the opposite main beams in the space as depicted in Fig. 3.5. The end connections is designed with a specified maximum tie strength which is greater than required tie force as recommended by the General Service Administration GSA (2013), see (3.13).

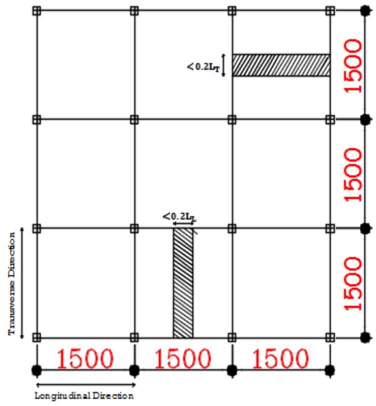

Fig. 3.11: Transverse and longitudinal Internal Ties with a uniform floor load. (0.5 scale)

For transversely embedded tie beam

Beam width is (0.2LL).

Thus, maximum total tie strength is equal to twice the required tie strength multiplied by width of tie beam.

$$T_{max} = 2 \times T_L \times 0.2L_L \quad (3.13)$$

Where,

Tmax – Maximum total Tie strength

But the required tie strength is expressed as,

$$TT_L = 3 \times W_F L_1 \quad (3.14)$$

Where,

WF – Floor load

L1 – Greater of the distance between adjacent floor spaces in the direction under Consideration.

2.9.4 Applied Floor Load after Column Removal

According to General Service Administration GSA (2013), a design strategy was introduced for loading the floor slab. Equ. 3.15 expresses the floor load after column removal,

$$P = 1.2DL + 0.5LL \quad (3.15)$$

Where,

DL (Dead Load) and LL (Live Load) were previously calculated to be 1168.54 kN and 810kN respectively.

III. Results and Discussions

This chapter presents the results and discussion on the non-line par finite element analysis progressive collapse of the prototype structure. Firstly, the result of the validation model is compared with published experimental data. Secondly, the results of the Elastic analysis us presented. Thirdly, the results of Nonlinear Finite element analysis are presented. The sensitivity studies such as: effect of (mesh density, compressive strength, tensile softening, and shear retention factor), and parametric studies such as, effect of (slab thickness, and reinforcement ratio.

3.1 Results of Validation Model

The result presented in the graph of Fig.4.2 shows that numerical data correlated appreciably with the experimental data. For instance, a maximum displacement of 574.62 mm was measured in the experiment compared to 561.66 mm obtained in numerical model. The both failure loads are compared. The measured failure load for the experiment is 50.48 kN while the numerical measured a value of 53.35 kN. The correlation of the published experimental data and that of the nodel's result showed a very close behaviour, and with this result in the comparison the model was adopted. The main model can be depicted in Fig 3.1

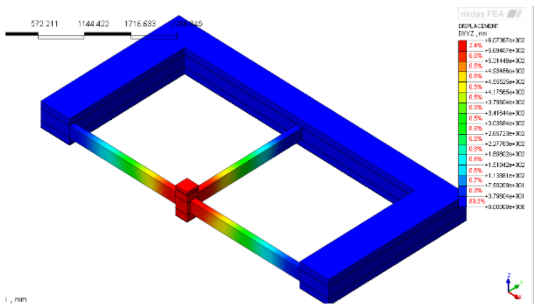

Fig 3.1: Main model

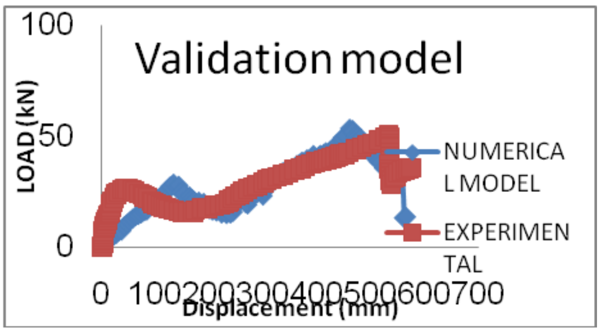

Fig.3.2: Comparison of numerical model with published experimental data

3.2 Nonlinear Finite Element Analysis (NLFEA)

3.2.1 Sensitivity studies

Prior to implementation of non-linear finite element analysis (NLFEA), various material models and parameters that influence failure at the connection are examined. Three types of tension softening model implemented in Midas finite element analysis (FEA), influence concrete tensile strength and shear retention are investigated and discussed.

3.2.2 Effect of Tensile Softening

Three types of tension softening models are investigated namely: Exponential, Hordijk and Brittle models as depicted in Fig.3.11. The Exponential and Hordijk softening models predicted failure in a very close range. In the exponential model, softening begins when the tensile stress due to applied load exceeds the tensile strength of the concrete. However, the Brittle model underestimated the failure load.

Based on this comparison, the exponential model was adopted.

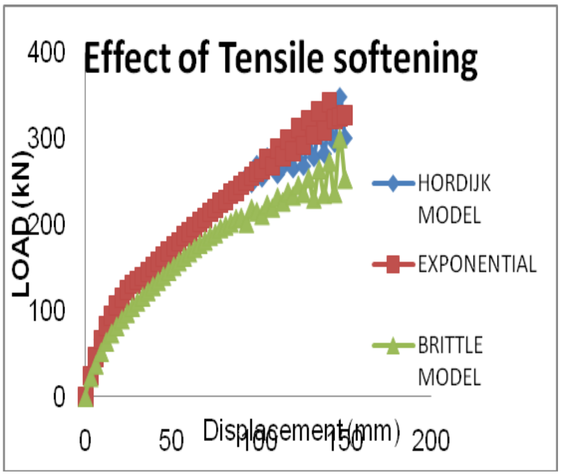

Fig. 3.5: Tension softening model

In similar studies by Fu F. and Lam D (2006), they carried out analysis on the tensile strength of concrete and it was concluded that the weaker the tensile strength the greater the maximum deflection. It was also observed that the tensile strength has significant effect on the building since

3.2.3 Effect of Slab Thickness

According to the investigation of Li et al. (2018), the effect of slab thickness using UDL loading scheme, it was evaluated, that, the resistance of the slab element to progressive collapse is higher with a greater slab thickness. The result in this study, using slab thickness of 50, 75, and 100mm, it comparatively shows similar characteristics.

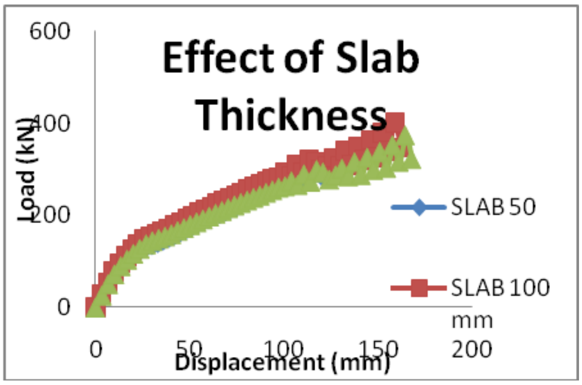

Fig.3.6: Effect of Slab Thickness

As shown in Fig.4.12 when the slab thickness was increased to 100 mm, the failure load increased to 402.63 kN at a displacement value of 158.80 mm. For slab thickness of 75 mm, the failure load of 372.8 kN at a displacement of 164.68 mm. For slab thickness of 50 mm the failure load of 350.87 kN and displacement 139.36 mm.

Based on these results, it can be concluded that as the slab thickness increases the robustness of the connection increases. Similar results and conclusion was also made by Yu et al. (2018)

3.2.4 Effect of Reinforcement Ratios

Fu F. (2010), observed that, the higher the steel grade, the higher the bending moment and axial forces and the lower the steel grade the higher the vertical deflection. It was then concluded in his study that, when plasticity has developed in the steel a higher grade steel exhibits a higher yielding and strain hardening stress, therefore, a higher bending moment and axial force.

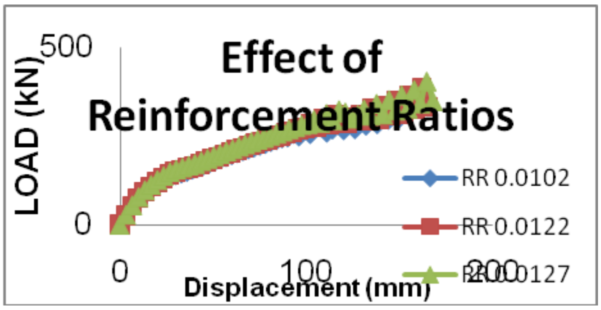

Fig 4.7: Effect of reinforcement ratio

In comparison to the model in this study, see Fig 4.2, for illustration. In the model different reinforcement spacing was selected, which increases or decreases the steel ratio. An increase steel ratio increases the reinforcement strength. It was observed that, higher steel ratio, decreases the maximum vertical deflection, and increases bending moment and axial forces. In other words, lower steel ratio, increases maximum vertical deflection, decreases beam bending moment, and lower axial force was observed.

In the observation from the model result, it can be concluded that, increasing the slab steel reinforcement ratio will increase the resistance capacity to progressive collapse or disproportionate collapse

IV. Conclusion

In this study, numerical models were used to investigate the response of reinforced concrete (RC) multi-storey building frame to extreme events. Progressive collapse of the structure was analyzed using the concept of column removal approved by various codes of practice on accidental design. Numerical analysis carried out include; Linear elastic analysis, eigenvalue analysis, Nonlinear finite element analysis.

The salient findings are highlighted as follows;

1. The NLFEA modelling scheme used was validated using published experimental study. Numerical result correlated appreciably with the experimental failure load and deflection values. Therefore, the modelling scheme was adopted.

2. Sensitivity study was on three tensile softening models (brittle, Hordijk and exponential) implemented in Midas FEA. The exponential model provides the best prediction of the failure load.

3. The degradation of shear stiffness due to the progressive damage of concrete in the post-cracking regime was captured using shear retention factor (β). The value β = 0.3 correlated well with the experimental results.

4. Robustness criteria provided in the design code (DoD, 2013) were used such as inclusion of tie-beam in the slab. It was observed that the inclusion of tie-beam enhances the resistance of the structure.

5. Increase in slab thickness enhances the resistance of the connection appreciably.

7. Increase in the material strength such as; compressive strength of concrete enhances the robustness of the connection.

V. Acknowledgement

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References

[1] Alogla, K., Weekes, L., && Augusthus-Nelson, L. (2016). A new mitigation scheme to resist progressive collapse of RC structures. Construction and Building Materials, 125, 533-545. *Construction and Building Materials*, 533-545.

[2] Baker, J. F. ((1948).). The Design of Framed Buildings against High-Explosive Bombs. In The civil engineer in war. *A symposium of papers on war-time engineering problems (pp*, (pp. 3-80)..

[3] Bao, Y., Kunnath, S. K., El-Tawil, S., && Lew, H. S. (2008). Macromodel-based simulation of progressive collapse: RC frame structures. Journal of Structural Engineering,. *Journal of structural engineeering*, 1079-1091.

[4] Bermajo, M., Santos, A. P., && Goicolea, J. M. (2017). Development of practical finite element models for collapse of reinforced concrete structures and experimental validation. *Shock and Vibration*, 1-9.

[5] Dat, P. X., && Tan, K. H. (2015 Dat, P. X., & Tan, K. H. (2014).). Experimental response of beam-slab substructures subject to penultimate-external column removal. *Journal of structural engineering*, , 141(7), 04014170.

[6] Deniz, D., Song, J., && Haijar, J. F. (2017). Energy-based seismic collapse criterion for ductile planar structural frames. *Engineering Structures*, 1-13 Deniz, D., Song, J., & Hajjar, J. F.

[7] Elsanadedy, H. M., Almusallam, T. H., Alharbi, Y. R., Al-salloum, Y. A., && Habbas , H. (2014 Elsanadedy, H. M., Almusallam, T. H., Alharbi, Y. R., Al-Salloum, Y. A., & Abbas, H.). Progressive collapse potential of a typical steel building due to blast attacks. *Journal of Constructional Steel Research*, pp 101, 143-157.

[8] Eurocode, C. E. (2003). Actions on structures, Part 2: Traffic loads on bridges. Brussels. *European Standard EN, 2*, 2.

[9] Fu, F. (2010 Fu, F.). 3-D nonlinear dynamic progressive collapse analysis of multi-storey steel composite frame buildings—Parametric study. *Engineering structures*, 32(12), 3974-3980.

[10] Fu, F. (2012). Response of a multi-storey steel composite building with concentric bracing under consecutive column removal scenarios. *Journal of constructional steel research*, 70, 115-126.

[11] Gao, S. (2019). Nonlinear finite element failure analysis of bolted steel-concrete. *Journal of Constructional Steel Research*, 62-76.

[12] Gsa, , u. (2003). Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. . Washington, DC.

[13] GSA. (2003). Progressive collapse analysis and design guidelines for new federal office building and major modrnization project. *The US General Service Administration*.

[14] Guo, L., Gao, S., & Fu, F. (2015). Structural performance of semi-rigid composite frame undercolumn loss. *Engineeering structures*, 112-126.

[15] Guo, L., Gao, s., Fu, F., & Wang, Y. (2013). Experimental study and numerical analysis of progressive collapsee resistance of composite frame. *Journal of construction steel research*, 236-251.

[16] Izzuddin, B. A., Vlyassis, A. G., & Elghazouli, A. Y. (2008). Progresssive Collapse of Multi-Storey Building Due to Sudden Column Loss Part 1. *Science Direct*, 1308-1318.

[17] Juan, dam, J. M., Prisi, F., Sagaseta, J., & Lu, X. (2018). Kamal et al. (2016), carried out study on resistance against progressive collapse on a reinforced concrete frame structures. This involves the provision of additional reinforcement bars in the mid-layer of reinforced concrete beams. In the investigation,. *Engineering Structures*, 122-149.*ructures*, 1223-1235.

[18] Li, S., Shan, S., Zhai, C., & Xei, L. (2016). Experimental and numerical study on progressive collapse process of RC frames with full-height infill walls. *Engineering failure analysis*, 57-68.

[19] Liu, J. I. (2010). Preventing progressive collapse through strengthening beam-to-column connection, Part 1 Theoretical analysis. . In *Journal of Constructional Steel Research*, (pp. pp 229-237.).

[20] Lu, X. L. (.2013). . Earthquake-induced collapse simulation of a super-tall mega-braced frame-core tube building. *Journal of Constructional Steel Research*,, 82, 59-71.
[21] Lu, X., Lin, K., Li, Y., Guan, H., Ren, P., && Zhou, Y. ((2017)). Experimental investigation of RC beam-slab substructures against progressive collapse subject to an edge-column-removal scenario. *Engineering StructureS*, 91-103, 149.
[22] Orton, S. (2007). Development of a CFRP *Journal of structural engineering*, 1-13.
[23] Sasani, M. (2008). Responses of a reinforced concrete infilled-frame structure to removal of two adjacent column. *Engineering structures*, 2478-2491.
[24] Sasani, M., & Sagiroglu, S. (2008). Progressive collapse resistance of hotel San Diego. *Journal of structural engineering* , 478-488.
[25] Sasani, M., & Sagiroglu, S. (2010). Gravity load redistributionand progressive collapse resistance of 20-storey reinforced concrete structures following loss of interior column. *ACI structural journal*, 636-644.
[26] Sasani, M., Bazan, M., & Sagiroglu, S. (2007). Experimental and analytical progressive collapse evaluation of actual reinforced concrete structures. *ACI structural journals*, 731-739.
[27] Sasani, M., Kazemi, A., Sagiroglu, S., & Forest, S. (2011). Progressive collapse resistance of an actual 11-storey structure subjected to severe initial damage. *Journal of structural engineering*, 893-902.
[28] Shan, S., Li, S., Xu, S., & Xie, L. (2016). Experimental study on the progressive collaose performance of RC frame with infill walls . *Engineering structures* , 80-92.
[29] Smith, P. P., Byfield, M. P., && Goode, D. J. (2010). Building robustness research during World War II. *Journal of Performance of Constructed Facilities*,, 24(6), 529-535.
[30] Song, B. I., & Sezen, H. (2013). Experimental and analytical orogressive collapse assessment of a steel frame building. *Engineering structures*, 664-672.
[31] Song, B. I., Giriunas, K. A., & Sezen , H. (2014). Progressive collapse testing and analysis of a steel frame building. *Journal of construction steel research*, 76-83.
[32] Sterossek, U. (2007). Typology of Progressive Collapse. *Engineering Structures*, 2302-2307.
[33] Tay, C. G., Ekoh, C. G., & Liew, J. Y. (2016). Efficient progressive collapse analysis for robustness evealuation of building experiencing column removal. *Journal of constructional still research*, 395-408.
[34] Kunnath, S., Li, F. W., Zhao, Y. B., Lew, H. S., & Bao, Y. (2015). Collapse test of three storey half scale reinforced concrete frame building. *ACI structural journal*, 429-438.
[35] Xu, G., & Ellingwood, B. R. (2011). Probabilistic robustness assessment of pre-Northridge steeel moment resisting frames. *Journals of structural engineers*, pp. 925-934.
[36] Yang, B. T. (2016). Experimental study about composite frames under an internal column-removal scenario. . *Journal of Constructional Steel Research*, 121,341-351.
[37] Yang, B. T. (2016). . Experimental study about composite frames under an internal column-removal scenario. *Journal of Constructional Steel Research*, pp 121, 341-351.